THE USE OF DOUBLE TWISTED WIRE MESH STEEL GEOGRIDS IN SOUTHERN AFRICA

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ABSTRACT

Weak compressible soils located along the east coast of Southern Africa invariably require improvement in their strength to safely carry both commercial and industrial structures. In particular large earth fills, or industrial surface beds cannot economically be carried on piles, or similar load transfer mechanisms. Accordingly, consideration has been given to strengthening weak soil by using locally available wire mesh steel geogrids manufactured by Maccaferri Southern Africa. Since the introduction of wire mesh geogrids to strengthen soils in KwaZulu Natal in 2002 there has been an increase in their application to strengthen weak soils below earth fills and industrial surface beds. This paper deals with 3 cases where wire mesh steel geogrids have been very successful in reducing settlement and improving bearing capacity.

INTRODUCTION

In KwaZulu Natal, the north eastern province of South Africa, geotechnical investigations have identified areas where weak soft clay, or loose sands present both settlement and bearing capacity concerns relating to stable development. In most instances the use of piles, or other similar mechanisms to carry loads are considered inappropriate in terms of cost, especially for small and medium sized enterprises. In order to provide a cost effective solution alternative methods of strengthening weal soils have been considered. The use of synthetic geotextiles to strengthen fill banks is well documented, but most synthetic geotextiles exhibit large percentages of elongation to achieve maximum tensile stresses. In Figure 1 below is shown elongation trends for a variety of materials, but the one of interest to the projects discussed in this paper is that of steel.
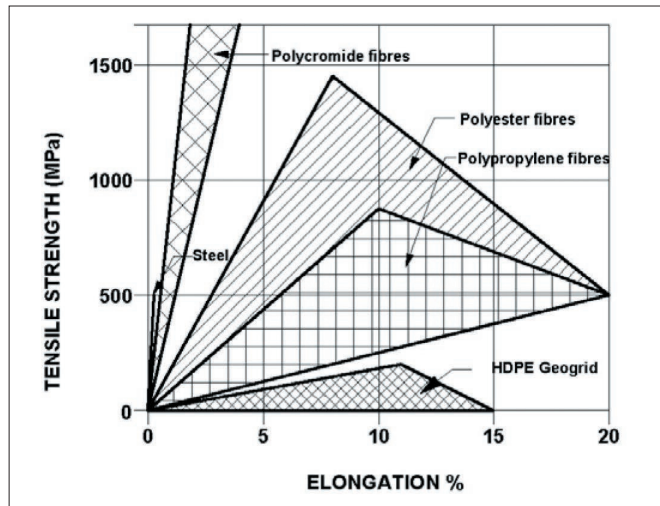

FIGURE 1

The type of wire mesh steel geogrids which have successfully been used in the three case studies mentioned in this paper to strengthen weak soils is a double twisted steel mesh, type 80, with transverse reinforcing steel rods inserted through the mesh. One version of this product has one side of the wire mesh panel lined with a pre-attached geotextile to act as a separation barrier. The make-up of the mesh panel is shown in Figure 2 below together with typical strength characteristics:
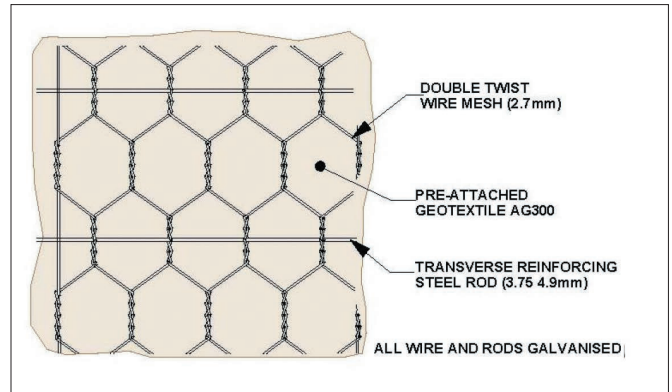

FIGURE 2 Details of Flexmesh Geogrid

• maximum Elongation at 1 500 MPa tensile stress is 1.0%.
• minimum tensile stress up to 500 MPa
• minimal transversal strength
  2.7 mm x 3.9 mm – 47 kN/m
  3.0 mm x 4.9 mm – 50 kN/m
• minimal longitudinal strength
  2.7 mm x 3.9 mm – 32 kN/m
  3.0 mm x 4.9 mm – 39 kN/m
• class A Galvanised.

