Drainage Reinforced Geocomposite for Marginal and Cohesive Slopes and Walls

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ABSTRACT: *Most practitioners and engineers who currently design walls and slopes still prefer to work with high quality and well graded granular fill as backfill. However, as we move towards a Net Zero world, engineers can make a significant difference and act to help reduce carbon emissions within projects and therefore address climate change. We also know that the infrastructure required to get to Net Zero needs to be resilient and financeable. Thus, considering marginal fills or recycled spoil generated as part of construction activities shall become the new normal for the creation of walls and slopes. Marginal fills typically have high silt and/or clay contents which, when loaded, have the potential to generate excess pore water pressures in the structural backfill. Poor drainage in the structural fill reduces the available strength of the fill, thus reducing the bond between the fill and the geogrid reinforcement. Therefore, to use marginal fill efficiently, adequate drainage must be provided in the reinforced soil structure. By using a geocomposite that combines reinforcement and drainage into geogrid sustainable and environmentally friendly slopes and walls can be designed and constructed. Between 2015 and 2017, an innovative design methodology and approach for the construction of reinforced slopes and walls, with low-permeability fills, was successfully used in the UK. The design and construction experience gained in the UK was presented also at the 11th International Conference on Geosynthetics, in Seoul (Brusa et al, 2018), to confirm the effectiveness of the system and to give engineers confidence in the use of marginal fills in reinforced soil systems. This paper will include the most interesting and latest case histories, including a railway and highway embankments done with such a technique.*

KEYWORDS: sustainability, drainage, geogrid, marginal and fined grained fill, steep slope

SITE LOCATION: **Geo-Database**

INTRODUCTION

The use of marginal fills on site can yield significant cost and environmental savings for a project. In the UK, any material leaving a construction site is considered waste and must be disposed of accordingly. This is expensive, as waste is subject to a £94.15/ton (2020 prices) landfill tax in addition to the cost of excavation and transportation of the soil. Thus, there is significant interest in utilizing material, once considered unsuitable, in site works. Low-permeability marginal fills can be used to construct reinforced slopes and walls, granted that adequate drainage is provided within the reinforced fill (Fukuoka, 1998; Kempton et al., 2000; Naughton et al., 2001). Without adequate drainage, excess pore water pressures would typically build up in the fill during construction. Pore water pressure reduces the internal shearing resistance of the fill, as well as the interface shear strength between the reinforcement and the fill (O'Kelly & Naughton, 2008 and Clancy & Naughton, 2011).

Between 2015 and 2017, several reinforced slopes and walls, up to 17 m in height, were constructed in the UK using excavated marginal fill from construction activities on site. The reinforced soil structures were constructed using a system consisting of a combined reinforcement and drainage geosynthetics product, referred to herein as the draining geogrid and a facing unit, to form the face, thus eliminating the need for a temporary shutter/formwork during construction. A drainage layer to the rear of the reinforced block was generally required (which can be either a geocomposite drain or a free draining granular chimney drain). The function of the back drain is to prevent the flow of water from the retained fill into the reinforced soil block and to remove water from the draining geogrid. A schematic of the system is shown in Figure 1. The draining geogrid was

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specifically designed for usage with low-permeability marginal fills. In presenting the case histories later in this paper, emphasis will be placed on the properties of the fill, the design and construction of the structures, and the quality control procedures implemented on site.

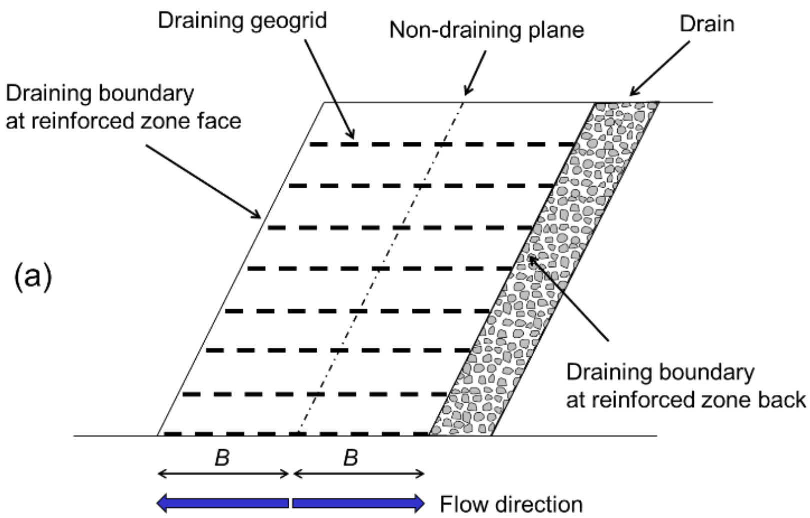

Figure 1. Schematic of the reinforcement and drainage system consisting of draining geogrid and draining boundary to the rear of the reinforced soil block.

PROPERTIES OF THE DRAINING GEOGRID AND DESIGN METHODOLOGY

The draining geogrid comprises both longitudinal and lateral strips. The function of the lateral strips is to maintain the grid geometry, i.e., to ensure that the longitudinal strips remain parallel and equidistant. Each longitudinal strip provides both reinforcement and drainage. A longitudinal strip is entailed of two components: (i) the reinforcement component, consisting of high-tenacity polyester yarns encased in a durable polyethylene coating that protects the yarns; and (ii) the drainage component, which consists of a channel in the profiled polyethylene coating. The drainage channel is bridged by a thermally bonded nonwoven geotextile filter, which allows water to flow from the fill to the channel while preventing soil particles from intruding into the channel. The draining geocomposite is available in a range of short-term characteristic strengths between 50 kN/m and 200 kN/m, as shown in Table 1. Due to the lower lift heights employed with site-won fill, the lower short-term characteristic strength (50 kN/m) is typically employed in design. The thermally bonded geotextile filter has the characteristics presented in Table 2.

