GEOSYNTHETICS AND THE 12M LOCK GATE: AN ACCOUNT OF THE USE OF GEOSYNTHETICS IN LAND PREPARATION FOR A NEW SEA LOCK IN SWANSEA

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Abstract: The regeneration of Swansea's docklands area under the Welsh Assembly sponsored SA1 redevelopment scheme has seen a dramatic and varied change in land use, with recent residential and commercial development concentrated around the Prince of Wales Dock. The proposed redevelopment of the dock as a major marina coupled with the continued requirement for commercial shipping access to the neighbouring Kings and Queens Docks, will require the construction of a sea lock with 12m high outer sector gates.

The lock is located within a former tidal basin which is underlain by over 6m of very soft marine mud overlying sand and gravel. Provision of a construction platform for the new lock entailed infilling a substantial part of the old tidal basin to form an 8m-high reinforced soil embankment, with face stability and durability provided by a cellular geosynthetic confinement system. Some 2,500 wick drains were installed through a construction blanket and the marine mud to permit a rapid completion of settlement within a managed timescale.

All construction methods and materials were selected following a systematic risk analysis in order to facilitate land-based operations, working initially at low tide, whilst satisfying stability, settlement and environmental constraints in an area with a tidal range of some 10m. Nearly 90% of fill materials were recycled from local sources within SA1, supported by extensive environmental risk assessment to ensure that the sensitive marine environment was protected and that client aspirations and economic requirements were met.

This paper deals with the use of geosynthetics in successfully delivering the adopted construction methods including their use within the working platform, management of the consolidation of the mud, stabilisation of the steep embankment, and provision of embankment face confinement. Typical values of soil and material design parameters will also be presented and discussed.

Keywords: Artificial embankments, coastal, consolidation, durability, ground improvement, reinforced embankments.

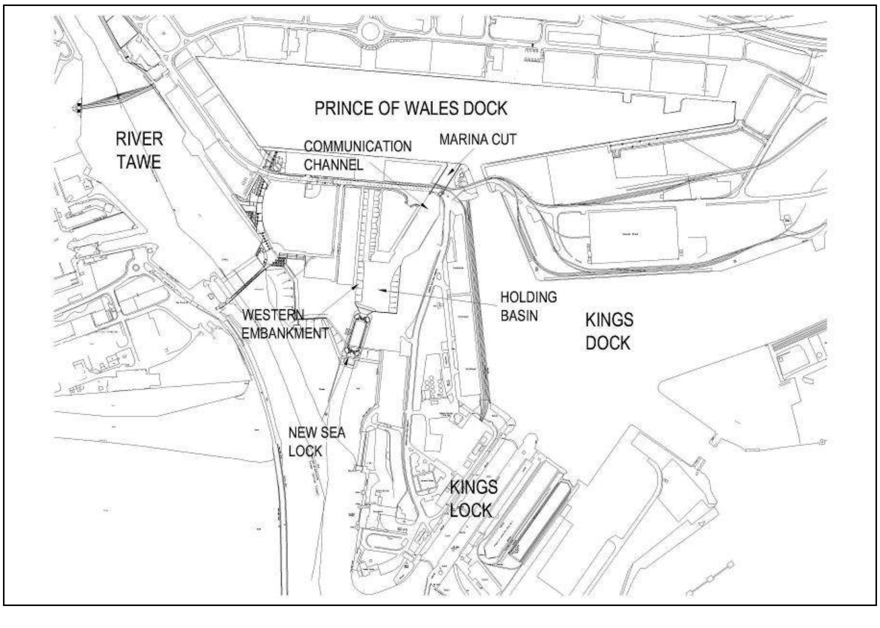

Figure 1. Schematic layout of Sea Lock Development Area

INTRODUCTION

The city of Swansea straddles the River Tawe in the south western region of Wales, United Kingdom, approximately 60km west of Wales' capital city of Cardiff. The construction of the Prince of Wales Dock in the late 19th century set the precedent for this stage of Swansea's industrial heritage with the later addition of the Kings and Queens Docks cementing the city's reputation as an area of significant maritime importance. Subsequent to the decline of the principle industrial activities, metaliferous and coal mining, the Port of Swansea has undergone a dramatic downturn in status, and thus the concept of the SA1 Waterfront Development was conceived as a means of transforming it into an area of major strategic importance for the wider development and future prosperity of Swansea.

