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Queen Anne's School

Queen Anne's School, Caversham – Playing
Field

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1 Introduction

1.1 Report Objectives

Queen Anne's School instructed ByrneLooby (BL) to undertake a geotechnical assessment of the site ground conditions and provide design options for potential remedial measures required considering the nature of the proposed development of the playing field.

The purpose of this Ground Investigation Report (GIR) is to summarise the results of ground investigation work undertaken at the site, detailing the factual information obtained and providing a geotechnical evaluation of this information. This report has been prepared in general accordance with the requirements of BS EN 1997-1:2004+A1:2013 *Eurocode 7. Geotechnical design. General rules*.

Following the L741.R01 *Phase I (Desk Study) Site Investigation Report* produced by BL, the aim of the proposed Phase II Intrusive Investigation consisted of:

1. Confirming the general ground and groundwater conditions across site.
2. Confirming the thickness of the superficial deposits overlying the chalk and any presence of dissolution features.
3. Confirming the character of the soils/rock making up the ground profile.

Provisional remedial solutions were proposed within L741.R01 which consisted of bulk-infilling sub-surface voids with compaction grout, constructing a reinforced concrete slab to bridge over any significant voids and providing alternative drainage systems to control groundwater infiltration. The information obtained within the intrusive ground investigation shall be used to assist in the development in the most suitable remedial solution.

Site information obtained from the desk study which primarily consists of the site details, history, local geology, mining history and natural subsidence is provided within L741.R01.

2 Ground Investigation

Due to the large scale of the playing fields, it was decided that the ground investigation should consist of both intrusive and non-intrusive works. The main objective of the non-intrusive investigation was to identify specific problem areas that could then be targeted and followed up with intrusive works.

2.1 Non-intrusive investigation

Electro-Magnetic Induction (EMI)

The EM investigation carried out by TGMS in October 2017 consisted of a 2D survey in which both horizontal and vertical transitions of conductivity are produced. Conductivity is the inverse of resistivity and can be measured by the ease with which electric current can pass through a material. Therefore, it can be theorised that cohesive/saturated materials are more conductive and less resistive than dry granular materials.

The results of both the horizontal and vertical EMI outputs from the investigation indicate a north to south trending area of high conductivity exists within the localised area of subsidence to the west of site. Additional areas of low conductivity are shown in the far east and central portion of the site. These areas align closely with the location of service networks indicated on the utility survey plans.

The higher conductivity area to the west has most likely to be due to the presence of either clay-rich materials or shallow chalk bedrock. The lower conductivity strips shown are due to the presence of a highly dense (possibly metallic) object in the ground, most likely a service cable/pipework. The large areas of additional low conductivity to the east of site are most likely due to the presence of superficial granular materials.

The horizontal EMI has an investigation depth of up to 750mm and the vertical EMI to a depth of 1.5m. Due to the limited depth of investigation additional geophysical investigation was recommended in the form of both Electro-Magnetic Conductivity and Electrical Resistivity Tomography.

Electro-Magnetic Conductivity (EMC)

A follow-up EMC investigation was carried out by Terradat in July 2019 which consisted of utilising electrical conductivity to detect areas of heterogeneous ground; similar to that adopted within the EMI survey. The aim of the EMC survey was to follow up on the outputs obtained from the EMI survey and potentially indicate anomalies within the chalk bedrock.

The survey measured conductivity within the top 4m of ground across the site extents. The results generally support the findings from the TGMS investigation.

Electrical Resistivity Tomography (ERT)

The ERT investigation was carried out by Terradat in July 2019 with the aim of measuring the resistivity of the sub-surface, as well as identifying geological boundaries and potential anomalies within the chalk bedrock.

To remain consistent between the conductivity and resistivity profiles, colour coding was kept the same for both high conductivity and low resistivity. The main outcome of the survey was that two distinct layers were found to be present across site. Layer 1 (L1) comprising of high resistivity material was found to be overlying Layer 2 (L2) comprising of low resistivity (highly conductive) material.

To the east of the site the thickness of L1 appears to be relatively consistent, extending to approximately 10m bgl. L1 then appears to dip down to the west of site within the topographic depression, extending to approximately 15m bgl. L1 appears to become less common to the west of site within the top 5-6m bgl (specifically within the topographic depression), with a localised concentration of higher conductivity apparent within the majority of presented cross sections.