Giroud et al. (2014) presented a methodology for designing reinforced slopes and walls using draining geogrids. The design methodology utilized the consolidation properties (coefficient of volume compressibility, mv, and the coefficient of consolidation, Cv) of the marginal fill for determining the vertical spacing of the draining geogrid layers and the rate at which the slope can be constructed. The design method presented by Giroud et al. (2014) includes two main elements:

1. Determination of the required hydraulic transmissivity of the draining geogrid and the required time to ensure rapid dissipation of pore pressure.

2. Determination of the stability and settlement of the reinforced wall or slope. This part is the classical design of multilayer reinforced structures (walls or slopes).

The reinforced zone is the area of fill that contains the reinforcement layers. A drain located at the back of the reinforced zone, seen in Figure 1, may be used for one or both of the following reasons: (i) to prevent ground water from flowing into the reinforced zone; and/or (ii) to halve the drainage length in the draining geogrid. If there is a drain at the back of the reinforced zone, the structure will have two draining boundaries, and a non-draining plane located mid-way between the two boundaries. If there is no drain at the back face of the reinforced zone, the structure has one draining boundary and the back of the reinforced zone is a non-draining boundary.

Table 1. Properties of the drainage geogrid.

Parameter	Grade of draining geogrid		
	50	80	200
Characteristic short-term strength: longitudinal direction (kN/m)	50	80	200
Nominal strain at characteristic short-term strength (%)	9	9	9
Grid size, warp/weft (mm)	75 x 450	75 x 450	75 x 450
Aperture size, warp/weft (mm)	51 x 426	51 x 426	42 x 426
Proportion of the plane sliding area that is solid	0.36	0.36	0.47

Table 2. Properties of the thermally bonded geotextile filter.

Parameter	Value, all grades
Pore size of geotextile filter, O_{90} (µm)	100
Permeability normal to the plane (l/m^2.s)	90
In plane flow at a normal pressure of 100 kPa	
Hydraulic gradient = 1.0	3.8
Hydraulic gradient = 0.5	1.9
Hydraulic gradient = 0.1	0.9

In analyzing the Giroud et al. (2014) design methodology, Naughton et al. (2015) showed that constructing one to two layers of the slope per day was a realistic target. This is not dissimilar to the rate used for steep reinforced slopes constructed from well-draining backfill material. Naughton et al. (2015) also showed that, with low-permeability fills, consideration needs to be given to the maximum vertical spacing of the draining geogrid. For practical purposes, they suggested that an upper limit of 0.6 m should be employed, primarily to guard against horizontal deformations of the slope. Their analysis also showed that a vertical reinforcement spacing of 0.4 - 0.5 m optimized the time for dissipation of pore pressures while, at the same time, requiring realistic and achievable transmissivities in the draining geogrid. While intuitively smaller vertical spacings may appear prudent, the required transmissivity in the draining geogrid is significant and it is not realistic to manufacture a draining geogrid with these large transmissivities. Furthermore, where global stability of the reinforced soil structure must be improved, it is preferable to increase the length of the reinforcement than to decrease the vertical spacing. Naughton et al. (2015) further showed that reinforced soil structures can be constructed from most low permeability fills once careful consideration is given to the drainage properties of the fill and the geometry of the reinforcement elements, particularly the vertical spacing of the draining geogrid.

The interaction between a geogrid and the surrounding soil is required when analyzing the stability of a slope. The coefficient of skin friction, α, can be defined as the ratio of the skin friction between the geogrid, δ, and the fill and the angle of friction of the fill, ϕ, through Equation 1.

$$\alpha = \frac{tan\delta}{tan\phi} \qquad (1)$$

Jewel (1996) suggested that the coefficient of skin friction for geogrids in fine fills could be as low as 0.4. However, O'Kelly & Naughton (2008) and Clancy & Naughton (2011) showed experimentally that the interaction coefficient for a draining geogrid is similar to that for a granular fill. These studies found that the excess pore water pressure in the immediate vicinity of the geogrid – fill interface dissipated rapidly, allowing the full shear resistance to be mobilized along the interface.

Data reported by O'Kelly & Naughton (2008) and Clancy & Naughton (2011) indicated that an interaction coefficient of 0.6 can be conservatively used in the design of reinforced soil slopes with marginal fill and a draining geogrid. This was the value used in the design of the case histories presented in this paper.

The draining geogrid dissipates the excess pore pressure generated during placement and compaction of the fill to 90% of its initial value. The remaining 10% of the theoretical excess pore water pressure was incorporated into the stability calculation using a pore pressure ratio, $r_u = 0.1$. The pore pressure ratio, r_u, was defined as the ratio of the pore water pressure in the fill relative to the applied total stress.

Partial reduction factors were applied to the characteristic short-term design strength to determine the long-term strength in accordance with ISO TR20432 (2007). Reduction factors were also applied to the measured transmissivity of the draining geogrid as recommended by Giroud et al. (2014), shown in Table 3.

Table 3. Reduction factors on strength and short-term hydraulic transmissivity of the draining geogrid.

Reduction factor	Value
Creep based on a design life of 60 years and a design temperature of 20^0 C	1.37
Installation damage based on silty sand fill (100% passing 20 mm and 50% passing 63 μm)	1.02 – 1.04
Weathering	1.0
Chemical/environmental effects based on a design temperature of 20^0 C and $4 \leq pH \leq 9$ for the fill	1.02
Factor of safety for extrapolation of data	1.02
Intrusion of the filter geotextile into the drainage channel	1.0
Compressive creep of the draining geogrid	1.0
Particulate clogging of the drainage channel	1.0
Chemical clogging of the drainage channel	1.2
Biological clogging of the drainage channel	1.0

TESTING OF MARGINAL FILL

The strength and consolidation properties of site won fill can vary greatly. It is necessary to conduct additional testing beyond that required for granular fill. It is recommended that strength testing is conducted in the large shear box and that the consolidation parameters are determined either from oedometer or Rowe/hydraulic cell testing. In both cases, the sample should be prepared to a target a dry density of 92% ± 2%. The maximum particle size should be limited to 20 mm in the large 300 mm x 300 mm shear box and 4 mm for a 20 mm high sample in the oedometer or Rowe/hydraulic cell testing. The applied pressure should be representative of the stress encountered in the slope.

