Design of geosynthetics over areas prone to subsidence

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ABSTRACT: Sinkholes can progress due to natural or human related activities causing problems for local communities, business and rail and road infrastructure. The use of high strength geosynthetics instead of traditional solutions could offer considerable sustainable and economic benefits and protection against unexpected collapse caused by subsidence. A comparison of the tension and strain requirements predicted by BS 8006 and EBGEO (RAFAEL method) indicated that BS 8006 predicted slightly higher tensions but that the trend was similar. Both design methods also showed that the required tension in the geogrid reinforcement increased exponentially while the design strain reduced as the void width increased relative to the height of the embankment. The reduction in the design strain was attributed to the need to restrict surface deformations to a pre-defined limit. The optimal embankment height to void width was found to be close to unity in both methods. The analysis also found that there was no advantage in using a high stiffness geogrid with a low ultimate strain in this application. The optimal design strain was found to be 3%-6%, corresponding to an ultimate strain in the geogrid of 8%-14%. Two recent UK case studies were geogrid reinforcement was used over areas prone to subsidence are also presented. The first from 2017, was a housing development over a former clay workings, while the second was a housing development over chalk solution features. The design philosophy and the final solutions adopted in each case study are presented and discussed.

Keywords: sinkhole; voids; uniaxial; geogrid;

1 INTRODUCTION

Subsidence is the sinking, or caving in, of the ground, or the settling of a structure, essentially due to the removal of support beneath the ground surface. Voids can result from either the presence of underground caverns caused by natural processes (e.g. soil erosion in karstic areas) or from man-made processes such as underground mining.

In areas under threat of collapse due to the formation of a void, a high strength geogrid reinforcement placed at the base of an embankment is now considered an accepted foundation engineering technique. The geogrid reinforcement is designed to prevent catastrophic collapse of the embankment and to prevent unacceptable surface deformations which may occur during the design life of the structure. Typically, the geogrid is designed for a 120 years design life. Generally, this method is used for infrastructure embankment supported over feature areas, however, geosynthetics have been used to span features within housing developments located in former quarries or historical mines.

2 COMPARISON OF BS8006-1:2010+A1:2016 AND EBGEO:2011

Two widely used design approaches in this application are BS 8006-1:2010+A1:2016, the UK design code for reinforced soil and the EBGEO:2011, the German code for reinforced soil.

2.1 *British Standard BS 8006-1:2010+A1:2016*

The BS 8006-1:2010+A1:2016 design approach assumes a constant volume of soil in the zone of the depression, which itself is assumed to be an inverted truncated wedge (longitudinal subsidence area) or cone (axisymmetric subsidence area). The dimension of the zone of depression can be quantified in terms of the vertical depth of the surface depression, d_s, and the deformation width at the surface of the embankment, D_s, as shown on Figure 1. In BS 8006-1:2010+A1:2016 the angle of draw, θ_d, is equal to the angle of friction of the fill above the geogrid reinforcement.

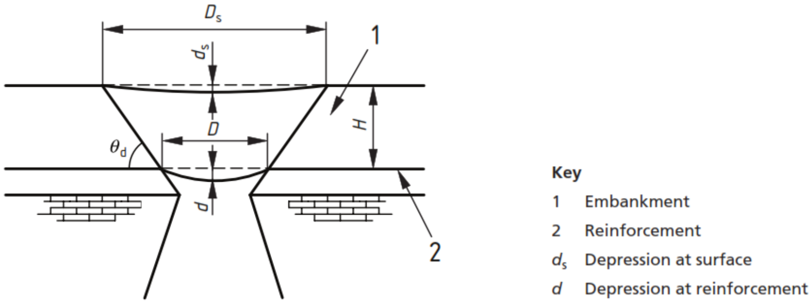

Figure 1. Definition of d_s and D_s in BS 8006, after BS 8006-1:2010+A1:2016

2.2 *German EBGEO (2011) standard*

EBGEO (2011) presents two methods for the design of geogrid reinforcement to limit the magnitude of surface deformations caused by subsidence. This paper considers the design approach adopted from the RAFAEL method only. The RAFAEL model assumes a cylindrical failure body forms in the fill above the geogrid reinforcement layer and that this cylinder has the same width as the void at the level of the geogrid reinforcement, Figure 2.