This development, part funded by European Union and Welsh Assembly Government (WAG), will transform this area of prime development land into a striking mix of residential, leisure and commercial land use with the Prince of Wales Dock reborn as a focal marina, with strong pedestrian and vehicular links to the city centre seen as key to the success of the development. At the heart of the reconfiguration of the Prince of Wales Dock will be its segregation from the wider commercial interests of the operator of the commercial port, Associated British Ports (ABP) and the development of a new Marina Cut linking the new marina with the proposed sea lock in the mouth of the former tidal basin. Figure 1 shows the proposed layout of the development area.

The extreme tidal range in this sector of the Bristol Channel, coupled with a requirement to permit access to the marina on a 24/7 basis, will necessitate the construction and installation of outer sector gates with a height of greater than 12m, as well as a proposed lock barrel length of 50m and width of 18m.

An extensive program of site investigation, as well as geotechnical and environmental laboratory testing was undertaken to provide detailed site specific geotechnical design information and to identify site specific design constraints, whilst ensuring that WAG requirements to use locally derived recycled materials would not adversely impact upon the sensitive marine environment. The information so obtained provided design parameters to facilitate the construction of an 8m high embankment on soft marine muds and reinforced earth embankments to form part of the land platform required to accommodate the proposed sea lock and holding basin.

GEOLOGICAL SETTING AND GROUND INVESTIGATION

The superficial geology of the immediate area is characterised by river and tidal deposits comprising a mix of organic cohesive soils and sand and gravel overlying mudstone and sandstone from the coal bearing strata of the Middle Coal Measures of Carboniferous age; bedrock is encountered typically at a depth of between 15m and 20m. The widespread metaliferous and mining industries within South Wales resulted in substantial thicknesses of made ground being deposited across wide areas of Swansea, with this material used in conjunction with dredged sand and gravel to form the docklands area.

Investigative works were undertaken to provide a clear understanding of the geological conditions on site with initial works undertaken between 2005 and 2007 in a number of phases. The initial investigations focused on the land surrounding the proposed sea lock site and the information used to provide a design manual for the planned development of the wider SA1 Waterfront Development. A marine based site investigation was commissioned, both to obtain geotechnical data for the design of the planned embankment and to establish levels of contamination within the marine deposits, as part of seeking a Department for Environment, Food and Rural Affairs (DEFRA) permit to deposit locally dredged materials. A total of 61 boreholes, 46 machine excavated trial pits, 20 super heavy dynamic probe holes and 7 window sample holes were sunk within the vicinity of the sea lock development.

The land based investigations revealed an extensive cover of made ground of up to 7m depth, overlying cohesive material containing localised pockets of soft organic clays and silts with some bands of fibrous peat before entering the dense gravels and underlying mudstone. The marine site investigation revealed the stratigraphy within the tidal basin to comprise approximately 6.5m of soft to very soft marine muds overlying approximately 7m of dense to very dense marine gravel.

The made ground was generally attributable to two main sources, the upper deposits representing locally derived soils from industrial processes comprising sand, gravels, slag, ash and brick, with the deeper materials principally consisting of dredged sand, gravel and some cohesive material, which were used to reclaim this part of the tidal estuary for the purposes of constructing the dockland area.

Table 1. Summary of Initial Design Parameters

Stratum	SPT value	Shear strength				Consolidation		Permeability
		Total stress		Effective stress				
	N (blows)	ϕ_u (°)	c_u (kPa)	ϕ' (°)	c' (kPa)	m_v (m^2/MN)	c_v (m^2/yr)	k (m/s)
Made ground – granular								
- up to 5m depth	6	-	-	28	0	-	-	-
- below 5m depth	11			30	0			
Made ground – cohesive	2	0	12	-	-	1.0-1.2	-	-
Land based marine deposits – granular								
- up to 7m depth	12	-	-	30	0	-	-	5×10^{-7}
- below 7m depth	24			35	0			5×10^{-7}
Land based marine deposits – cohesive								-
-above 7m depth	3	0	20	28	0	0.4	2	
-below 7m depth	6	0	30	28	0	0.4	2	
Tidal basin marine deposits – granular								
- Sand	33	-	-	38	0	-	-	-
- Gravel	50				0			
Tidal basin marine deposits - cohesive	0	0	5	28	0	-	-	-

PROJECT CONSTRAINTS AND CONSTRUCTION WORKS

The initial proposals to permit the construction of the sea lock site entailed the removal of all the superficial marine deposits present within the approach channel, the footprint of the lock bund and the over the entire area of the holding basin by marine dredger to expose the natural sands and gravels. The resulting void within the holding basin was to be used for the future deposition of surplus materials arising from the excavation for the lock barrel construction. In order for this proposal to proceed, the necessary approvals were sought from the relevant regulatory authorities, which in this instance included DEFRA, the Environment Agency and the City and County of Swansea.