Recommended site compaction during construction is to achieve at least 90% of maximum dry density. The compaction requirements are similar to those employed for granular fill used in the construction of reinforced soil slopes (SHW, 2009). The compaction curve for the fill and all proxies for quality control testing, typically moisture condition value and nuclear density probe, should be determined in the laboratory before construction commences.

DESCRIPTION OF REINFORCED SOIL STRUCTURES CONSTRUCTED IN THE UK USING MARGINAL FILL

Between 2015 and 2016, several reinforced soil structures were constructed in the UK using the draining geogrid and marginal fills. The following sections give an overview of these projects, together with construction photographs. The reinforced soil structures were constructed using a system consisting of facing units, draining geogrid and geocomposite or drainage stone back drain, shown in Figure 1. The reinforced soil slopes were constructed in a similar manner to slopes constructed from granular fill. The primary difference was that the rate at which the slope could be constructed was controlled to ensure the dissipation of pore water pressures before constructing the next sequence of the slope. The primary focus in presenting the case studies is on the properties of the site won fill and the design and construction of the reinforced soil structures.

Reinforced structures using the draining geogrid and marginal fills were designed using BS 8006-1 (2010) for internal stability, BS EN 1997 (2013) for global stability, and the method presented by Giroud et al. (2014) for the dissipation of excess pore pressures in the marginal fill. The time to dissipate excess pore water pressures was generally 24 – 48 hours; however, on some projects, longer dissipation times were required as dictated by the consolidation parameters of the fill and the earthworks operations on site. In all cases, the excess pore water pressure was dissipated to 90% of its initial value after each construction sequence.

North Gawber Colliery, Barnsley, Yorkshire, 2015

A reinforced steep slope of approximately 300 m in length that varied in height from 3.6 m to 8.4 m was built along the site boundary to create platforms for future development as part of the North Gawber Colliery regeneration. The face of the slopes was at 70^0 to the horizontal. The reinforced soil structures were constructed from site won colliery spoil, assumed to be Class 7D material (SHW, 2009) and having the properties given in Table 4. The fill can be described as a granular to cohesive fill (BS 6031, 2009). The structures were built in 0.4 m lifts at a rate of 2 lifts (0.8 m) per day and with a rest period of one day between the construction of subsequent layers. The ratio of reinforcement length to slope height varied between 1.11 for the lowest height slope to 1.50 for the higher slopes, as shown in Table 5.

Table 4. Properties of site won fill, North Gawber Colliery, Barnsley.

Plasticity Index (%)	Optimum moisture content (%)	Maximum dry density (Mg/m3)	% passing 63 µm sieve	Unit weight (kN/m3)	Angle of friction (0)	m_v (m^2/MN)	C_v (m^2/year)
32	13 – 35	1.33 – 1.74	4 – 82	16.4 – 20.1	25	0.2	8

Table 5. Reinforcement length and slope height at Island Road, Reading.

Slope height (m)	Reinforcement length (m)	No. or lifts	Ratio reinforcement length to slope height
3.6	4.0	9	1.11
4.2	5.0	10	1.19
6.0	9.0	15	1.50
8.4	12.0	21	1.46

Island Road, Reading, Berkshire, 2016

As part of a new commercial retail park development over an old waste pit, reinforced soil structures were required to form a swale for attenuating and discharging storm water during storm events. The swale was created with reinforced soil structures that varied in height from 1.8 m to 3.0 m. The backfill consisted of competent site won fill material complying with a mix of Class 2A and Class 2C material (SHW, 2009). The fill had a maximum particle size of 125 mm, between 15 – 80% passing the 63 µm sieve, and was classified as a cohesive fill (BS 6031, 2009). The properties of the fill are presented in Table 6. The structures were constructed in 0.6 m lifts. A characteristic Cv of 14.5 – 15.0 m/year was used in design. The reinforcement length and slope height are presented in Table 7. Overall, the ratio of reinforcement length to slope height varied between 0.4 for the low height slopes to 1.0 for the higher slope heights. A traffic loading of 10 kPa was considered along the top of the bund. The structures were constructed at a rate of two layers / day (1.2 m) with a one-day rest period between construction phases. The contractor scheduled the lift height and dissipation time into the construction program, resulting in no delays to construction activities.

Table 6. Properties of site won fill, Island Road, Reading.

Liquid limit (%)	Plasticity Index (%)	Unit weight (kN/m3)	% passing 63 µm sieve	Apparent cohesion (kPa)	Angle of friction (0)	m_v (m^2/MN)	C_v (m^2/year)
39 – 53	19 – 32	18.8	15 – 80	0	25	0.2 – 0.4	2.5 – 16

Table 7. Reinforcement length and slope height at Island Road, Reading.

Slope height (m)	Reinforcement length (m)	No. or lifts	Ratio reinforcement length to slope height
1.8	3.5	3	0.51
1.8	2.5	3	0.72
2.0	3.0	3	0.67
2.4	3.5	4	0.69
3.0	3.0	5	1.00

Millbrook Park, Barnet, London, 2016

Millbrook Park is a new housing development located in north London. The reinforced soil structure, which varied in height between 3.5 m and 4.7 m (retained height), supported a road and was constructed with site won fill consisting of London clay having the properties presented in Tables 8 and 9. The site won fill was described as brown silty gravelly clay of high to very high plasticity (BS 5930, 2020). The particle size distribution had a maximum size of 75 mm, with 42 – 56% passing the 63 µm sieve, and was classified as a cohesive fill (BS 6031, 2009). A traditional geogrid wrap-around construction technique was used to form the slope face on this project. The construction sequence was agreed with the contractor and consisted of constructing two layers (0.8 m high) per day, with a rest day before the next construction phase. The ratio of base length of reinforcement to height of slope was 0.8 and 1.25 for the 4.0 m and 5.2 m high slopes respectively.