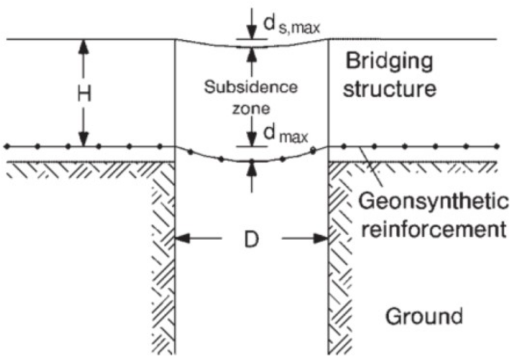

Figure 2. RAFAEL analysis model in EBGEO:2011

2.3 *Predicted reinforcement tensions and strains given by BS 8006-1:2010+A1:2016 and EBGEO:2011*

In designing geogrid reinforcement over areas prone to subsidence the design philosophy is to iterate and adjust the ratio d_s/D_s and the strain in the reinforcement until the short-term strength requirement in the reinforcement is less than, but close, to the available short-term geogrid strength capacity. The design problem involves a complex interaction between the fill/foundation properties, the fill/void geometry and the reinforcement properties (Naughton & Kempton, 2004). For motorway and other principal roads, the d_s/D_s ratio is usually limited to a maximum of 1% (BS 8006-1:2010+A1:2016 & EBGEO:2011). Greater d_s/D_s ratios way also be acceptable but this is subject to design requirements, particularly the allowable surface deformation.

The geogrid reinforcement design tensions and corresponding strains predicted by BS 8006-1:2010+A1:2016 and EBGEO:2011 for a range of longitudinal void widths and embankments heights is presented in this paper. The d_s/D_s ratio was limited to 1% and the design parameters listed in Table 1 were used in the analysis. Figure 3 shows a general comparison of the predicted geogrid tensions from both design approaches examined. While the trend was the same, BS 8006-1:2010+A1:2016 predicted slightly higher tensions than EBGEO:2011 (RAFAEL method).

Figure 4 presents the analysis for a 4m high embankment, which was representative of the overall analysis, with both BS 8006 and the RAFAEL methods predicting similar tensions and strains. The overall trend was also representative of the analysis and was consistent between the two design methods: as the void width increased, the geogrid tension increased exponentially and the design strain reduced. The reduction in the design strain was directly attributable to the need to maintain d_s/D_s = 1%, therefore, requiring a reduction in the design strain in the geogrid reinforcement. An interesting trend was observed for all embankment heights examined: for a void-width less than the height of the embankment the predicted reinforcement tension was in the range of currently manufactured geogrids (< 1,600kN/m) and the strains were in the range 3 – 6%, with 6% being the maximum allowable design strain (BS 8006-1:2010+A1:2016). Where the void width was greater than the embankment height the required geogrid reinforcement tension increased dramatically, which resulted from using a significantly lower design strain (>3%). Again, the lower design strain in the geogrid reinforcement was required to limit the d_s/D_s = 1%. The predicted geogrid reinforcement tensions and strains were independent of the type of geogrid used.

Table 1. Design parameter for analysis

Parameter	Value
Unit weight of the fill	19kN/m^3
Angle of internal friction of the fill	30deg
Cohesion	0
Surcharge loading	30kPa
Height of the fill above the reinforcement	Varies between 1m to 10m
Void width	Varies between 1m to 10m
Geosynthetic Reinforcement characteristics - 120 years at design temperature of 20ºC, 4 ≤ Ph ≤ 9.5 and ramification failure fn = 1.1 (Category 3 - high)	Partial Factors (BBA certificate)
Creep	1.39
Installation damage	1.02
Durability	1.1