Both double twisted mesh grids are typically used for providing instant hardstanding for emergency roadways, for access across weak soils providing poor subgrade support, and for road stabilisation. The high strength low elongation properties of the mesh together with its local availability drew attention to its possible use as an additive to weak soils to improve their tensile strength. By improving the soil tensile strength both settlement control and bearing capacity would improve. The following 3 case studies provide a clear insight into the advantages of using the mesh as a strengthening agent:

• St Helliers Townhouse Development, Durban
• STAM Sugar Store, Maputo, Mozambique
• large Distribution Centre, Durban.

FIGURE 3 Typical section through the wetland
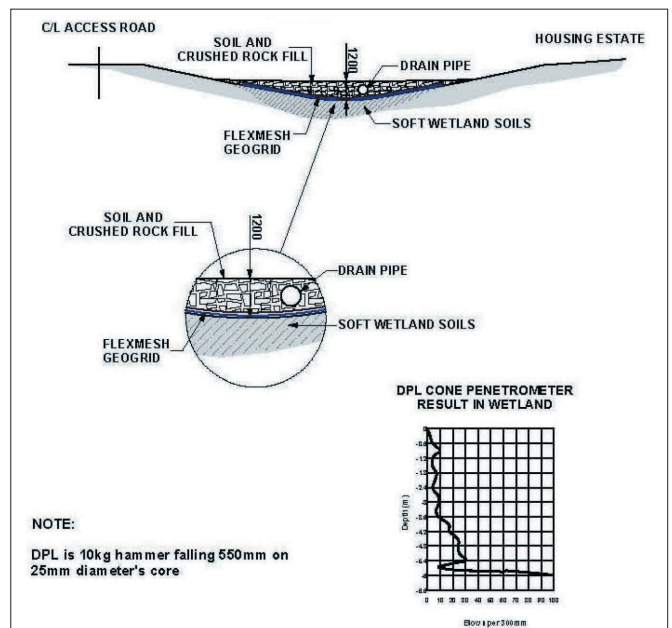
ST HELLIERS TOWNHOUSE DEVELOPMENT, DURBAN, SOUTH AFRICA

In 2002 a townhouse scheme was proposed on the gentle slope of a hill located above a lake. In order to access the site from a residential road a wetland had to be crossed. An investigation carried out in the wetland showed the underlying soils to consist of 3.0 to 4.0 m normally consolidated soft layers of silty clay interspersed with loose silty fine sands. The wetland forms part of the upper reaches of the lake and is under water for most of the rainy season. In order to keep development costs under control it was decided to replace the original concept of a low-level bridge, with an earth fill bank supporting the access road.

Figure 3 shows a typical section through the wetland together with the location of the geogrid. The geogrid was introduced to reduce the amount of settlement expected due to the weight of the fill as well as prevent slip failure of the sides of the fill. A single geogrid layer was introduced directly onto the surface of the wetland followed by crushed rock. The thickness of the rock layer was 1.0 m. A further 1.5 m of soil from the site was added to raise the fill about 2.2 m above the wetland. This surcharge of fill on the wetland soil was anticipated to result in 50 to 100 mm of settlement, but more likely slip failure of the sides of the rock fill. A drainage pipe was built into the earth fill to allow drainage from the wetland into the lake. It was envisaged that settlement of the earth fill would result in the drain pipe being partially covered by wetland soil. Provision was made to place a second drain pipe above the initial one to allow continued flow from the wetland into the lake. This was however not necessary. The geogrid performed extremely well providing the necessary strength improvement in the underlying soft and loose soils to prevent slip failure and cap settlement to within a few millimeters. In Table 1 below is shown the magnitude of expected settlement in comparison to actual settlement.