Table 8. Classification properties of site won fill, Millbrook Park, Barnet.

Liquid limit (%)	Plasticity Index (%)	Unit weight (kN/m3)	pH	% passing 63µm sieve	m_v (m^2/MN)	C_v (m^2/year)
68 – 78	40 – 58	18.6 – 19.4	7.9 – 10.1	42 – 56	0.2*	7*

* assumed values

Table 9. Compaction and strength properties of site won fill, Millbrook Park, Barnet.

Optimum moisture content (%)	Maximum dry density (Mg/m3)	Apparent cohesion (kPa)	Angle of friction (^{0})
20.5 – 22.4	1.63 – 1.69	0	26

Palmerston Park, Tiverton, Devon, 2016

Permanent reinforced soil retaining structures were required to support the access road and to retain a bank-cutting created for a social housing development on a sloping site in Tiverton, Devon. Several structures, varying in height from 5.3 m to 17 m above existing/finished ground level and with 70^0 face angles from the horizontal were constructed as part of the scheme. The reinforced soil structures were constructed from site won fill consisting of clayey sandy gravel of low to intermediate plasticity (BS 5930, 2020). The site won fill had between 9 – 44% passing the 63 µm sieve, and was classified as a granular to cohesive fill (BS 6031, 2009). The properties of the fill and reinforcement length are presented in Tables 10 and 11 respectively. Images of the 16 m high slope both during construction and approximately 10 months later are shown in Figures 2 and 3 respectively. The higher strength draining geogrid was used in the lower third – half of the higher structures. The design rate of construction was to construct four layers (1.52 m) per day. The ratio of reinforcement length to slope height varied from 0.57 – 0.75.

Table 10. Properties of site won fill, Palmerston Park, Tiverton.

Liquid limit (%)	Plasticity Index (%)	Unit weight (kN/m3)	% passing 63 µm sieve	Apparent cohesion (kPa)	Angle of friction (^{0})	m_v (m^2/MN)	C_v (m^2/year)
33 – 38	10 – 15	22	9 – 44	0	28	0.1	57 – 73

Table 11. Reinforcement length and slope height at Palmerston Park, Tiverton.

Slope height (m)	Reinforcement length (m)	No. or lifts	Ratio reinforcement length to slope height
16.0	10	42	0.63
13.7	8	36	0.59
10.6	6	27	0.57
8.8	5	23	0.57
8.4	5	22	0.60
5.3	4	15	0.75

Figure 2. 17 m high structure constructed from draining geogrid and site won fill, Palmerston Park, Devon, October 2016.

Figure 3. Vegetated structure constructed from draining geogrid and site won fill, Palmerston Park, Devon, Summer 2017.

M40 Noise Bund, Banbury, Oxfordshire, 2016 and 2017

A noise bund, varying in height between 2.4 m and 6.4 m above the existing ground level, was constructed along the M40 motorway in Oxfordshire. The bund, which was 1,300 m in length, shielded a new housing development from the visual and noise generated by the motorway. The bund consisted of a 70^0 reinforced soil steep slope on the motorway side and an unreinforced 1V:3H slope on the housing side. The bund was constructed entirely from site won fill consisting of Dyrham Formation and Charmouth Mudstone (classified as Class 2A and 2B (SHW, 2009)) with the properties listed in Table 12. The fill had a maximum particle size of 125 mm, had between 15 – 80% passing the 63 µm sieve, and was classified as an intermediate to cohesive fill (BS 6031, 2009). Figure 4 shows the placement and compaction of the fill during construction. A traffic surcharge along the crest of the slope of 10 kPa was used in design. The rate of construction varied with slope height and consisted of 2 lifts (0.8 m) /day for the first 1.6 m, then 2 lifts (0.8 m)/ day with 1 rest day for slope heights between 1.6 m and 3.6 m and then 2 lifts (0.8 m) / day with 2 rest days for heights greater than 3.6 m. The reinforcement lengths for the different slope heights are presented in Table 13.

The M40 Noise Bund was shortlisted in the sustainability category at the Ground Engineering Awards 2017.

Table 12. Properties of site won fill, Banbury Noise Bund, Oxfordshire.

Unit weight (kN/m3)	Optimum moisture content (%)	Maximum dry density (Mg/m3)	% passing 63 μm sieve	Apparent cohesion (kPa)	Angle of friction (0)	m_v (m^2/MN)	C_v (m^2/year)
19.5	22.3	1.62	15 – 80	0	24	0.3 – 0.5	5 – 40

Table 13. Reinforcement length and slope height at Banbury Noise Bund, Oxfordshire.

Slope height (m)	Reinforcement length (m)	No. or lifts	Ratio reinforcement length to slope height
2.4	2.0	5	0.83
3.6	4.0	9	1.11
4.8	4.0	12	0.83
5.6	4.5	14	0.81
6.4	5.5	16	0.86


(a)


(b)

Figure 4. (a) Placing and (b) compacting site won fill material over the draining geogrid. The drainage boundary at the back of the reinforced soil block is also visible and consists of a geocomposite drainage element.