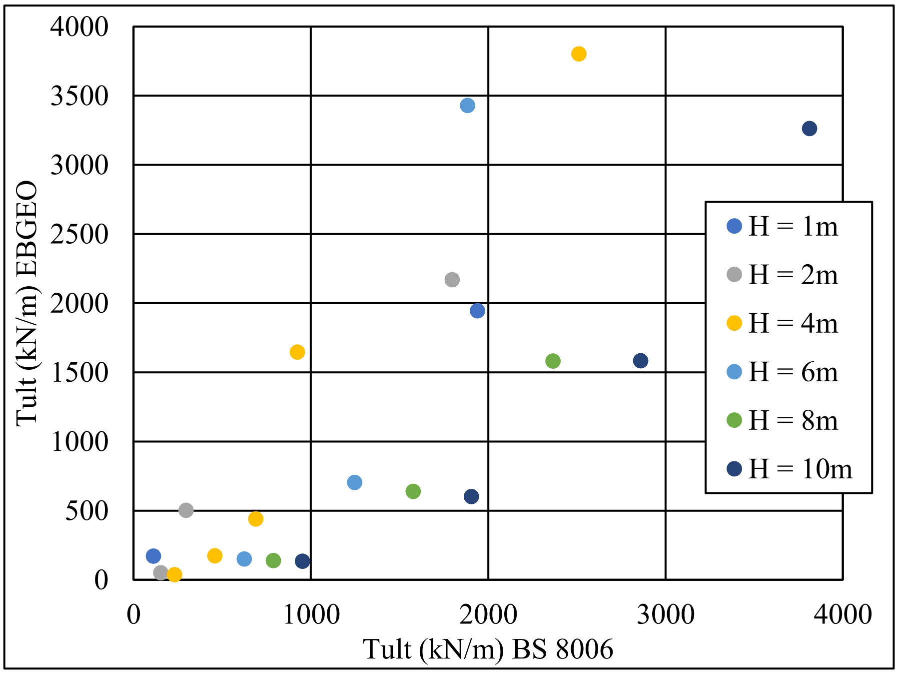

Figure 3. General comparison of predicted short-term geogrid tensions from BS 8006-1:2010+A1:2016 and EBGEO:2011 (RAFAEL method)

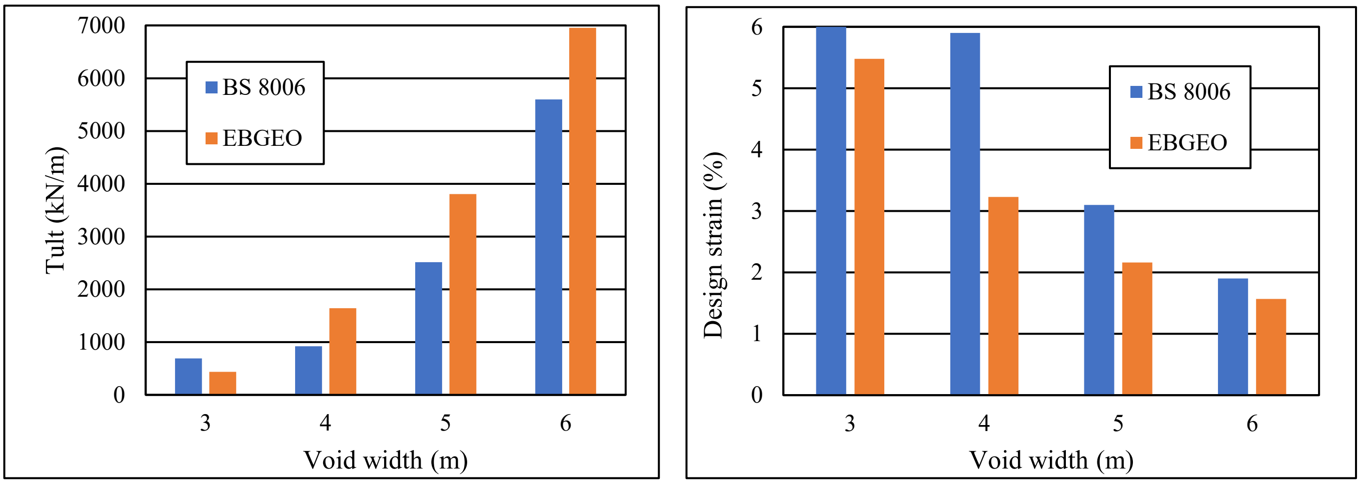

Figure 4. (a) Predicted short-term geogrid tension and (b) design strain for a 4m high embankment over different size voids

The analysis indicated that the most economic benefit of using geogrid reinforcement was achieved when the potential void width was less than the embankment height. In this situation, the design strain in the reinforcement was in the range 3% - 6%. Limiting the strain to less than 3% resulted in an exponential increase in the reinforced short-term geogrid strength, which was not, either economic or practical, to achieve. Therefore, geogrids with high stiffness and modest ultimate strains (in the range 8% - 14%) are preferred to very stiff geogrid (working strain < 3%) that have low ultimate strains (less than 8%).