TABLE 1 Predicted and actual settlement

Estimated m_v m²/kN		Earth Fill Load kN	Settlement	
Range	"N"		Predicted (mm)	Actual (mm)
0.0004 to 0.0006	<2	50	60 to 90	>20

The range of mv values shown in Table 1 have been derived using the work undertaken by Brink et al (1982) and Webb (1970).

The predicted and estimated settlement magnitudes shown in Table 1 provide an insight into how effective the wire mesh steel geogrids can be in reducing settlement, but more importantly improving soil strength to prevent slip failure in the soft clay. From the estimated SPT "N" value derived for the soft clay located on the surface of the wetland soil, the undrained shear strength of the soft clay would be in the range 10 to 15 kN/m². Since the earth fill applied load is about 50 kN/m², some 3 to 5 times that of the shear strength of the clay, it is reasonable to expect that without the geogrid the sides of the earth fill would have failed.

STAM SUGAR STORE, MAPUTO, MOZAMBIQUE

The expansion consisted of a new sugar store measuring 40 x 140 m and having a storage capacity of 50 000 tonnes.

The store is contained on all 4 sides by a 6 m high concrete retaining wall connected to the concrete surface bed. The height of the cone of sugar at the centre of the store is 18 m when filled to capacity, exerting floor bearing pressures of 50 kN/m² below the walls and about 170 kN/m² at the centre. An extraction tunnel was to be located below the surface bed running the length of the store on the centre line. Early evaluation of the geology of the site suggested that the entire sugar store would need to be supported off piles to mitigate the expected settlement resulting from consolidation of the underlying soft clayey harbor soils.

Investigations carried out within the footprint of the proposed sugar store showed that the site is underlain by about 1.6 m of well compacted imported fill. In turn this fill is underlain by deposits of very soft to soft dark grey organic clay interspersed with layers of loose to medium dense grey fine to coarse sand. Underlying the clayey horizon is hard to very dense layers of clayey sand and sandy overlying siltstone bedrock.

These soils recorded the following SPT 'N' values from 2 boreholes put down at either end of the proposed stores.

TABLE 2 Recorded SPT 'N' Values

Depth (m)	Description	N Value
0.0 to 1.6	Dense sandy fill	±15
1.6 to 2.0	Very soft clay	Less than 3
2.0 to 3.0	Soft clayey sand	3 to 6
3.0 to 5.0	Loose to medium dense sand	6 to 18
5.0 to 6.5	Medium dense fine sand	11 to 18
6.5 to 7.5	Medium dense fine sand	11
7.5 to 9.0	Stiff clayey sand	27
9.0 to 12.0	Stiff to hard clayey sand	44