The required dredging license was obtained from DEFRA in December 2005, although this license did not permit the deposition of materials deemed contaminated beyond stringent marine criteria. The marine site investigation demonstrated that repair and maintenance works within the former dry dock to the north of the tidal basin, which ceased operations in 1968, had given rise to significant and widespread pollution of the marine muds. Dredging was, as a consequence, restricted to the area beyond the mouth of the basin to form the marina approach channel and a 5m wide trough to be used to form the toe of the outer face of the proposed lock seawall. Dredging was undertaken by UK Dredging, a wholly owned subsidiary of ABP, and completed in August 2006; UK Dredging acted as sub-contractor to Morrison Construction (MC), appointed under successive contracts by WAG in early 2006, under a NEC Option A contract administered by White Young Green (WYG).

As a result of the soft marine deposits remaining within the tidal basin and under the footprint of the planned sea lock, extensive enabling works were required within the entrance to the former tidal basin to enable plant from the surrounding land areas to access the marine muds within the basin. Initial preparatory works were undertaken during summer of 2006 to form a stable toe constructed of UK Highways Agency Class 6A selected fill and locally procured quarry waste, at the seaward extent of the sea lock construction bund. The toe was placed immediately following dredging works along the tidal channel to a depth of approximately -6.50mAOD, which marks the typical boundary between the marine muds and the underlying sand and gravel.

Enabling works on the landward side of the sea lock bund involved the overcoming of an elevation difference of approximately 8m; this was managed through a staged filling operation using long reach tracked excavators probing and removing the soft muds, allowing the placement of UK Highways Agency Class 6 engineered fill to form an access road which was continued across the rear of the lock bund to form the landward toe.

These and subsequent enabling works were undertaken within a very limited access window at near mean tide level of approximately 0.0mAOD, within a tidal range at this point of the Bristol Channel of -5.0m OD to +5.5m OD. Such works required considerable advanced planning on the part of MC's Project Manager for the procurement of specialised plant, ready access to locally stockpiled materials and mobilisation of staff during unsocial working hours to ensure timely completion of key construction works.

Upon completion of the restraining toe bunds, the infilling of the area between the seaward and landward toes was undertaken via a controlled program of tidally controlled works to construct a temporary plant access platform. These works involved the placement of a geosynthetic fabric on top of the marine mud, with an approximately 1m thick construction blanket formed using imported UK Highways Agency Class 6A and locally recycled UK Highways Agency Class 6F1 selected fill materials. The construction blanket thus formed the working platform at approximately +1mOD allowing the installation of some 2500 wick drains required to accelerate the consolidation of the underlying soft marine muds, as well as reducing the amount of settlement remaining upon completion of the placement of fill to form the 8m high sea lock bund.

The greater portion of the materials employed were recycled from the wider SA1 development area; the exception being the use of UK Highways Agency Class 6I/J material for the construction of the reinforced earth (Fortrac 80/30-20 and 110/30-20) face of the western embankment forming the inner tidal basin.

The reuse of materials arising from redevelopment works within SA1 was a critical economic driver in achieving realisation of the £300M WAG flagship project, as previous industrial processes undertaken within the dock area, metal processing, arsenic works, ship repairs and coal handling would generally render the surplus arisings being classified as Hazardous Waste for disposal at commercial landfill sites.

The reuse of locally generated fill materials and the realisation of the sea lock constructional requirements offered a potentially logical and practical means of eliminating very significant transportation and disposal costs, and reducing fill importation costs and logistics. However, the use of these materials within the sea lock works was only fully sanctioned following extensive Qualitative Risk Assessment work on all surplus materials, carried out by WYG Environmental, using relevant marine threshold values for soil, leachate and groundwater. Close liaison with the various departments of the local EA office was essential in resolving issues of temporary storage and unrestricted reuse to ensure compliance with Waste Management Regulations.

Although presented with a very limited construction period, inclement Swansea weather, tidal working conditions and problems of material sourcing, the initial phase of the Sea Lock construction was successfully completed by WYG and MC within programme and budget in August 2007.