2.4 *Considerations in selecting partial load factors*

The short-term ultimate tensile strength of the geosynthetic would be dependent not only on its stress-strain characteristics, but also on the design life of the project, creep, installation damage and environmental durability (e.g. pH). Many reinforcement materials that present good short-term strength and deformation characteristics, sometimes show poor viscous behavior under static loads in the long-term. Partial factors for creep must be applied to account for deformation associated with creep strain of the geogrid over the design life (typically 120 years). This creep-strain deformation is estimated to be 20-40% of the initial geogrid reinforcement deformation. BS 8006-1:2010+A1:2016 also restrictions the magnitude of creep strain over the design life to 2%.

In relation to damage, generally, geogrid reinforcement over potential voids will experience two types of damage:

1. Installation damage resulting from placing the geogrid reinforcement in the ground and compacting material over it on site.
2. Damage to the geogrid reinforcement caused by subsidence and the potential immediate response of the geogrid reinforcement to carrying load. The geogrid reinforcement on the edge of the void may move inwards towards the void causing scrapping between the soil and the reinforcement. The potential for damage is very high in that case and the consequence could be dramatic. Uncoated PET materials, in particular, must be property assessed for this type of damage.

Geogrid reinforcement manufactured from polymer such as PVA or uncoated PET when placed into ground with high watertables could suffer a reduction in strength due to hydrolysis. PVA is a biodegradable polymer, and its degradability is enhanced through hydrolysis because of the presence of hydroxyl groups on the carbon atoms. Moreover, it is water-soluble and has a hydrophilic nature which can accelerate environmental degradation. Rates and environmental conditions for degradation may vary for other polymers, but must be considered carefully in design.

3 HOUSING DEVELOPMENT OVER MINING FEATURES AT UNDISCLOSED LOCATIONS IN THE UK

3.1 *Development over a former clay working*

In 2017 a large UK based housing developer proposed a new development over a former clay works. The main section of the site comprised an area of historic ball clay and deep pit workings which have historically been infilled with non-engineered fill materials (clay waste), Figure 5. Historically, these clay workings were backfilled with end-tipped naturally occurring clay mineral waste to depths of between 3m and 9m below existing ground level. As part of the main scope of redevelopment works, an earthworks program was designed to remove this clay waste to the level of the natural clay, with the material then being replaced and compacted in accordance with the UK Specification for Highway Works by means of a site-specific earthworks specification.

Upon commencement of the works within the northern areas of the site, numerous and extensive historic mine features with diameters of in the order of 4m (including square pits and adits/roadways) were encountered, with these mine features initially extending to depths of between 4.5m and 5.5m below existing ground level. The mine features varied in width and were generally backfilled with a loose, clayey, sandy gravel, which was typical of this type of feature and dates the workings back to the 1800's.

Following the identification of such a volume of mine working features, an engineered solution was required to mitigate against any future potential subsidence/settlement from the identified and remaining unidentified mine features. Many potential solutions were examined to mitigate potential risks from the remaining mine workings, including sterilization, improvement techniques (including dynamic compaction, drill and grout and vibro-compaction), removal and piling. These solutions were ruled out on either a cost, time, program or viability basis.

Figure 5. (a) Remain of mine props found on site & (b) Excavation operation over the mining features

The preferred engineering option was the installation of a reinforced geogrid mattress, which would span the identified and unidentified mine features founded on the natural ground surrounding the backfilled mine features. This mattress was to be constructed using a high strength geogrid buried with a minimum of 5m of engineered fill to provide additional strength and stiffness should any movement within the underlying ground occur, Figure 6.