The borehole soil samples obtained did not realistically reflect the variable nature of the soils below the site. Cone penetrometer tests (CPT) were carried out at the site to provide more detail regarding soil strength variation. Set out below in Figure 4 is the result of one such cone penetrometer test carried out at the site.
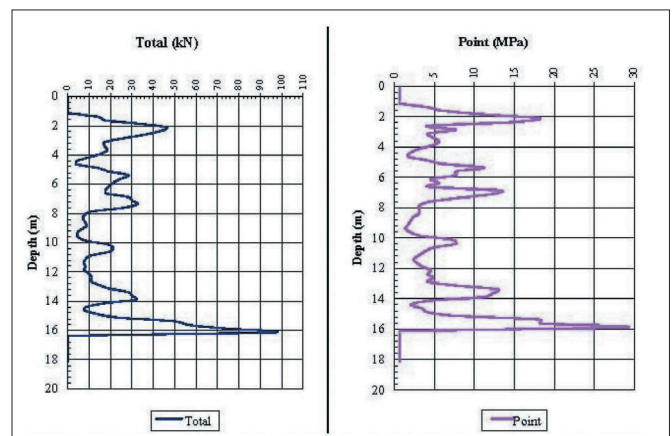
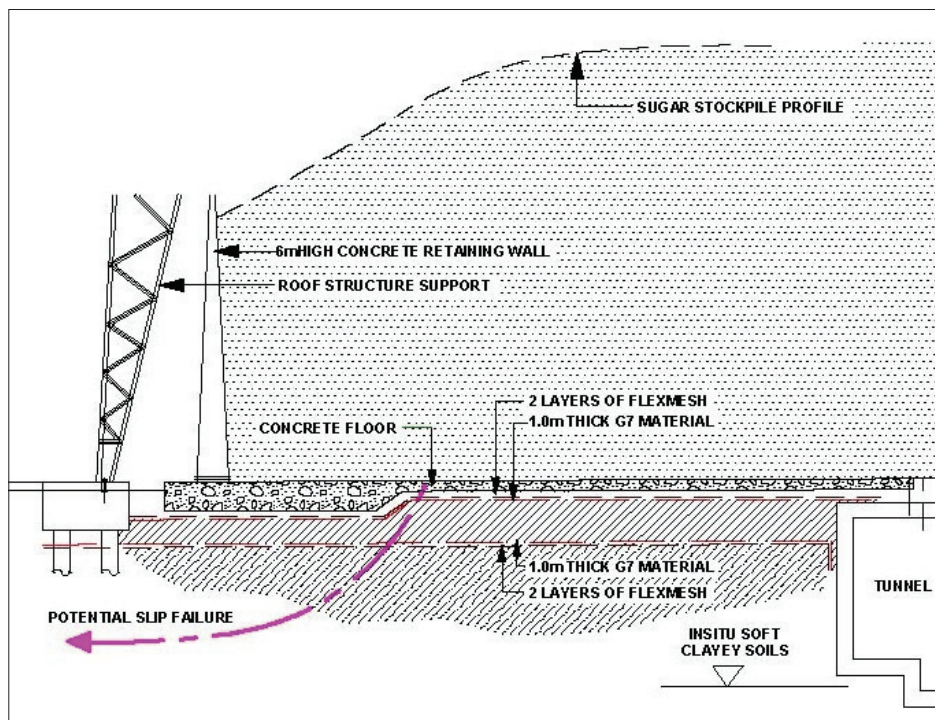**FIGURE 4** Cone penetration test – DCP8

Using the expressions of de Beer and Martens (1975) and Schmertmann (1970) settlements were estimated assuming no strength improvement and allowing for a creep period of five years. The following settlements were estimated.

TABLE 3 Estimated Settlement Range

Location	Edge (50 kN/m²)	Centre (170 kN/m²)
Northern End	Up to 30	Up to 90
Centre	Up to 40	Up to 100
Southern End	Up to 75	Up to 225

The magnitude of the settlement estimated in Table 3 illustrates the need for either piling, or some other form of soil strength improvement. The cost of piling would have put a halt on construction of the store as the proponents did not have the additional funds. A request was extended to the geotechnical consulting practice of Moore Spence Jones (Pty) Ltd to investigate ways and means of providing an economical solution to
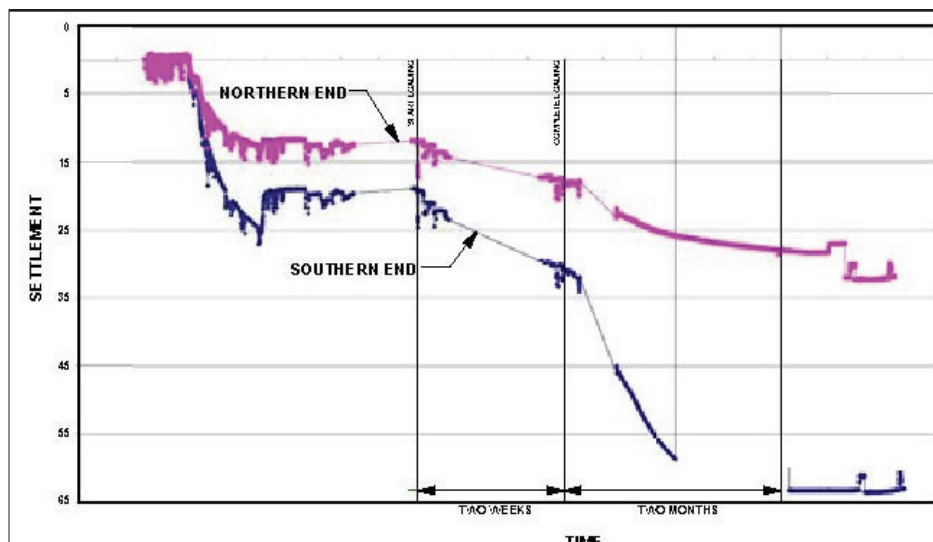
FIGURE 6 Extensometer results