Several localised regions of vertical and sub-vertical high resistivity zones extend through the base of L2 and well into the suspected underlying chalk bedrock layer. These localised regions appear to extend laterally by approximately 20-30m on average and are generally located centrally and to the east of the playing field. These regions have been identified as anomalies requiring further investigation.

The level of groundwater was not known at the time of the survey which would assist with determining whether the two layers are hydrologically controlled. However, from the groundwater observations carried out during the ground investigation, it can be theorised that the layer boundaries are more likely to be geologically controlled (granular deposits or Weathered Chalk overlying a more competent Cemented Chalk bedrock), rather than hydrologically controlled. L1 is thought to primarily comprise of highly Weathered Chalk and the underlying L2 layer is thought to consist of the more intact Chalk material.

2.2 Intrusive investigation

Intrusive ground investigation was carried out in the form of Cone Penetration Testing (CPT), Rotary cored boreholes (RO) and machine excavated Trial Pits (TP).

Cone Penetration Testing (CPT)

Lankelma Ltd were instructed to carry out 45 No. CPT boreholes in August 2019 with the aim of determining the makeup and characteristics of the superficial deposits, Weathered Chalk and Cemented Chalk bedrock across the site extents.

The CPTs were positioned in approximately 50m grid spacings across site, together with specific locations identified from the geophysical survey. Within the 45 No. CPTs advanced, 2 No. piezometers were installed within the topographic depression to the west of site. 4 No. Monster Steek Apparaat (MOSTAP) samples were also obtained within one borehole (CPT 38) to the south-west of site to assist with the interpretation of the ground profile. Refer to as-built exploratory plan within Appendix A for further details.

The penetrometer measurements consisted of cone tip resistance (q_1), friction sleeve resistance (f_1) and dynamic pore water pressure (u_2). See Section 3 of the report for description of the material properties.

Rotary cored boreholes (Ro)

Geotechnical Engineering Ltd were instructed to carry out 2 No. triple-core barrelled rotary boreholes in August 2019 with the aim of obtaining a detailed ground profile with groundwater levels, as well as achieving suitable penetration into the chalk bedrock as to assess the presence of any potential solution features that may exist.

Of the two rotary positions, RO1 was advanced within the far east of site to a maximum depth of 12.5mbgl (41mOD) with the purpose of gaining information regarding the possible reasons for the lack of surface deformation within this region. RO2 was advanced within the topographic depression to the west of site to a maximum depth of 20mbgl (28.6mOD) to gain insight as the potential causes of the surface deformation.

Standard Penetration Tests (SPTs) were undertaken at 1m intervals to 5mbgl and then at 1.5m intervals to termination depth within each of the rotary boreholes. The SPTs were carried out in accordance with BS EN ISO 22476-3:2005 + A1:2011.

Trial Pits (TP)

During the advancement of the rotary boreholes, Geotechnical Engineering Ltd were also instructed to advance 2 No. machine excavated trial pits with the aim of determining detailed ground profiles and groundwater flow within the superficial deposits that could then be compared to adjacent CPT logs for clarification.

The TPs were located in the far north and south regions within the topographic depression to the west of site. TP01 was advanced within the south-east corner of site to a maximum depth of 4.10mbgl (43.9mOD). TP02 was advanced at the northern boundary of site within the topographic depression, to a maximum depth of 4mbgl (45.9mOD).

2.3 Groundwater

Groundwater was recorded within two CPT piezometer installations (CPTi) located in the topographic depression, as well as within the two rotary boreholes during the investigation. Groundwater was not encountered within the trial pit locations. Table 1 illustrates the information obtained from the groundwater level measurements.

Table 1 – Groundwater level measurements

Location	WEST					EAST
Hole ID	CPTi 02		CPTi 36		R02	R01
Date of measurement	27/8/19	29/8/19	27/8/19	28/8/19	30/8/19	28/8/19
Groundwater level (mOD)	None recorded	None recorded	None recorded	None recorded	35.8	50.0
Groundwater depth (mbgl)	-	-	-	-	12.8	3.5
Base of borehole (mOD)	45.7	45.7	37.2	37.2	28.6	41.0
Borehole base depth (mbgl)	2.3	2.3	12.6	12.6	20.0	12.5

From the recorded results, the level of groundwater is indicated to be at a far higher level to the east of site than the west. No groundwater has been recorded within the CPT installations, however the recorded water level in R02 is shown to be below the base of the CPT boreholes.