A geotechnical assessment of the proposed house foundations was undertaken using the finite element suite of software - Plaxis 3D. The purpose of the analysis was to show the impact that a void forming directly beneath the reinforced mattress would have on the settlement of the foundations. In the finite element modelling (FEM), the engineered fill was modelled using a Mohr-Coulomb constitutive model, and was undertaken with Serviceability Limit State (SLS) conditions using unfactored soil parameters. The soil was modelled using undrained or short-term material behavior in which stiffness and strength were defined in terms of effective stress properties. The geosynthetic reinforcement was modelled using the geogrid element in Plaxis, with the stiffness parameters derived from the short-term stress-strain curve for the geogrid reinforcement.

The soil and structural parameters used in the PLAXIS model are summarized in Tables 1 and 2 and were based on the findings and information provided by the client.

Table 2. Soil parameter used within the FEM model

Element	Bulk Unit Weight γ_b (kN/m^3)	Undrained shear strength/cohesion c_u/c' (kPa)	Friction angle φ (^{o})	Young's Modulus E_u / E' (MPa)
Engineered Fill – Class 2C	19	50 / 5	27	12 / 9
Natural Ground	20	75 / 5	21	24 / 18

Table 3. Parameters of the structural element within the FEM model

Element	Bulk Unit Weight γ_b (kN/m^3)	Plate Thickness (m)	Stiffness, E (kN/m^2)	Possion's ratio v	EA_1(MPa)
Foundation beams	25	0.5	28×10^6	0.15	-
Uniaxial Geogrid (1,600kN/m)	-	-	-	-	15,253

Ground level was taken as 0mOD with groundwater at -5mOD and the foundation beams were modelled as plates with a pro-rata thickness to provide a representative flexural thickness. The thickness of the engineering fill was taken as 5m minimum and the geogrid was installed at the interface of the engineering fill and underlying natural soil at -5mOD. Voids of 3.5m^2, 4.0m^2 and 5.0m^2 of 0.15m depth were modelled with the surface of the void modelled at -5mOD directly beneath the geogrid. The voids were positioned beneath a corner of the house footprint. A schematic of the FEM model is shown in Figure 5.

The design assumed an allowable bearing pressure of 60kN/m² over the ground floor of the houses, with line loads calculated based on the width of the concrete foundations. The analysis was based on line loads on footings all being fully realized which would not be the case during the design life of the house.

The construction sequence was analysed in four stages within the FEM model. The first stage involved building the ground model and solving for the initial state with only the ground and groundwater profiles activated. This generated the initial soil stresses. The second stage involved the installation of foundation beams and activation of line loads which caused settlement of the foundations, and the installation of the geogrid. The third stage involved the consolidation of the model following construction, assuming a scenario where a potential void occurred in the future. The final stage involved resetting the displacements to zero and deactivation of the void profile which caused settlement of the geogrid and foundations.

The results of the three analyses for a geogrid with a short-term strength of 1600kN/m is summarized in Table 4, with a graphical output of the PLAXIS analysis shown in Figure 6. The predicted ground movements indicated that a 1,600kN/m uniaxial geogrid would limit potential settlement of the foundations at ground level to 25mm and result in a predicted differential settlement of less than 1 in 500 should a void of 4.0m^2 plan dimension or less open beneath the geogrid. However, the predicted settlement for a 5m^2 plan dimension void exceeded the UK National House Building Council (NHBC) threshold of 25mm settlement.

The predicted axial forces in the geogrid reinforcement predicted by PLAXIS were significantly lower than those calculated by BS 8006-1:2010+A1:2016 which agrees with published data on the topic (Zhuang et Al., 2014). It also reflects that the finite element analysis was acting as a continuum model.