sugar store. A maximum factor of safety of 0.9 was calculated, and this together with the fact that the full applied pressure on the floor edge was as much as three times the shear strength of the underlying clays gave clear indications for the need to improve soil strength. The design adopted made allowance for increasing the layers of Flexmesh above and below the engineered soil raft until the factor of safety against rotational failure of the concrete retaining wall exceeded 1.5.

Part of the design included for the 1st time phase loading of the sugar store to allow excess pore water pressures to be controlled through gradual dissipation. Unfortunately the sugar store was fully loaded over a space of a few days. The settlement curve shown in Figure 6 below indicates the reaction of the sugar store to sudden loading and the effect of rapid drainage through the sand lenses.

The full load stayed in place for 1 to 2 months. At the edge of the store settlement did not appear to exceed 15 mm. At the southern end of the store where severe settlement was expected magnitudes of up to 65 mm appear to have taken place. More importantly remained stable with very little if any evidence of movement.

construction and filling the sugar store has without any need to resort to remedial measures. performed very well reducing differential settlement ble tolerances regarding structural damage, ure of the boundary concrete retaining walls.

DURBAN

centre was proposed on a site to the north of problems in the form of an extensive wetland distribution centre was to be built. Drainage had been directed into the wetland thereby saturation of the clayey sandy soil which .0 m. Underlying the clayey soil is medium In order to accommodate dock levelers fill ed to the site. This added a surcharge of between 60 to 80 kN/m². Foundations supporting the large surface structure as well as surface bed load added to the fill surcharge. Column foundation bearing pressures are 150 kN/m², and below the narrow aisle 14 m high racking system the bearing pressure is about 40 kN/m².

In Figure 7 below is shown schematically the layout of applied loads exerted on the compressible soil below the wetland.

In Table 4 below is shown the soil strength differences as recorded at DPL 20 and 8 shown in Figure 8. These differences indicate potential differential movement across the distribution centre.

Shown in Figure 8 is the approximate extent of the wetland superimposed on the footprint of the distribution centre. Taking the applied loads into account as mentioned above the
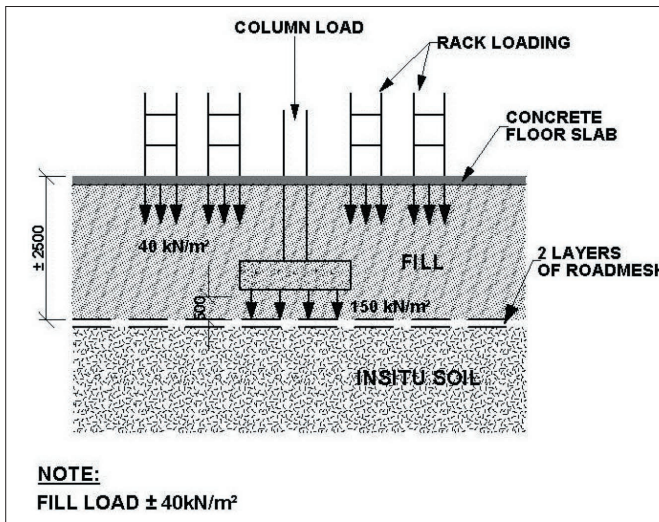
FIGURE 7 Typical section through distribution centre

TABLE 4 DPL Cone Test Results

Location	Blow Count per 300 mm Penetration									
DPL 8	17	23	14	28	43	37	25	66	60	98
DPL 20	2	4	10	4	4	7	6	4	8	9
DPL 8	Ref									
DPL 20	7	9	17	18	24					
	Hammer weight = 10 kg			Falling 550 mm						
	Cone = 25 mm diameter			Rods = 16 mm diameter						

settlements given in Table 5 were calculated using the expressions of Brink et al (1982) and Webb (1970).