MODELLING OF SETTLEMENT OF MARINE MUDS

Monitoring of Filling Operations

The very low recorded shear strength and physical grading of the marine muds remaining below the proposed construction platforms were predicted to give rise to excessive settlement and adverse pore water pressures during and after the infilling stage of the works. Based on initial settlement predictions formed using a Microsoft Excel spreadsheet using the ground condition information then available, and subsequently augmented by the results of the ground investigation works, management of this settlement process required consideration of the use of vertical drains and possible surcharging.

An undertaking of this nature invariably involves the development of close working relationships between designer, contractor and client. MC's Project Manager on site was keen to contribute to the practical implementation of the overall requirements of the scheme and was proactive in adapting working practices to incorporate and protect for the duration of the contract, a total of eight settlement and groundwater monitoring stations, which were an essential element of WYG's requirement to verify the validity of design assumptions for consolidation of the marine muds and the installation of the geotextile reinforcement of the western embankment.

Four of the monitoring stations were located within the newly constructed western embankment and the remainder within the sea lock bund areas of the site. A total of eight stations were considered sufficient to ensure adequacy of data points without imposing upon MC's ability to undertake the reclamation and filling of the tidal basin. Each settlement and groundwater monitoring station was contained within a 2m diameter concrete manhole ring to preserve integrity of the installed instrumentation and to minimise the risk of damage from heavy plant used in the filling operation.

On completion of the wick drain installation, settlement monitoring plates were installed via localised excavation through the construction blanket and underlying geosynthetic. Metal tubes were attached to $1m^2$ metal settlement plates, which were placed on the interface of the fill and underlying marine muds. A 50mm slotted HDPE standpipe was forced into the marine mud within each station in order to monitor the response in groundwater pressures within the marine muds to the ongoing filling operations.

The bulk of the monitoring effort was concerned with the measurement of settlement as this was considered to be a critical factor in ensuring the optimal operational and maintenance conditions for the proposed Sea Lock, which with a design life requirement of 120 years and lock gate operational tolerances of 3mm, was intolerant to future differential settlement of the underlying strata.

Settlement Modelling

The relatively simple schematic model of the bund and western embankment was designed to represent predicted ground conditions, the planned construction / loading sequence and the use of surcharge where material availability permitted. The following settlement influencing factors were integral to the design of the spreadsheet.

- settlement predictions for multiple compressive layers;
- a facility to specify staged loading, including unloading (removal of surcharge);
- allowance for the combined effects of vertical and horizontal drainage where vertical drains are installed;
- secondary compression estimates; and
- a graphical output to allow predicted and measured settlement-time plots to be compared, so that the monitoring data could be used to obtain progressively improved consolidation parameters, enabling steadily improved predictions to be made of remaining settlements and settlement rates.

Initial analyses

Initial settlement predictions were obtained using values of the coefficient of volume compressibility, m_v, and the vertical and horizontal coefficients of consolidation, c_v and c_h, obtained from known relationships and laboratory tests performed on samples obtained from the various ground investigations. Laboratory values of m_v are normally considered to give a reliable guide to field behaviour but, especially in marine deposits, laboratory c_v and c_h values are known to be unreliable since they do not include effects due to the macro-structure of the soil, such as interbedded sand layers which tend to speed up consolidation. The degree of uncertainty was progressively reduced throughout the infilling and subsequent monitoring period, as the nature of the settlement model employed allowed for the continual refinement of design parameters based on the observed degree of settlement over time.

These initial spreadsheet predictions were used to estimate the effects various combinations of loading sequence, vertical drain spacing and surcharging would have on the amount of settlement and consolidation rate. The flexibility of the model enabled the optimum vertical drainage spacing configuration to be realised. It was decided, based on initial estimates, that the consolidation process could be sufficiently accelerated by the use of vertical drains alone, without the need for surcharging.

Monitoring and back analysis

Once drain installation and filling commenced, settlements were measured at the eight monitoring stations on a daily basis for an initial period of 4 months, with weekly monitoring undertaken for a further 3 months and then monthly for 3 months. The data was entered into the settlement model, with the remaining predicted settlement and rate of consolidation being regularly updated to refine the combined curves for predicted and measured settlement with time for each of the stations. As mentioned previously, the inherent uncertainty surrounding the use of laboratory derived values of m_v, c_v and c_h was judged to have been adequately addressed by comparison of the predicted and measured settlement rates to construct values of these parameters specific to the materials used on site.