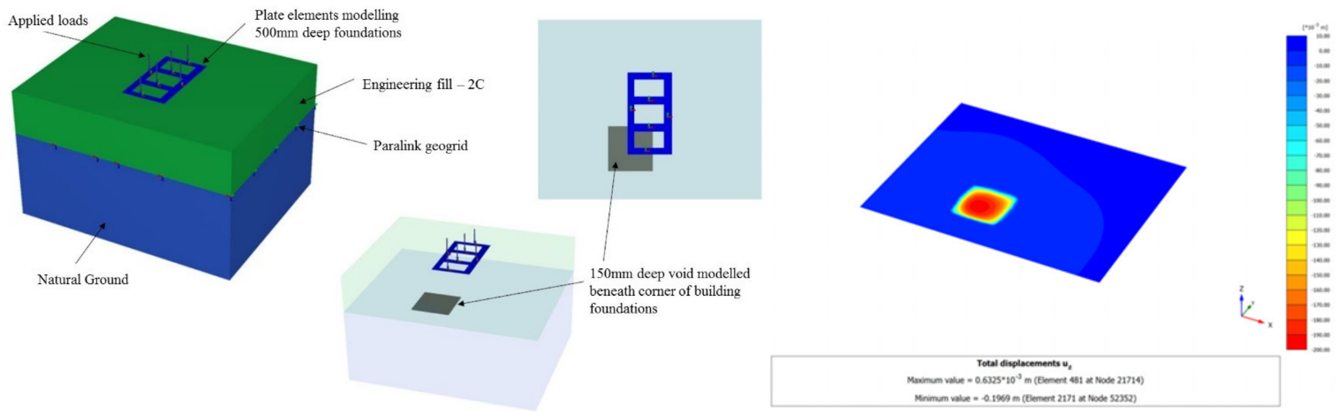

Figure 6. (a) FEM model setting up & (b) Typical FEM output considering a void diameter of 5.0m

Table 4. Summary of FEM analysis results

Void Diameter (m)	Displacement of foundation (mm) [Max. differential displacement]*	EA_1(MPa) [E']
3.5m	-8 to +1 [9]	-26 to +1 [27]
4.0m	-19 to +3 [22]	-38 to +2 [40]
5.0m	-127 to +17 [144]	-197 to +6 [203]

*positive values denote heave, negative values denote settlement

When assuming an embankment height of 4.5m to 5.0m and designing in accordance with BS 8006-1:2010+A1:2016 the maximum circular void diameter when limiting differential settlement to 1 in 500 with a 1,600kN/m uniaxial geogrid is 4m, which is also consistent with the FEM modelling results.

Further FEM analysis, together with a specific and controlled earthworks specification, demonstrated that a 1,350kN/m uniaxial geogrid could also be used for a maximum void span of 4.0m assuming an embankment height of approximately 7m was present between the geogrid and underside of the house foundations, while limiting the surface deflections across the strip foundations to less than 25mm.

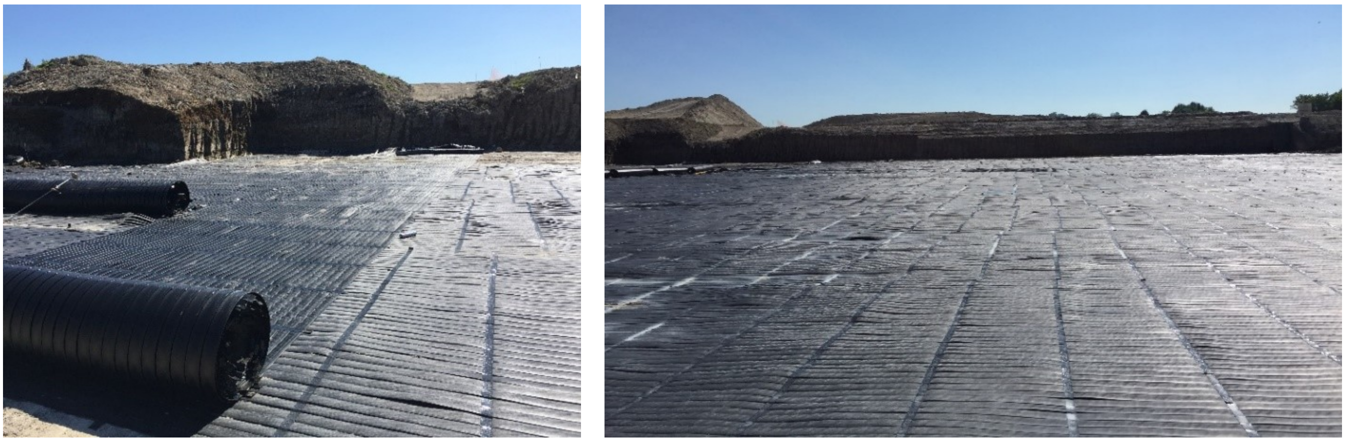

Figure 7. The final solution considers two perpendicular layers of 1,350kN/m geosynthetic mattress laid across the site (a total of approx. 20,000sqm of uniaxial high strength geogrid where supplied)