In view of the concerns expressed regarding differential settlement across the distribution centre it was agreed that soil improvement was necessary. Conventional methods of soil improvement proved to be expensive. It was decided to investigate the use of wire mesh steel geogrids

FIGURE 8 Showing wetland footprint

LARGE 45 000m² DISTRIBUTION CENTRE
SITE LAYOUT SHOWING DRAINAGE WETLAND
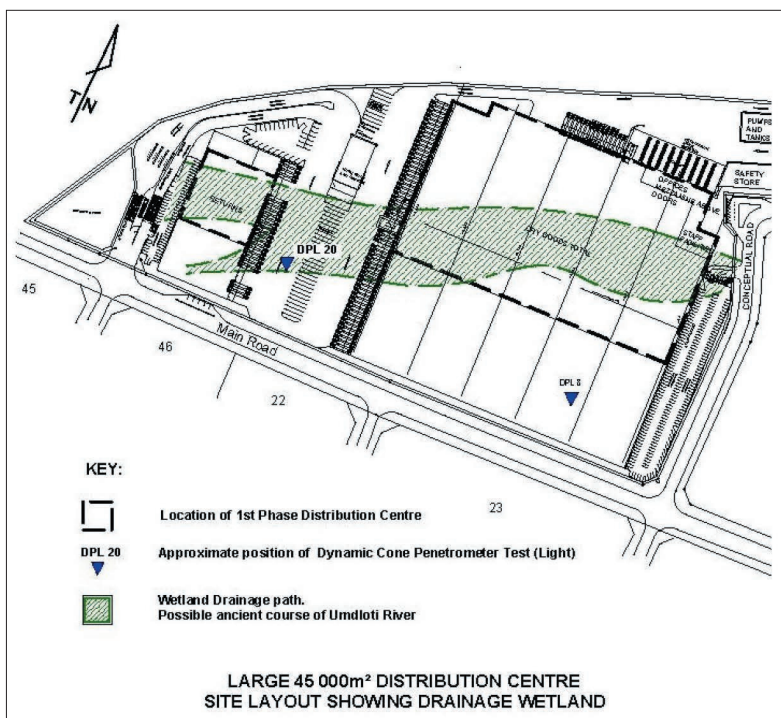
TABLE 5 Estimated settlement range (mm)

Location	Applied load (kN/m²)	Settlement
Edge structure	Fill + columns + floor (240)	25 to 35
Centre structure	Fill + columns + floor (240)	85 to 95

to induce tolerable differential settlement. This meant that the geogrids would carry the load of the fill, foundations and surface bed.

The geogrids were laid with the main rods 3.9 mm laid at right angles to each other in each of the 2 layers. In all, some 100 000 m² of geogrid were laid down over the footprint of the distribution centre. The geogrid layers extended at least 3.0 to 5.0 m (depending on the thickness of the fill) beyond the footprint of the distribution structure to provide sufficient anchorage and to prevent slippage.

The surcharge of the fill and the design length of the geogrid beyond the structure ensured that the full transversal load of 47kN/m of each layer could be developed. Furthermore, column foundations were placed at least 0.5 m above the 2 geogrid layers and bearing pressures kept to 150 kN/m² to avoid stressing the geogrid wire and rods. By extending the geogrids beyond the edge of the distribution structure maximum anchorage into the wetland was assured. This allowed the geogrids to introduce an added tensile component into the insitu soil to resist consolidation. The fill was completed in 3 months followed by construction of the foundations. At present the top structure is 95 percent complete and settlement monitoring indicates that settlement across the distribution structure is not more than 10% of the predictions given in Table 5 with the full applied load from fill and foundations in place. The racking loads are still to be applied, but software modeling that has been used and updated using the recorded settlements, predict total settlement which is not likely to exceed 15 to 20 percent of the predicted values given in Table 5. More important however, is the impression that the more harmful differential movements expected are not likely to materialize.