Jenkins Lane, Barking, London, 2017

Reinforced soil structures, varying in height between 6 m and 7.6 m, were required as part of a commercial development at Jenkins Lane in London. The site-won fill consisted of made ground which was generally described as a black or brown and grey clayey, silty, sandy fine to coarse angular to sub-rounded brick gravel with occasional glass, wood, ash, clinker, ceramics, plastic, waste metal, rebar, organic material, and occasional concrete and brick cobbles. Localized areas of finer grained cohesive Made Ground were also encountered. Given the varied nature of the site won fill, a decision was made to only utilize fill that met the requirements of Class 2 fill in the Specification of Highway Works (SHW, 2009), thus removing all non-soil waste from the fill. Tables 14 and 15 present the properties of the site won fill. The fill had a maximum particle size of 125 mm, between 15 – 80% passing the 63 μm sieve, and was classified as an intermediate to cohesive fill (BS 6031, 2009). The slope angle was between 64 – 65^0 from the horizontal and was constructed at a rate of 2 lifts/day. The lift height was 0.38 m and the reinforcement length at each slope height is presented in Table 16.

Table 14. Classification properties of site won fill, Jenkins Lane, Barking.

Liquid limit (%)	Plasticity Index (%)	Plasticity	Unit weight (kN/m3)	pH	% passing 63 μm sieve	m_v (m^2/MN)	C_v (m^2/year)
59	27	CL	18.6	6.9 – 7.6	0 – 87	0.5*	12*

* assumed values

Table 15. Compaction and strength properties of site won fill at Jenkins Lane, Barking.

Optimum moisture content (%)	Maximum dry density (Mg/m3)	Apparent cohesion (kPa)	Angle of friction (^{0})
15 – 20	1.46 – 1.77	0	24

Table 16. *Reinforcement length and slope height at Jenkins Lane, Barking.*

Slope height (m)	Reinforcement length (m)	No. or lifts	Ratio reinforcement length to slope height
6.08	7.3	16	1.20
7.60	9.1	20	1.20

North Bexhill Access Road, Phase 1 and Phase 2, East Sussex, 2016 & 2017

The North Bexhill Access Road project was a 2.4 km single carriageway road designed to accommodate future employment land to the North of Bexhill, East Sussex in the southeast of England. Reinforced soil structures were constructed from site won Tunbridge Wells Sand / Ashdown formation, which was specified as Class 2A (SHW, 2009) material of intermediate plasticity (BS 5930, 2020) along the extremities of the road embankment. The fill had a maximum particle size of 125 mm, between 15 – 80% passing the 63 µm sieve, and was classified as an intermediate to cohesive fill (BS 6031, 2009). The properties of the site won fill are presented in Tables 17 and 18. The embankment was partially over peat and in those locations, was supported on Controlled Modulus Columns; most of the embankment was over soft clay and directly placed on a reinforced foundation consisting of unidirectional high tenacity polyester based geosynthetic basal reinforcement, as shown in Figure 5. The slope was constructed in 0.38 m lifts with a draining geogrid of short-term characteristic strength of 50 kN/m at each layer. The rate of construction was 2 layers per day (0.76 m) with no rest day. The reinforcement length used in each structure is given in Table 19.

The North Bexhill Access Road scheme received the Institution of Civil Engineers Southeast England Engineering Excellence Award in 2017.

Table 17. Classification properties of site won fill, North Bexhill Access Road, East Sussex.

Liquid limit (%)	Plasticity Index (%)	Plasticity	% passing 63 µm sieve	pH	m_v (m^2/MN)	C_v (m^2/year)
44	23	Intermediate plasticity	15 – 80	5.2	0.5*	12*

* assumed values

Table 18. Compaction and strength properties of site won fill at North Bexhill Access Road, East Sussex.

Optimum moisture content (%)	Maximum dry density (Mg/m3)	Apparent cohesion (kPa)	Angle of friction (^{0})
18	1.74	0	25.5

Table 19. Reinforcement length and slope height at North Bexhill Access Road, East Sussex.

Slope height (m)	Reinforcement length (m)	No. or lifts	Ratio reinforcement length to slope height
3.80	6	10	1.58
4.56	6	12	1.32
5.32	6	14	1.13
5.54	7	14	1.26

Queensway Gateway Link, Hastings, East Sussex, 2017

The Queensway Gateway project consisted of a single lane road embankment, 300 m in length and 9 m high, connecting the A2690 and the A21 with an underpass tunnel pedestrian walkway and an attenuation pond. The site was underlain by the Ashdown Formation (sandstone, siltstone, and mudstone). The Wadhurst Clay Formation overlies the Ashdown Formation. Alluvial deposits were also present. The draining geogrid was used as reinforcement for the embankment shallow slope with 1V:2H inclination. The backfill was described as stiff slightly sandy clayey silt and had the properties listed in Table 20. Figure 5 shows the construction of a shallow slope at Queenway Gateway. Construction occurred during the dry summer months and a line of damp soil, resulting from dissipation of pore water pressures in the body of the slope, was observed where each layer of the draining geogrid intersected the face of the slope, as shown in Figure 5.

Table 20. Properties of site won fill, Queenway Gateway, Hastings.

Liquid limit (%)	Plasticity Index (%)	Plasticity	Unit weight (kN/m^3)	Apparent cohesion (kPa)	Angle of friction (^{0})	m_v (m^2/MN)	C_v (m^2/year)
42	20	Low	19	0	26	0.5*	12*

* assumed values

Figure 5. Shallow slope during construction at the Queensway Gateway Link project where the drainage geogrids are placed in layers at equidistance vertical centers; the facing element of the slope will be created using hydro seeding over an erosion geosynthetic mat.

East Midlands Gateway (EMG), Nottingham, 2018

In 2016, Roxhill Kegworth Ltd proposed the construction of the East Midlands Gateway Strategic Rail Freight Interchange. The development site covers approximately 374 ha in total. The development comprises a Strategic Rail Freight Interchange (SRFI), incorporating several large warehouses and associated infrastructure, a rail freight container terminal, and a landscape bund to the north, providing visual screening for the villages of Lockington and Hemington. The embankment to support the private railway, linking the EMG Rail Terminal to the national rail network, was built using the reinforced soil slope technique. The slope angle varies between 1:1 to 70^0 and has a height that varies between 2.5 m and 7.5 m.