During construction, Figure 7, voids greater than 4m in diameter were compacted with a suitable vibratory roller and resultant voids caused by compaction were infilled with engineered fill as per earthworks specifications. The finished platform was also subject to a 50-day maintained load test on completion.

3.2 *Development over chalk features*

A new development of up to 170 houses along with up to 2,000m^2 of retail space, a community hub, public open space, sustainable urban drainage, earthworks, structural planting, substations and associated infrastructure is planned to be construct over an area of chalk in the UK.

The site was formerly a farm and associated yard, with farm buildings in the western quarter of the site. A large part of the site was quarried for gravel and chalk during the early 1960s. During the early 1980s the site was turned into a depot. Prior to that, in 1879, chalk extraction occurred in the northwest and southeast of the site. Excavations in surrounding villages have encountered unrecorded underground workings for chalk extraction and there was a potential risk for voids and underground Chalk mining to be present below the site, Figure 8.

The Chalk was found to be affected by the formation of solution cavities, with the solution features infilled with loose River Terrace deposits. The site investigation identified solution features up to 22m deep, which had been backfilled with loose gravels. The solution features identified to date were between 0.6m and 6.0m in diameter. To mitigate the risks of loose ground and voids from the identified solution features and to meet the low risk requirements of the Client, it was recommended that excavation to the top of the chalk was undertaken. Any solution features identified would be grouted and suitable excavated soils replaced and compacted to a structural fill specification (including the placement of a geogrid) in accordance with the UK Specification for Highways Works. The preferred option was to strip all overburden above the chalk from across the site to identify potential infilled solution features.

Where such features were identified, probing methods would be used to determine the density of the infill. Very loose or loose features would be pressure grouted to remove the risk of collapse. However, the pressure grouting solution was considered expensive.

Thus, an alternative solution using high strength uniaxial geogrids was designed and installed on the surface of the excavated chalk, spanning any loose infill materials. The geogrid was covered with between 2.0m and 4.0m of engineered fill.

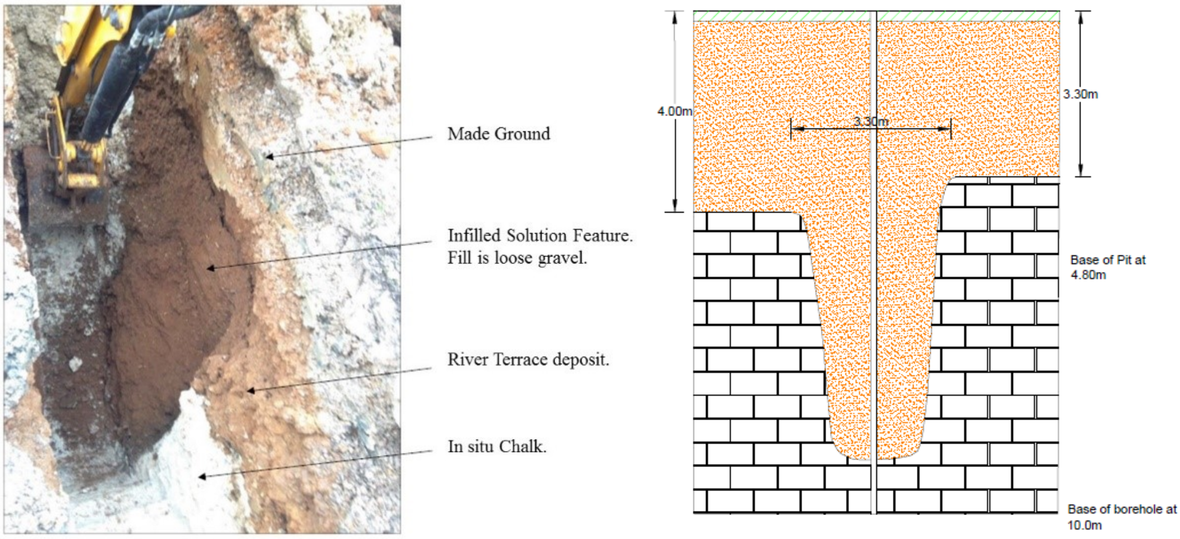

Figure 8. (a) Typical feature stratigraphy found on site & (b) Typical section of the shaft