In the early stages of consolidation, when the time-settlement plot is almost linear, any difference between predicted and measured settlements may be overcome by modifying either m_v or c_v/c_h values. Since it is generally considered that laboratory m_v values are the more reliable parameter, any differences observed were initially overcome by modifying the c_v/c_h values. However, as consolidation of the marine muds progressed, the most appropriate combination of adjustment of the parameters became apparent. Furthermore, in the latter stages of the consolidation period, the monitoring results also allowed an estimate to be made of the secondary compression parameter, $C_{\alpha\varepsilon}$.

As successive settlement readings became available, improved settlement estimates were realised by performing this back-analysis, the accuracy of predictions steadily increasing as the monitoring period increased. This continuing process of monitoring and re-appraisal of the theoretical settlement-time calculations enabled predictions of the required settlement period to be confirmed with reasonable confidence from an early stage. Predictions of the remaining settlement, once consolidation was substantially complete, and its development over time, could also be confidently predicted. The benefits of being able to provide the client with accurate predictions of settlement rates and of the scale of time required for consolidation can be appreciated, as these predictions were extensively used in the onward programming of the remaining aspects of the overall sea lock and associated infrastructure.

Material properties

Values of m_v and c_v/c_h obtained from back analysis are compared with laboratory design values, where available, in Table 2. It can be seen that the assumption that laboratory m_v values are normally realistic and that laboratory c_v values are normally lower than reality, proved to be reasonably true for the SA1 development site. The field derived values of m_v display some overlap whilst the values of c_v show a marked difference from the laboratory derived values.

Table 2. Comparison of geotechnical design parameters

Design Parameter	Laboratory Derivation	Field Derivation
m_v (m²/MN)	1.2 – 3	0.56 – 1.44
c_h (m²/yr)	-	0.23 – 0.50
c_v (m²/yr)	0.35 – 0.81	1.2 – 6
c_h:c_v	-	1:5.2 – 1:12
c' alpha	-	0.04

USE OF GEOSYNTHETICS

Treatment of Foundation Soils/Working Platform

It was originally envisaged that the very soft mud deposits would be removed by dredging in order to provide a suitable foundation for the proposed embankment works to fulfil the dual requirements of stability and minimisation of the long term settlements. Contamination levels within the muds precluded this action and therefore treatment using deep in-situ soil mixing techniques by means of specialised marine based plant to form continuous rows of stabilised / solidified columns was proposed. A geotextile reinforced granular mattress was proposed above the barrettes to promote arching within the fill and distribute the embankment loads. The overall DEFRA licence for the marina development permitted the use of the cement/concrete columns within the tidal basin, however due to delays in the award of the construction contract, an alternative solution was proposed by MC which would allow the rapid treatment of the mud deposits between tides utilising readily available plant. Based on a detailed assessment of the consolidation characteristics of the mud deposits it was decided that geosynthetic wick drains could be utilised to increase the rate of consolidation within a managed time period compatible with the overall marina development programme.

It was necessary to construct a working platform over the very soft marine muds to permit the safe installation of the wick drains. BRE Report BR470 'Working Platforms for Tracked Plant' 2004 has generally been adopted within the piling industry as a good practice guide for the design, installation, and maintenance of working platforms. The design method detailed within the report is based on punching shear through a stiff platform overlying a soft material and the analysis has been extended to include the use of geosynthetic materials to reduce the required thickness of granular platform material. The punching shear theory is generally limited to a subgrade with undrained shear strength in excess of 20kPa, however in the case of the very soft mud the undrained shear strength was less than 8kPa. CIRIA SP123 'Soil Reinforcement with Geotextiles' 1996 provides a more general method for the control of bearing capacity failure by the use of geosynthetics and was therefore adopted as the basis of the design.

Based on the calculated bearing pressures of the proposed wick drain rig (100kPa) it was calculated that an 800mm thick granular platform would be required, reinforced with a 40kN composite geogrid/geotextile to perform the dual functions of reinforcement and separation. MC substituted the geogrid/geotextile composite material with a 100kN polyester geotextile within the construction to perform both functions, with the increased stiffness having the effect of minimising the mobilised strains within the material. Although the use of an analytical procedure to determine the required platform thickness provides a basis for the construction, the actual method of construction over very soft soils, described in Section 7, has as much, if not a greater, effect on the performance of the platform.