CONCLUSION

The case histories mentioned in this paper together with other instances where steel wire geogrids have been used successfully to strengthen soils below industrial structures, as well as reservoirs, clearly indicates that in Southern Africa these geogrids have provided successful economical solutions to strengthen weak soils. It is however important to adopt a geogrid system that will integrate the weakness of the soil with the applied loads of the structure to provide acceptable bearing pressures and settlement. Projects for possible geogrid stabilization must be chosen with care taking into account soil strength variation and the structures tolerance to settlement.

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ST HELENA AIRPORT PROJECT

Graham Isaac

WorleyParsons RSA – Lead designers
Sub-contracted to Basil Read – Main Contractor

INTRODUCTION

The St Helena Airport Project provides many unique and unusual features requiring advanced engineering ingenuity and planning. The remote location of the island has major logistical considerations as almost everything, excluding rock and water, has to be shipped to the island. The island's heritage and fragile environment provides a significant legacy of international acclaim requiring preservation and particular awareness in detailing aspects of the project's design and construction processes.

Although construction works for the Airport continue through to February 2016 many of the associated infrastructure works are well underway with the detail design work scheduled for completion at the end of July 2014.

PURPOSE OF THE PROJECT

St Helena is one of the most geographically isolated islands in the world located approximately 1 950 km from the south- west coast of Africa and 2 900 km from South America. Since the island's discovery in 1502 the only access has been by sea with the current maximum size and weight of any single component having to be transported by the mail ship, the RMS St Helena. Landing infrastructure on the island is limited, with no breakwater or mooring facilities at the seafront (Jamestown Bay). Cargo is transported ashore using towed barges and passengers are ferried to and from the ship by small launches.

The economic viability of St Helena is dependent on the frequency and reliability of access for people and goods to the island. The airport project is destined to change the lives of all St Helenians, known as Siants, not only in providing employment with opportunities of new skills, but it will boost the island's economic growth with increased tourism and stimulate the expansion of support industries.

AIRPORT INFRASTRUCTURE DESIGN ASPECTS

Landside Engineering

The remoteness of the island, its size, logistics of materials supply, unique geology, topography and climate, endemic biodiversity with sensitive environmental heritage, ethnic diversity and history have all prompted innovation in the design of specific aspects of the Project Infrastructure. In addition the project design features required careful consideration of appropriate constructability and programming of design delivery requiring close integration of the design and construction team members. Features of the project consist of the following:

- Construction of a temporary jetty for off loading all construction plant as St Helena has no permanent wharf facility.
- Temporary fuel installation dedicated for the construction plant demands (1.5 million litres facility).
- Permanent bulk fuel installation for storage of 6 million litres of fuels (Jet A1, Diesel and Gasoline).
- 14.5 km access road with the fist 3.5 km having to traverse severe rocky terrain with a maximum vertical grade of 15% and cross slopes of 1 in 3. The majority of this section of the road has been designed and constructed largely in cut.
- Bulk earthworks for the airfield consists of drilling and blasting of predominantly basaltic igneous rock for the bulk fill of the Dry Gut located at the southern end of the runway (with 100 m high terraced profile requiring 8 million m³). This aspect of the project presented the biggest challenge in ensuring stringent final level tolerances in supporting a concrete runway pavement (6 mm in 3 m straight edge) – refer to section below for a more detailed description of the design and construction process.
  - Associated with the rockfill processing, borehole explorations were conducted to source suitable yielding groundwater, and the construction of temporary water storage dams (4 x 2 million litre HDPE lined facilities).
- Construction of a concrete dam to attenuate runoff from the Dry Gut catchment area to facilitate controlled stream-flow release upstream of a 2.0 m x 680 m long concrete box culvert.
  - Structural design considerations for the required 120 year design life with use of appropriate and proven performances from concrete additives.
  - Further value engineering has resulted in this structure being replaced with an open channel drain with the excavated material being used for balancing the Dry Gut fill requirements.
- 2 km long concrete surfaced runway which gives an effective 1550 m available landing distance.
- Building services including electrical power supply and reticulation, thermal modelling, cooling via mechanical ventilation and wet services to the Airport Terminal Building, Air Traffic Control Tower and Fire Services.