A design approach considering a shallow embankment was then undertaken. A plane strain approach (with one layer of geogrid reinforcement used for up to 3.3m diameter voids). For larger diameter voids (between 3.3m and 6.0m in diameter) an axisymmetric layout consisting of two orthogonal layers of geogrid reinforcement to control both the anchorage length and the surface deflection was selected. The design in areas of both known and possible solution features was based on a ds/Ds ratio of 2%, in accordance with BS 8006-1:2010+A1:2016. In this design, it was assumed that the existing ground level would be over excavated to create an engineered fill overburden thickness of at least 2.0m over the geogrid reinforcement. The geogrid reinforcement was designed for a design life of 120 years. The over excavation or lowering of the exiting ground level is often required in these of project. The principle benefit is to increase the overburden height and thus change the embankment height (H) to void diameter (D) ratio. The optimum H/D ratio is close to unity.

Table 5. Summary of analysis performed with BS8006:2010

Void diameter (m)	Embankment overburden (m)	Required short-term tension (kN/m)
0.6	2.0	127
1.4	1.0	174
3.3	2.0	662
2.0	2.7	514
2.0	2.7	514
2.0	2.0	598
6.0	2.0	808

4 CONCLUSION

BS 8006-1:2010+A1:2016 and EBGEO:2011 provide simplified methods for designing geogrid reinforcement over areas prone to subsidence. The analysis conducted in this paper showed that BS 8006-1 and EBGEO:2011 (using the RAFAEL method) generate similar outputs: as the void width increased the geogrid reinforcement tension increased and the design strain decreased. While the trend was the same, BS 8006-1:2010+A1:2016 predicted slightly higher tensions than EBGEO:2011.

Both methods predicted very high tensions in the geogrid reinforcement particularly where the height of the embankment was low relative to the width of the void. The maximum void width that could be spanned was governed by the ultimate short-term tensile strength of the geosynthetic and the maximum allowable strain in the geosynthetic to control surface deformations. The analysis presented in this paper showed that the optimum ratio of embankment height to void width was close to unity. Where the width of the void was large relative to the height of the embankment the required strength of the geogrid reinforcement was very high and it was considered neither economic or practical to use a geogrid reinforcement solution. The high geogrid tensions came directly from the need to reduce the design strain in the geogrid reinforcement to maintain the desired d_s/D_s ratio. The analysis indicated that the optimal design strain was in the range 3% – 6%, indicating that a geogrid reinforcement with a short-term ultimate strain in the range 8% – 14% would suit this application best. Using very stiff geogrid, with low ultimate strain < 8% are not practical as the required short-term strength (> 1,600 kN/m) is not achievable using current manufacturing processes.

FEM modelling, even if sensitive to the input data and on the geotechnical model chosen by the user, could provide a significant benefit in evaluating the expected maximum settlement and the deformation limits, saving in the required strength of the reinforcement, while still controlling the desired d_s/D_s ratio.

BS 8006-1:2010+A1:2016 or EBGEO:2011 are useful for a preliminary analysis, while FEM modelling can be used to finalize the design, particularly where the design requires strict surface deformation limits. FEM predicted the smallest surface deformations, which can be attributed to a more rigorous modelling of the fill directly over the potential void.

The case studies presented in this paper indicate that spanning potential void features with high strength geogrid reinforcement is a practical solution. While geogrid reinforcement is suitable in this application, there are scenarios where other solutions may carry lower risks. Before a design can be finalized, all risks must be assessed and incorporated into the design process. It is always recommended for these types of applications to rely on material extensively tested and certified by independent national authorities or institution.

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