2.4 Laboratory Testing

The following laboratory tests, summarised in **Error! Reference source not found.**, were carried out on samples retrieved during the ground investigations. Testing was carried out in accordance with the *BS-1377 range - Method of tests for soils for civil engineering purposes*.

Table 2 – Laboratory test quantities

Test	No. of Tests
Moisture Content	10
Atterberg Limits	10
Particle Size Distribution (PSD)	5
Bulk Density (soil)	1
Particle Density (rock)	4
Point Load Test	3
BRE SD1 Suite	4

3 Ground Conditions

3.1 Ground profile

The site ground conditions have been appraised from the exploratory hole log descriptions obtained. Table 3 and Table 4 below summarise the stratigraphy encountered within two cross-sections obtained from both the east and west portion of the site, attached in Appendix B together with a plan illustrating the section locations. Section A-A' represents a section obtained within the topographic depression and Section B-B' a section obtained from the far east of site, both orientated north to south.

Table 3 - Section A-A' Summary

Soil	Description	Thickness (m)	Depth to base (mOD)	Typical SPT
Gravelly silt	Soft to firm, sandy very gravelly clayey SILT. Gravel is of sub-angular to sub-rounded coarse flint.	0.7 – 1.1	47.0 – 48.8	7
Clayey silt	Firm, slightly sandy very clayey SILT.	0.5 – 2.5	44.9 – 48.3	9
Sands/gravels	Medium dense, very sandy angular to sub-rounded fine to coarse GRAVEL of flint.	0.8 – 2.8	42.1 – 45.8	20
Putty Chalk	Structureless CHALK composed of off-white slightly gravelly SILT. Gravel is of sub-angular and sub-rounded fine to coarse very weak chalk. [CIRIA Grade Dm].	3.2 – 9.9	35.0 – 42.3	9-18
Weathered Chalk	Very weak to weak low-density off-white CHALK. Fractures are extremely closely spaced. [CIRIA Grade B5].	4.1 – 6.0	36.1 – 41.8	75-103
Cemented Chalk	Weak medium density white CHALK. Fractures are 5-30° closely spaced. [CIRIA Grade B4].	>4.0	-	143-750

** highly weathered Putty Chalk layer only encountered within RO2 & CPT18*

Table 4 - Section B-B' Summary

Soil	Description	Thickness (m)	Depth to base (mOD)	Typical SPT
Hard gravels	Dense to very dense sandy angular to sub-rounded fine to coarse flint GRAVEL.	2.8 – 3.0	49.7 – 50.8	45-50
Putty Chalk	Structureless CHALK composed of off-white sandy gravelly SILT. Gravel is of sub-angular and sub-rounded fine to coarse very weak chalk with rare inclusions of angular fine to coarse flint. [CIRIA Grade Dm].	1.6 – 3.2	47.6 – 48.9	20-35
Weathered Chalk	Very weak low density white with frequent black specs CHALK. Fractures are randomly orientated extremely closely spaced. [CIRIA Grade C5].	0.90	46.7	-
	Very weak to weak low-density off-white CHALK. Fractures are randomly orientated. [CIRIA Grade B5].	7.4 – 9.7	40 – 41.5	19-39

** Putty Chalk only encountered in RO1, CPT11 & CPT22*

Superficial Deposits

Within Section A-A', the superficial deposits primarily consist of cohesive materials in the form of gravelly clayey silt over a maximum thickness of 3.0m from surface. A sandy gravel layer was found to underly the cohesive materials in the majority of the boreholes in this section, with a maximum thickness of 2.8m. The maximum thickness of the superficial deposits within Section A-A' was recorded at 6.0m.

SPT N values recorded within RO2 ranged from 7 to 20 within the superficial deposits, indicating firm consistency cohesive materials and medium dense granular materials in accordance with *Table 8 of CIRIA Report 143 – SPT Methods & Use*.

Within Section B-B', the superficial deposits consist entirely of dense flint gravels over a maximum thickness of 3.0m from surface. SPT N values recorded within RO1 ranged from 45 to 50 within the superficial deposits, indicating very dense granular materials.