The rail embankment, which was 980 m in length, was constructed during 2018 using a combination of site won fine grained material, firm to stiff red -brown clay, meeting SHW (2009) Class 7C for the reinforced soil block, Class 7A (selected fill above, below, and behind the reinforced soil block) and Class 2 (general fill on the unreinforced embankment extremity). Class 2 and 7A have 15 – 100% passing 63 µm sieve, while Class 7C has 15 – 45% passing. All three classes have a maximum particle size of 100 mm. The properties of the fill are presented in Table 21. The slopes were constructed with lift heights of 0.55 m at a rate of 1.1 m (2 lifts) every 4 days. The ratio of reinforcement length to slope height varied from 1.65 to 2.0, Table 22. Figure 6 presents shows the different stages of embankment construction.

Table 21. Classification properties of site won fill (Class 7C) used in reinforced block, EMG, Nottingham.

% passing 63 µm sieve	Unit weight (kN/m3)	Apparent cohesion (kPa)	Angle of friction (^{0})	m_v (m^2/MN)	C_v (m^2/year)
15 – 45	20.5	2	25	0.117	25

Table 22. Reinforcement length and slope height at East Midlands Gateway, Nottingham.

Slope height (m)	Reinforcement length (m)	No. or lifts	Ratio reinforcement length to slope height
4.0	8	8	2.00
6.0	10	11	1.67
7.0	12	13	1.71
7.9	13	15	1.65

(a)

(b)

(c)

Figure 6. (a) Installation of the draining geogrid, (b) finished embankment before laying of track, and (c) completed embankment with track installed.

Compaction and Quality Control Testing

Marginal fills have higher fines contents, higher optimum moisture contents, and lower maximum dry densities than traditional predominately granular materials. This can make the compaction of marginal fills more challenging. BS 6031 (2009) states that the engineering properties of fill will differ depending on their percentage fines (passing 63 μm sieve). Fill with greater than 35% fines behave as a fine-grained soil, while fills with less than 15% behave as course-grained soil. Intermediate fills between 15% and 35% fines content, when used in shallow slopes, are generally considered to behave as course-grained fills. However, when intermediate fills are used with a draining geogrid in steep slopes (where the face is between 45^0 to 70^0 from the horizontal), it is assumed that the fill will behave as a fine-grained soil.

Projects in the UK mainly utilize a well-graded granular wet, dry, or stoney cohesive fill, which are classified as Class 2A, 2B, and 2C (SHW, 2009) respectively for embankment construction, excluding reinforced soil. Class 2A and 2B have a grading with 100% passing 125 mm, 80 – 100% passing 2 mm and 15 – 100% passing 63 μm and 2C fills has 100% passing 125 mm, 15 – 80% passing both 2 mm and 63 μm. These fills are typically intermediate to cohesive fill (BS 6031, 2009). The compaction of Classes 2A, 2B, and 2C uses a method specification where the layer thickness and number of passes of a particular type of compaction plant are specified (SHW, 2009). The method of compaction is such that the layer thickness and compaction plant vary from project to project. The construction of reinforced soil slopes with granular backfill in the UK utilizes Class 6I/6J (SHW, 2009), which is a granular fill (BS 6031, 2009), having 100% passing 125 mm, 85 – 100% passing 75 mm, 15 – 100 passing 2 mm, and 0 – 15% passing 63 μm. Compaction of 6I/6J is also by method statement. However, contractors have a strong preference to use Classes 2A, 2B, and 2C for reinforced soil structures along the extremities of embankments, as it minimizes the number of different materials on site and the need to delineate between different fill type areas across the embankment section.

Quality control testing of the compacted marginal fill is critically important when using marginal fill, as the dry density achieved is very dependent on the moisture content. Given the varied nature of the fill and the method approach used during compaction, it is important to conduct compliance testing to ensure that the measured dry density on site is greater than 90% of the maximum dry density for the fill. Compaction quality control testing in fine grained fills is problematic, as most tests require a determination of the fill moisture content, which can delay reporting of results by 1 – 2 days. Experience in the UK indicates that nuclear density testing is a suitable proxy to moisture content testing, and this is used widely on projects constructed from marginal fill. The number of tests is taken as 1 – 2 tests per 1,000 m^3 of material up to a maximum of 5 per day. This requirement is in line with recommendations by Trenter and Charles (1996) and HA 44 (1995).

DISCUSSION OF CASE HISTORIES

Several case histories were presented which demonstrated the effectiveness of using a draining geogrid and marginal fill in the construction of steep slopes.

The engineering characteristics of the marginal fills was found to vary significantly between projects. Angles of friction for the marginal fills were in the range of 24^0 – 26^0, lower than the 30^0 conservatively used with granular fills. The consolidation characteristic of the fills also varied, but the values used correspond to soils with a permeability of the order 10^{-7} – 10^{-9} m/s, corresponding to very poor draining materials. The optimum moisture content of the fills was in the range of 15 – 23%. The fills were generally clay of low to intermediate plasticity. Compaction was performed using a method statement (SHW, 2009) designed to achieve at least 90% of maximum dry density. The use of a method statement for compaction does not negate the need for quality control testing. This is particularly important when using marginal fill in the reinforced zone.

BS 8002 (2015) provides a relationship for UK soil between the plasticity index, I_p, and the constant angle of friction, ϕ_{cv}, shown in Equation 2. Figure 7 shows a reasonably good relationship between the plasticity index and the constant value angle of friction for all the projects presented in this paper. Estimated angles of friction determined from Equation 2 are sufficient for a preliminary design assessment for UK fills. However, it is recommended that the angle of friction is always confirmed by appropriate laboratory testing using the compaction conditions expected on site.

$$\phi_{cv} = 42^0 - 12.5 log_{10} I_p \qquad (2)$$

The length of the draining geogrid at the base of the structures was found to vary between 0.6 – 2.0 times the maximum retained height of the structure, shown in Figure 8, with the majority requiring reinforcement length between 0.8 – 2.0 times the maximum retained height. These reinforcement lengths are higher than those typically required for steep slopes constructed with good quality imported granular fill, which is generally in the range of 0.6 – 0.7 times the retained height. The longer reinforcement lengths are required due to the lower angle of friction for the marginal fill and, thus, the lower interaction achieved between a marginal fill and the reinforcement than that experienced by good quality granular fill.