Imported Class 6A and locally recycled Class 6F1 selected fill materials were placed within 1 hour access windows at low tide, and the operation proved a success in allowing the installation of some 2,500 wick drains to a depth of between 7 and 9m in a 6 week period.

Reinforced Earth Western Embankment

A requirement of the proposed Western Embankment was the need for it to be constructed at a relatively steep inclination to facilitate mooring of vessels within the holding basin pending movement through the lock. In order to provide a stable embankment it was necessary to utilise reinforced earth techniques to provide the required tensile capacity within the fill material. A number of slope inclinations were investigated and a slope of 1:1 chosen based on economy and robustness. Although the reinforcing of embankments with geosynthetics is fairly well established and the methods of analysis are well documented, the proposed embankment at SA1 had a number of complicating factors which influenced the design and specification of the materials including drainage within the embankment, static and dynamic water regimes, seepage, scour, erosion and construction in a tidal regime.

The choice of fill material for use within the reinforced earth embankment was restricted due to the SA1 sustainability target of re-using the majority of site won materials. The ideal would have been to use a granular material with a high permeability in order to manage the construction and in service water pressures within the embankment; however the specification was restricted to a target grading, shear strength, and minimum compaction requirement. The available material was of relatively low permeability such that the pore water pressures within the embankment could reach relatively high values, particularly following impounding of the marina followed by a rapid drawdown event. These high water pressures present adverse conditions for stability of the embankment, interaction of the geosynthetic material with the fill, and stability of the embankment facing.

A seepage analysis was undertaken based on an estimated permeability, however due to the variability of the fill it was decided to adopt a cautious approach of assuming full hydrostatic pressures within the embankment with an external low tide level. A number of analysis methods were used and compared including BS8006 1995 Code of Practice for Strengthened/Reinforced Soils and Other Fills, HA68/94 Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques, and CIRIA SP123. The latter two methods provide a simplified table/chart based approach to determine the required length of the geosynthetic and the required vertical spacing based on a theoretical horizontal stress. The use of UK Highways Agency specifications are applicable in this instance as this design involves the construction of reinforced embankments, as well as using specified earthworks materials for use below water. These specifications are recognised design standards within the UK.

BS8006 provides a framework for the use of number of alternative methods and is suited to the analysis of complicated geometry and soil/water conditions, and was subsequently used to check the stability of the geosynthetic reinforcement layout derived from HA68/94 and CIRIA SP123.

Both the minimum geosynthetic length and the maximum vertical spacing were dictated by HA68/94 ($L_B/H = 2.1$ for HA68/94 compared to 1.8 for SP123, and K=0.5 compared to K=0.4). The type of fill, and the strain at failure, dictated the use of high tenacity polyester material as the geosynthetic reinforcement. Interaction of the geosynthetic with the fill material, and in particular with the type of facing, is crucial to the transfer of stress from the soil to the reinforcement. To maximise the stress transfer a geogrid material, as opposed to a geotextile, was chosen. Fortrac grades 110/30-20 were required towards the base of the embankment where the horizontal stresses are greatest, and grade 80/30-20 utilised towards the top of the embankment to provide economy. The basal 2m of embankment required a 400mm vertical spacing of reinforcement, increasing to 600mm for the remaining 5m. The maximum spacing of 600mm was dictated by the stability of the facing. Both ultimate limit state and serviceability limit states of failure were checked to BS8006 for the range of internal, external, and compound failure mechanisms, with no modifications to the derived layout being required.