Putty Chalk

Analysing the two sections, there appears a notable difference in the thickness of the CIRIA Grade Dm 'Putty Chalk' material. To the west of site along Section A-A', the thickness ranged from 3.2 to 9.9m and to the east of site along Section B-B', the thickness ranged from 1.6 to 3.2m.

SPT N values recorded within RO2 (Section A-A') ranged from 9 to 18 indicating a very weak Chalk. N values within RO1 (Section B-B') ranged from 20 to 35 indicating a very weak to weak Chalk. Although the remoulded Putty Chalk material has been characterised in a similar way, the difference in strength is notable within the two rotary boreholes.

Weathered Chalk

The thickness of CIRIA Grade B5 weathered chalk appears to be greater within Section B-B' with a thickness range of 7.4 to 9.7m compared to a range of 4.1 to 6m within Section A-A'.

SPT N values within RO2 ranged from 75 to 103 indicating a weak Chalk and N values within RO1 ranged from 19 to 39, indicating a very weak to weak Chalk.

Cemented Chalk

CIRIA Grade B4 Cemented Chalk was only recovered within RO2, with a recorded thickness of >4m. Rotary borehole RO1 was considered onsite to have refused on the Cemented Chalk layer at 12.5m bgl but a sample was not recovered to confirm this assumption.

4 Results

The results of the laboratory testing within the chalk strata have been divided up for testing carried out in both the east and west regions of site, see Table 5 and Table 6.

Table 5 - Laboratory test results (east)

Borehole	Depth (mOD)	Material Type	MC (%)	IP (%)	IL (%)	Bulk Density (kN/m ³)	UCS (MN/m ²)
RO1	49.7	Putty Chalk	27.5	9	28	-	-
RO1	46.7	Weathered Chalk	26.6	7	51	-	-
RO1	43.7	Weathered Chalk	-	-	-	18.8*	-
RO1	41.1	Weathered Chalk	-	-	-	18.7*	-
RO1	41.1	Weathered Chalk	-	-	-	-	1.0

* Derived from particle density results based on 30% porosity. MC = moisture content, IP = plasticity index, IL = liquidity index, UCS = Uniaxial Compressive Strength

Analysing the liquid and plastic limit testing, both the Putty Chalk and Weathered Chalk can be graded as a very low plasticity material. The liquidity index values indicate the Putty Chalk and Weathered Chalk lie between their respective liquid and plastic limits. The bulk density values of the Weathered Chalk samples lie within typical values for Chalk, however the UCS value of 1MN/m² is lower than the typical range of 4 to 7.7MN/m² encountered within medium/high density chalks (obtained within CIRIA C574 Chapter 7, *Shallow Foundations*).

Table 6 - Laboratory test results (west)

Borehole	Depth (mOD)	Material Type	MC (%)	IP (%)	IL (%)	Bulk Density (kN/m ³)	UCS (MN/m ²)
RO2	42.5	Putty Chalk	32.1	9	90	16.9	-
RO2	37.7	Putty Chalk	29.2	8	65	-	-
RO2	35.1	Putty Chalk	25.9	8	36	-	-
RO2	34.4	Weathered Chalk	-	-	-	18.8*	-
RO2	33.2	Weathered Chalk	-	-	-	-	1.2
RO2	30.0	Cemented Chalk	-	-	-	18.6*	-
RO2	28.8	Cemented Chalk	-	-	-	-	1.6

* Derived from particle density results based on 30% porosity.

Both the Putty Chalk and Weathered Chalk within the topographic depression area can also be graded as a very low plasticity material. The Putty Chalk sampled at 5mbgl (42.5mOD) displayed a natural moisture content approaching its liquid limit ($IL = 90\%$), the remaining Putty Chalk samples had a liquidity index within the middle third of the liquid index.

The bulk density of the Putty Chalk appears to be markedly lower than that derived for the Weathered Chalk, the latter remaining consistent with that recorded within RO1. The recorded UCS values fall below what is expected for structured Chalk material.