Palmerston Park had significantly shorter lengths of reinforcement (0.6 – 0.75 times the maximum retained height) than other projects. The fill at Palmerston Park was, overall, of better quality than that used at the other projects, with an angle of friction of 28^0, a fines content (passing 63 µm) of 9 – 44%, and a description of clayey sandy gravel.

Lower short-term strengths are typically used with marginal fills, as the design process is not trying to maximize the geogrid spacing for stability considerations, but rather to optimize the spacing for the dissipation of excess pore water pressures. Draining geogrids typically have lower short-term strength and are placed at smaller vertical spacings as compared to reinforced slopes constructed from traditional geogrids and granular backfill. This is an important design distinction.

In the case studies presented, the excess pore water pressures were designed to be dissipated to 90% of their initial value within 24 – 48 hours, as shown in Table 23, depending on the quality of the marginal fill. This facilitated the construction of between 1 and 4 layers of reinforcement every 1 – 2 days. This rate of construction did not affect the construction time for any of the structures presented in this paper. No movement or deformation of the structures was observed either during or post-construction. This was attributed to allowing 90% dissipation of pore water pressures during each construction sequence (layers constructed in one go). With the pore water pressure dissipated to 90% of its initial value, a pore water ratio, $r_u = 0.1$, was used in stability calculations. The rate of construction is an important design consideration when using marginal fills and can be adjusted to optimize earthworks on site. With a drainage plane at the top and bottom of each layer, theoretically any number of layers can be constructed in a day, as the dissipation time for each layer is similar. However, for practical considerations, the number of lifts that can be constructed is limited based on site constraints (availability of fill and access for construction equipment).

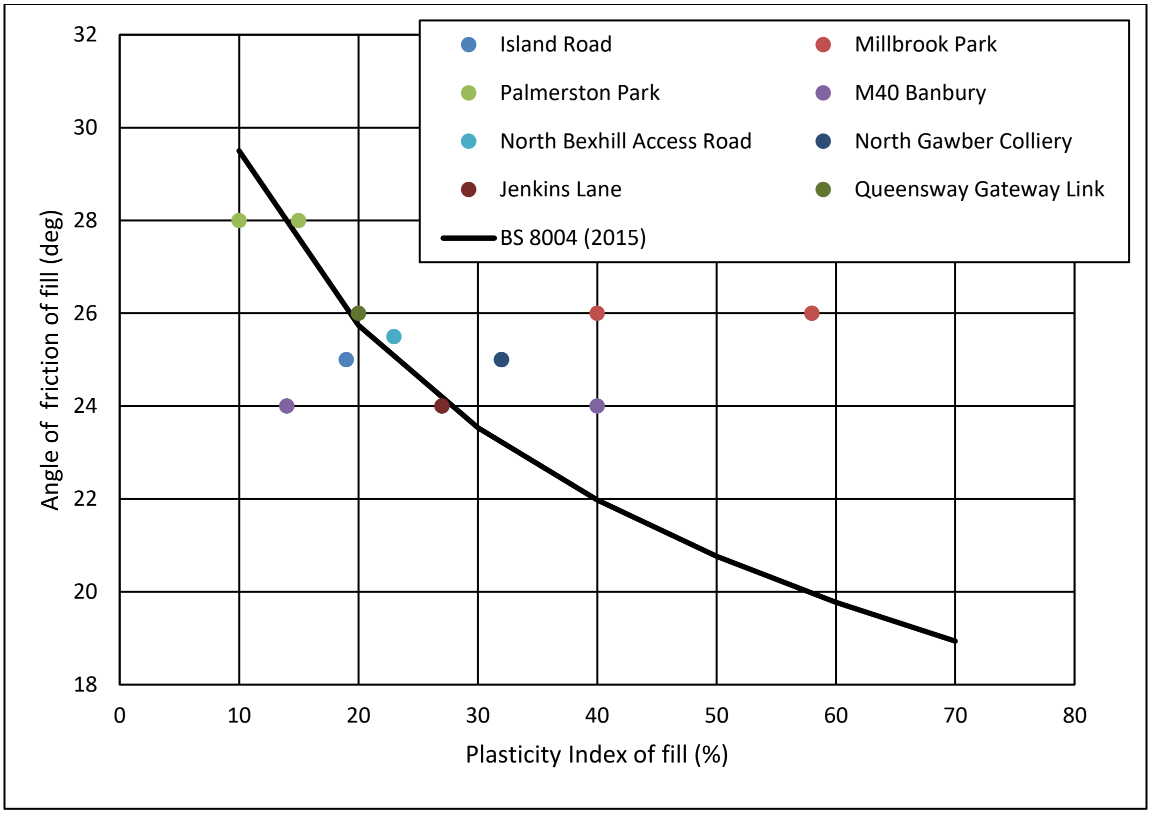

Figure 7. Relationship between angle of friction and plascity index for the project reported in this paper.