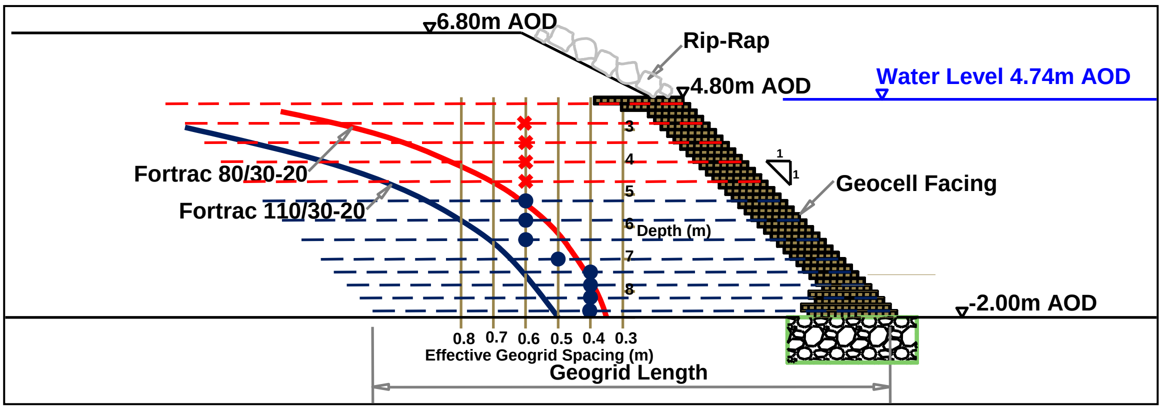

Figure 2. Maximum Geogrid Vertical Spacing Diagram and Actual Geogrid Vertical Spacing

A number of alternative facing methods and materials are available for use with reinforced earth slopes. A temporary geogrid wraparound slope was constructed as part of the site enabling works, however the conditions of tidal range and wave attack highlighted weaknesses in the facing and a loss of fill resulted in slumping of the face. The type of facing also had to be sufficiently robust to perform the function of an armour layer and be modular such that it could be installed quickly during low tide and allowed to become submerged immediately after placement without detriment during high tide. A rip rap type of facing was considered and discounted due to risks associated with adequate retention of the retained fill under tidal and wave conditions, and the requirement to provide sufficient anchorage for the geogrid reinforcement.

A geocell type facing material was chosen due to its ability to be filled with mass concrete 'off-line' prior to placement and lifted into position during the tidal window. Due to potential degradation effects from the high alkaline content of cast concrete a HDPE material by PRS-GB was chosen. Each geocell layer was 200mm high and each lift set back 200mm to create the required face inclination. The required width of the geocell was derived from an analysis of the required width to mobilise sufficient anchorage force within the geogrid reinforcement to stabilise near surface failure planes and provide sufficient anchorage for the geocell facing to prevent localised movement, bulging, and pull-out failure under static and dynamic conditions.

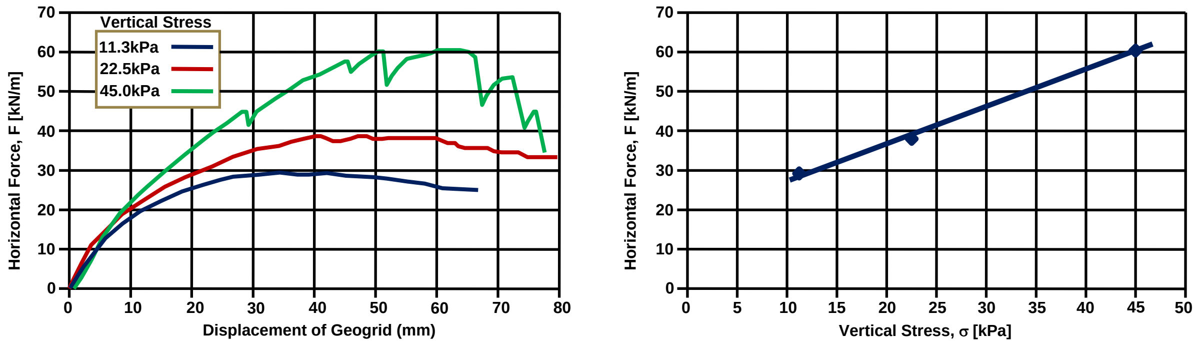

Figure 3. Geogrid/Geocell Connection Capacity

The failure mechanisms are akin to those considered within the analysis of segmental modular concrete block walls, with the added problem of water and wave effects. In order to assess the connection capacity of a segmental block wall testing of the connection is normally undertaken in accordance with the US National Concrete and Masonry Association methods which measure the available pull-out resistance for particular grade of geosynthetic reinforcement for a given concrete block width, under various normal load conditions. The resulting connection capacity is then used within the analysis of the retaining wall by limiting the available geosynthetic force to that of the factored connection capacity. Connection capacity tests undertaken using Fortrac 110/30-20 and PRS-GB Geocell are shown in Figure 3 based on an embedded geosynthetic length of 0.8m**.**

The vertical stress at an individual geosynthetic location can be evaluated using the concept of 'hinge height', as used for segmental block walls. This stress is modified by the effects of static water and dynamic wave action to produce a design effective vertical stress. The generation of dynamic excess pore water pressures due to wave action and the structural response of the armour layer were assessed using a wave run-up and run-down analysis in accordance with CIRIA SP83 1991 'Manual on the use of Rock in Coastal and Shoreline Engineering'. It was therefore possible, using the tested connection capacity, to determine a peak horizontal force and a serviceability force which would cause failure or excessive movement at the connection and to compare this to the actual force and thereby calculate the required geocell width.