5 Assessed cause of the subsidence

A key finding from the Desk Study Site Investigation Report (L741.R01) was that the most likely mechanism behind the observed settlement, put forward by the author, was due to infiltration of surface water into the underlying Chalk causing a weathering effect. This weathering effect most likely creating solution features in the form of solution pipes or collapse sinkholes. Natural cavities in the form of solution pipes were considered to be located in the far eastern portion of site from the desk study but no evidence for these was found in the investigation.

Findings from the investigation revealed no obvious indicators to the presence of fissures or locally voided regions within the ground. A likely cause of ground subsidence would be if loose overlying material was being eroded to fill any fissures created in the Chalk. Although not identified there is potential for fissures to be present below or between the investigation points.

However, the results of both the non-intrusive and intrusive investigations only appear to signify the presence of a thicker layer of a highly Weathered Chalk underlying the area of the topographic depression to the west of site, when compared to the chalk profile to the east of site.

Considering the findings, it is theorised that the thicker layer of putty chalk has been created by excessive groundwater percolating through surrounding ground and reacting with the underlying chalk, causing it to weather into a mouldable/softer material. Although the Groundwater Table (GWT) is low to the west of site, Putty Chalk located above the GWT appeared saturated upon sample recovery during the ground investigation. This suggests that groundwater flow is regularly occurring via an external source. Anecdotal evidence indicates that water loss from the school swimming pool, located in close vicinity to the depression, has been occurring for many years and still continues. Although not considered to be the sole cause of the weathering, this water loss is considered to be exacerbating the subsidence problem. Therefore, it is recommended that the leak is fixed as soon as possible.

Due to the absence of laboratory consolidation testing due to inadequate sample recovery within the rotary boreholes, empirical relationships obtained from CIRIA C574 have been used to determine stiffness characteristics of both intact chalk and remoulded Putty Chalk.

CIRIA C574 Chapter 4, Mechanical Properties of Chalk explains that significant reduction in the Young's Modulus of a Chalk material occurs when the imposed bearing pressure increases beyond its yield stress. Generally, with increasing water content, the liquid limit of a remoulded Chalk is likely to be reached due to its low plasticity. This would significantly reduce its yield stress properties. In layman's terms this means that when the imposed stress exceeds the yield stress of a Chalk, the rate of settlement significantly increases. Therefore, it can be theorised that a remoulded Putty Chalk would have a lower yield stress and would therefore undergo a higher rate of settlement under a lower bearing pressure than that of an intact Chalk material.

CIRIA C574 Chapter 7, Shallow Foundations – Section 7.5 presents secant undrained Young's Modulus (E_s) value ranges for both low density Grade B and Grade D Chalk materials (obtained from reviewed test data). The findings showed that the lower bound E_s for a low-density Grade

B Chalk is typically 200 MN/m^2 and for a Grade Dm Chalk the E_s is 6 MN/m^2 . The immediate settlement of a soil is a direct function of its undrained Young's Modulus; therefore, it can be theorised that under identical loading the Grade Dm Putty Chalk would be more likely to experience greater settlement than that of a Grade B intact Chalk.

Although the above findings are based on comprehensive data analysis, specific consolidation characteristics of the site-won chalk would have ideally been confirmed via laboratory testing.

The low topography of the western area of site means that surface water infiltration is more likely here and this will increase the weathering effect on the underlying Chalk. If left untreated this weathering effect would weaken the chalk, potentially leading to further subsidence.

6 Design Options

Firstly, it is considered that the eastern area of the site that has shown no significant signs of settlement and does not require specific remediation.

To mitigate against further subsidence within the topographic depression located to the west of site, a series of remedial options have been proposed to provide stability to the ground in this region.

6.1 Geotextile separator and geogrid reinforced soil

To prevent the possibility of groundwater washing out fine grained materials within the superficial deposits and to provide additional tensile strength to the ground overlying the Weathered Chalk, an option is to introduce a combination of geogrid layers within compacted reworked site won superficial deposits together with a non-woven geotextile basal layer overlying the putty chalk.

By utilising a geogrid and geotextile combination, lateral confinement forces within the soil would be increased which would create a bridging effect over the underlying Putty Chalk. This will help to reduce the settlement that transfers up to the surface if continued settlement of the underlying Putty Chalk occurred. This methodology does not treat the underlying Putty Chalk but provides a bridging layer over the top, designed to mitigate the extent of significant differential settlement observed at the surface.