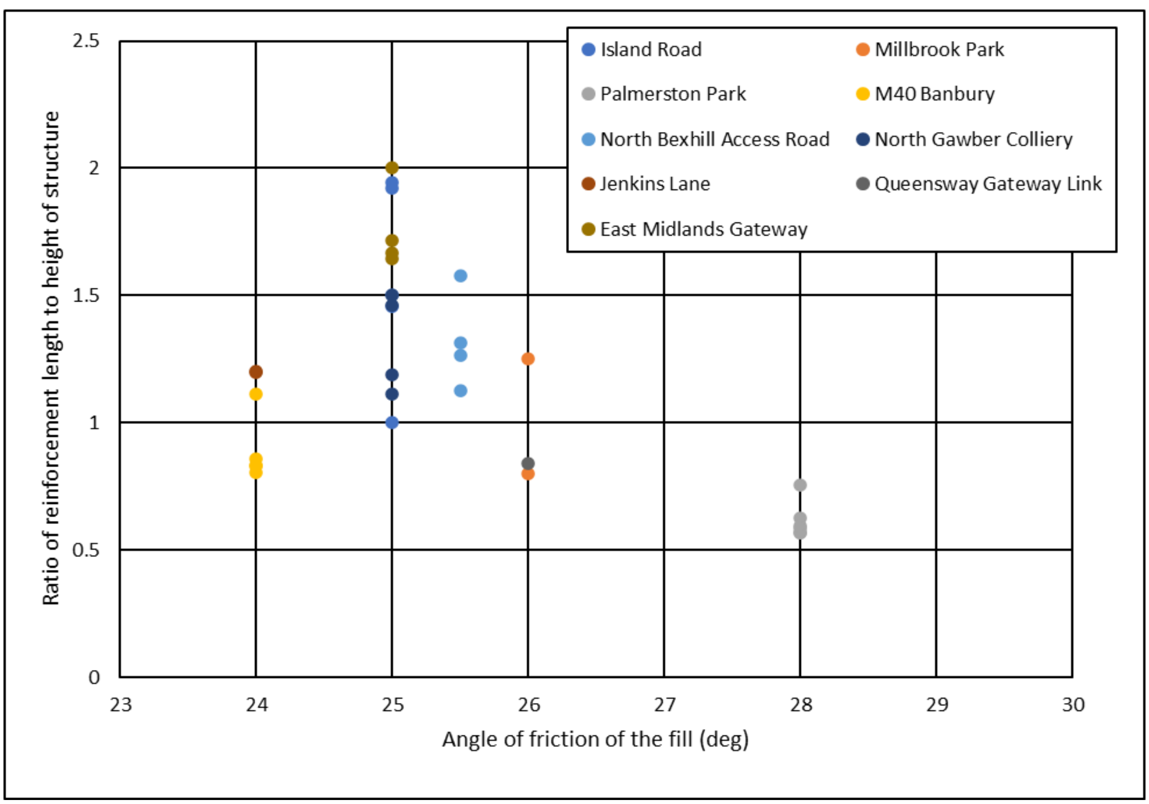

Figure 8. Ratio of reinforcement length to height of slope.

Laboratory measurements, depicted in Table 23, showed that the coefficient of consolidation can vary greatly, is typically in the range of 2.5 – 73 m^2/year, and the coefficient of volume compressibility can vary between 0.1 – 0.5 m^2/MN. In several projects, assumed values of the consolidation parameters were used. The assumed values were selected based on the range of values observed at other projects and the analysis presented by Naughton et al. (2015).

The optimum moisture content of many of the marginal fills presented in this paper is close to 20%. Compaction at moisture contents above this value can be problematic. In the UK, the optimum time for construction and earthworks in general is late Spring to early Autumn. Only minor delays were experienced for some of the case histories presented in this paper due to adverse weather conditions.

Table 23. Summary of consolidation properties and rate of construction for all case histories.

Project Name	% passing 63 µm sieve	mv (m^2/MN)	Cv (m^2/year)	Lift height (m)	Rate of construction
North Gawber Colliery	13 – 35	0.2	8	0.40	2 lifts per day with 1 day rest period
Island Road	15 – 80	0.2 – 0.4	2.5 – 16	0.60	2 lifts per day with 1 day rest period
Millbrook Park	42 – 56	0.2*	7*	0.40	2 lifts per day with 1 day rest period
Palmerston Park	9 – 44	0.1	53 – 73	0.38	4 lifts per day
M40 Noise Bund	15 – 80	0.3 – 0.5	5 – 40	0.40	2 lifts per day with 2 day rest period
Jenkins Lane	0 – 87	0.5*	12*	0.38	2 lifts per day
North Bexhill Access Road	15 – 80	0.5*	12*	0.38	2 lifts per day
East Midlands Gateway	15 - 45	0.117	25	0.55	2 lifts every 4 days

CONCLUSIONS

Significant monetary and environmental savings can be achieved for construction projects by utilizing excavated site won marginal fills in earthworks, especially as the structural fill in reinforced soil structures. Concerns around the generation of excess pore pressures in marginal fills during construction are often cited as a reason not to use these soils in this application. The case histories presented in this paper have shown that a wide range of marginal fills, ranging from colliery spoil to made ground to high plasticity clays, have been successfully used in constructing reinforced soil structures up to 16 m in height when combined with a draining geogrid.

Using a draining geogrid, where both drainage and reinforcement functions are combined into a single product, is ideally suited to these applications. Having a drainage element at each reinforcement layer immediately improves the soil – geosynthetic interaction and allows the marginal fill to be considered drained in terms of the selection of strength characteristic for use in design. The case studies presented in this paper have shown:

1. A wide range of marginal fills, with varying engineering properties, can be used as structural back-fill in reinforced soil structures.

2. The engineering characteristics of marginal fills can vary significantly and are inferior to those of expensive imported granular fills. This does not, however, negate their use as structural backfill.

3. An adequate quality control program must be put in place during construction. In the UK, most marginal fills are Class 2 fills (SHW, 2009). Marginal fills have high optimum moisture contents and are generally compacted slightly wet of optimum. Compaction should achieve at least 90% of maximum dry density. Onsite quality control must ensure that the design dry density is achieved.

4. The dissipation of excess pore water pressure is time dependent. However, by careful selection of the vertical spacing of the draining geogrid, the rate of construction of the reinforced soil structure can be adjusted to meet the earthworks program on site, ensuring that a structure with marginal fill can be constructed at a similar rate to that using granular fill.

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