Based on iterations within the analysis the required geocell width was calculated as 1.4m (7 cells). In order to provide a sufficiently durable armour layer, prevent loss of embankment fill material due to piping, provide for a rapid installation and provide for economies of materials, the 7 cell width unit was divided into the front 3 cells being filled with mass concrete in advance of the main works, and the rear 4 cells being pegged and filled in-situ with a graded drainage stone to act as a filter (Figures 4 and 5). The 'pre-cast' 2.5m x 1.4m geocell panels were lifted into position during low tide and had sufficient mass to prevent displacement during high tide. The basal 4 geocell layers were increased in size up to 13 cells to control seepage beneath the toe of the slope and prevent piping.

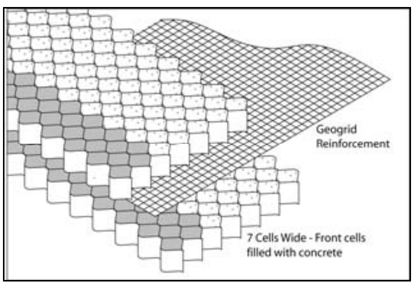

Figure 4. Isometric view of Geocell Connection **Figure 5.** Geocell armour during construction

GEOSYNTHETICS INSTALLATION

Working Platform

The construction method employed in the installation of a working platform over very soft soils is critical to the successful performance. Often the temptation is to place too large a thickness of granular material to create a capping layer; however the weight of this material alone can result in a bearing capacity failure. The fill material must be placed in a controlled manner and MC achieved this by use of a long reach excavator from pre-prepared access points at low tide which manoeuvred the roll of geotextile into position and then carefully placed the granular fill onto the geotextile. The placed fill material was then used as the leading edge in order to progressively place the geotextile and fill into the basin to construct the platform to its full width; the tracks of the excavator effectively compacting the fill layer. The platform performed as designed and supported the weight of plant and machinery for the installation of the wick drains and the subsequent construction of the embankment.

Reinforced Earth Western Embankment

The programme of works was devised to avoid extensive temporary works and enable the construction of the embankment to be undertaken during what were initially very limited access windows at low tide. The time taken to prepare the working platform was utilised to pre-cast the concrete armour layer within the front 3 cells of the proposed geocell facing, as shown in Figure 6. The prepared geocell facing units were quickly and efficiently lifted into position at low tide using a normal 360 excavator and a bespoke lifting mechanism developed by MC, see Figure 7.

Figure 6. Precast Geocell Armour Layer

Figure 7: Positioning Geocell Armour Layer

The inherent stability of the concrete filled geocell facing enabled the part constructed embankment to be submerged during high tide without detriment to the constructed section. The process of placing the geogrid reinforcement, and placing and compacting the fill material followed conventional embankment construction, enabling rapid and economical construction under difficult conditions.

CONCLUSIONS

Without the use of geosynthetics within this engineering application, this project would have been subject to greater construction preparation, construction costs and time periods. The formation of the working platform was only rendered practical by the use of geosynthetic materials – geofabric and geogrid, which also significantly reduced the amount of imported material needed to build the platform. Further substantial economic and time savings were made through the use of wick drains to initiate a rapid and controlled consolidation period that was required prior to the construction of the 8m high sea lock bund and embankment, as well as the geosynthetic materials – geogrid and geocell, used in the construction of the western embankment.

The nature of the end use of the holding basin dictated a slope angle of 1:1 to allow for suitable mooring conditions for vessels entering the proposed sea lock. The use of a geocell facing system enabled this slope angle to be achieved whilst ensuring its overall stability and integrity was assured throughout its design life. The geocell system obviated the need for using specialist quick setting concrete allowing for more control on quality and robustness of finish on the armour face of the western side of the new holding basin.

The issue of sole reliance on laboratory derived engineering parameters has also been further explored and it is the opinion of these authors that, although laboratory testing is considered a viable and integral part of engineering design, there can be no substitute for in situ field derived values that can be used to validate original designs. Opportunities to obtain field specific data should be created and taken during any initial data gathering site investigation stage, where appropriate and when conditions allow.

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