6.2 Lime Stabilisation

By improving the engineering properties of the superficial deposits through the addition of a lime binder, the stiffness and strength of the treated ground would increase.

The superficial deposits would be required to be excavated and subsequently mixed with the lime binder to form the stabilised material before being reintroduced.

With these strength enhancements combined with increased durability, the effects of surface deformation within the treated area would be reduced.

This methodology does not treat the underlying Putty Chalk but provides a bridging layer over the top to mitigate the extent of settlement observed at the surface. A trial would also be required to see if this methodology was valid and what dosage rates would be recommended.

6.3 Vibro-stone columns

Vibro-stone columns increase the allowable bearing capacity at the surface by displacing the superficial deposits (and therefore densifying and increasing their strength and stiffness behaviour). They would be founded down on the intact Chalk and therefore do present a potential pathway for water ingress into the aquifer. This would need to be considered.

6.4 Surcharging the ground

Pre-loading the existing ground within the topographic depression prior by introducing additional materials to increase the level of the playing field prior to the permanent works is considered an effective method of limiting settlement.

By allowing controlled consolidation to take place prior to construction this ensures that any further ground subsidence is mitigated.

The surcharge material will most likely consist of imported granular material that would be built up to an appropriate level as to impose an appropriate bearing pressure required to consolidate the existing ground.

The effectiveness of the surcharge is dependent on the magnitude of load that can be safely applied to the ground and the time taken for consolidation to occur. It is likely that the construction programme would be significantly affected by a surcharge solution.

6.5 Deep Soil Mixing

Similar to lime stabilisation, deep soil mixing consists of enhancing the strength and stiffness properties of the existing ground using a combination of bentonite and cementitious binders.

However, the technique differs by using a mechanical power mixer to inject the binders to the ground in-situ. This allows the process to be carried out more quickly and allows the additives to be mixed in at greater depths than that of lime stabilisation.

This methodology would be appropriate for treating of the Putty Chalk although consideration of the type of binder and the presence of the underlying principal aquifer would need to be made.

6.6 Drainage controls

As mentioned in both the Desk Study and Section 3 of this report, excessive groundwater migration into the topographic depression region is considered to be creating the highly weathered/soft Chalk layer. This highly Weathered Chalk layer is believed to be undergoing high levels of settlement, creating surface subsidence.

Introducing drainage controls around the depression region would capture groundwater and allow it to be discharged elsewhere reducing the risk of weathering and any erosion of moisture sensitive materials. This would mitigate the reduction of the stiffness parameters of the intact Chalk and reduce any ongoing settlement.

The attenuation basin could be formed by interlocking a series of Wavin aquacell infiltration units (or similar) which are designed to capture and store high volumes of groundwater whilst discharging it at a controlled rate. This method will reduce surface water ingress but does not mitigate against water flowing beneath the level of the drains or within the Chalk itself.

6.7 Compaction grouting

To improve the properties of both the superficial deposits and underlying Chalk within the topographic depression, compaction grouting within a series of boreholes could be undertaken. This remedial option was put forward in the Desk Study, specifically for the purpose of infilling sub-surface voids to strengthen the ground.

The principle of compaction grouting is that a high viscosity mortar grout displaces existing loose ground within the immediate vicinity of the borehole, creating overlapping 'grout bulbs'. The grout would improve the strength and stiffness characteristics of the ground, reducing the settlement potential of the ground. The grout would also reduce the ground permeability, protecting any underlying Chalk but potentially requiring drainage to avoid saturation and waterlogging of the sports pitch. There is also environmental consideration with the underlying principal aquifer as it is difficult to know exactly where the grout will go when pumped.

6.8 Reinforced Concrete slab

Another option put forward in the Desk Study report is to bridge over any significant voids located within the chalk strata by casting a reinforced concrete slab within competent ground above. The slab would be constructed within localised regions of the Putty Chalk material. Therefore, significant excavation and subsequent re-compaction of overlying soils would be required.

Although areas of voided ground in the Chalk were not identified during the investigation, this option could be used to reduce any differential settlement effects.

6.9 Options appraisal

To determine the optimal remedial solution for this site, each of the eight options has been critically appraised. The main advantages and disadvantages of each option are summarised in Table 7.

Table 7 - Options appraisal

Design Option	Advantages	Disadvantages
Geogrid/ geotextile	• Easy to install • Cost-effective • Durable	• Does not address underlying Putty Chalk and hence may not fully negate surface movements
Lime stabilisation	• Durable • Low maintenance	• Requires an onsite trial • Can be expensive to construct • Time consuming • Does not address underlying Putty Chalk and hence may not fully negate surface movements • Materials to be imported to site
Vibro-Stone columns	• Addresses underlying putty chalk • Highly durable	• Expensive to construct • Potential to contaminate principal aquifer • Materials to be imported to site

Design Option	Advantages	Disadvantages
	Sustainable	
Surcharging ground	Can reduce risk of consolidation settlement following the construction phase	- Extends construction period - Safety risk due to bearing stability and slopes - Materials to be imported to site
Deep Soil Mixing	- Easy to install - Durable - Low maintenance	- Expensive to construct - Potential to contaminate principal aquifer - Materials to be imported to site
Drainage controls	Cost-effective	- Not a robust solution (can crack or fill with debris) - Requires deep excavation to address Putty Chalk - Time consuming - Maintenance required - Risk of flow beneath the drainage controls still weakening the Chalk
Compaction grouting	- Addresses underlying putty chalk - Highly durable - Easy to install	- Expensive to construct - Potential to contaminate principal aquifer - Materials to be imported to site
RC Slab	- Robust - Will most likely negate differential settlements	- Expensive to construct - Time consuming - Materials to be imported to site

6.10 Preferred design solution

ByrneLooby consider the most cost-effective design solution to consist of installing a combination of both **geotextile and geogrid layers** within site-won material.

Existing superficial deposits within the region to the west of site, where the topographic depression is located, should be fully excavated. From both the intrusive and site information, this depth typically ranges between 3.7 to 4.5m bgl.

Once the existing ground has been removed, a suitable Terram non-woven geotextile layer should be placed to retain fine material within the compacted backfill into the underlying Chalk.

The excavated soil should then be re-introduced over the geotextile and compacted in layers in accordance with the Specification for Highways Works (SHW) 600 Series. At each layer interface, Tensar biaxial geogrid mesh should be placed with the aim of enhancing the tensile strength and stiffness properties of the existing superficial soils. Both the geotextile and geogrid mesh should extend laterally beyond the topographic depression and tie into adjacent competent ground to create a suitable bridging effect over any underlying chalk that is prone to subsidence.

The design outlined above is considered to limit excessive differential settlements, however surface movements may not be fully mitigated due to deflections within the reinforced occurring in the long-term when the underlying chalk settles. To further increase the rigidity of the ground, ground strengthening measures within the underlying chalk in the form of either micro-piles or vibro-stone columns could be installed. The additional strengthening measures should be constructed in combination with the reinforced superficial soils to provide a complete solution. However, it should be noted that this combination method would be considerably more expensive and time-consuming to construct. The predicted deflections for each option could be assessed at detailed design stage to determine the benefit of the deeper ground strengthening and compared against the expected long-term performance of the pitch at ground level.

An additional recommendation is to incorporate a suitable drainage blanket into the surfacing of the new pitch development with the purpose of diverting surface water to an appropriate collection point, recommended to be located within the south-west corner of the field. By doing this any excessive surface water migration into the underlying ground would be mitigated.

7 Conclusion

Following on from the range of options proposed within Section 6, a detailed design is highly recommended following selection of the preferred option. We also recommend a follow-up meeting to discuss the options with the yourselves and an appointed principal contractor in due course.

Other considerations that should be made:

- Monitor surface profile to the east of site. Although no subsidence has occurred and remediation is not required at this time, Putty Chalk was located with the boreholes advanced in the eastern regions and therefore has the potential to subside.
- Observe surface water flow within the localised depression. Any information that can be communicated to ByrneLooby in the interim would aid the remediation design process.
- Any water leakage from the swimming pool should be assessed and if found to still be going on, this should be fixed as soon as possible.

Appendix A – As-built Exploratory Hole Plan

Appendix B – Cross-Sections A-A', B-B' & C-C' & Plan

Appendix C – Terradat – Geophysical Survey Report

Appendix D – Lankelma – CPT Report

Appendix E – Geotechnical Engineering – Ground Investigation Factual Report