Dry Gut Rockfill Embankment

The strength and settlement characteristics of the available materials to be used in the rockfill determined the side slopes and construction processing performance of the fill embankment.

The total of approximately eight million cubic meters of fill required for the construction of the airport platform was sourced within the Airport Development Area (ADA). Most of this material used to fill the Dry Gut, which was a steep-sided valley at the southern end of the runway platform. The Dry Gut fill, to a maximum of about 100 m in depth, extends beneath the Runway End Safety Area (RESA) where post-construction settlement needed to be minimised.

Only rock that required blasting to excavate was used for the fill as no other material has been characterised and proven suitable by testing. Rippable material that did not require blasting to excavate was used to fill areas where settlement considerations were not critical.

Design Criteria

The design criteria for the Dry Gut fill as per the Employer's Requirements was specified as follows:
- Earthworks to comply with OTAR Part139
- Allowable tolerance for concrete runway surface 6 mm in 3 m straight edge
- 120 year design life for the earthworks structure.

As the works were considered similar to the construction of a rockfill dam, it was proposed that the philosophy defined in the following publications should be adopted for the design and construction of the Dry Gut fill:
- The Rock Manual: The use of rock in hydraulic engineering, by CIRIA; CUR; CETMEF (2nd edition, 2007), published by CIRIA, London, and
- The US Corps of Engineers approach for dams, which defines 1.5 as an allowable factor of safety (FOS) for slope stability analysis.

In the first Section – use of rock, the following is stated:

"Standard solutions do not generally exist in this field of engineering. To develop a robust, site-specific rock-based solution for a project it is necessary to consider a wide range of issues including materials, environmental conditions, construction methodology, maintenance regime (..)"

The same principle as above applies to rockfill dams. In rockfill structures the aim is to compact the material to form a dense matrix and

maximise settlement during compaction as well as the interlock between large hard rock particles. In line with rockfill dam construction methodology, it was proposed to use a construction method based specification. This method has since been refined following the results of field tests during the construction process and following extensive trials on site particularly during the early stages of the fill construction. Settlement of the fill matrix has been continuously monitored once the fill depth exceeded about 20 m. The monitored settlement is to be taken into consideration when determining the final construction levels to accommodate the projected settlement and hence ensure that the upper surface of the fill remains within the prescribed tolerances throughout its design life.

Site Investigations

Field Survey Controls
The control system used for the survey was based on the St Helena Local Co-ordinate system and consists of 18 Beacons covering the footprint of the Airport site and Dry Gut and set up using post process static, Post Processed Kinematic GPS data (PPK), GPS survey methods.

Geotechnical Site Investigations
Geotechnical investigations were conducted on Prosperous Plain and the Dry Gut for mapping of exploratory drilling, trial pits, borehole cores, discontinuity surveys and results of the analysis of the borehole core logs and laboratory data for the proposed airfield earthworks.

In general the broad material types were mapped with the following observations of the Dry Gut basin and valley sides:
• surface soil layer comprising relatively thin layers of transported soil (hillwash) and residual lava
• weathered tuff (volcanic ash) formed distinct sub-horizontal layers within the lava rock. These could be seen along the steep slopes, where the rock exposure were "layered" by these bands of softer material
• pockets of weathered sedimentary rock or sedimentary soils within the volcanic rocks. This material was further tested to determine whether suitable for use in the bulk fill or discarded as spoil
• lava rock weathered to different degrees, ranging from highly weathered to unweathered.

Construction Water
At the initial stages of the contract extensive groundwater investigations were conducted in the vicinity of Prosperous Bay Plain to determine the prospects of providing the ultimate construction water requirements from deep boreholes. 18 No. boreholes were drilled in the vicinity of the airfield and surrounding areas with pump tests indicating positive groundwater yields for construction water requirements.

Seismic Hazard Analysis
Due to the limited number of known (recorded) seismic events in the area, it was not possible to perform the usual probabilistic analysis and consequently a deterministic seismic hazard analysis was performed by Prof. Andrzes Kilko for the project requirements.

The outcome of the analysis indicated that the island has a low seismic hazard potential and the predicted mean Peak Ground Acceleration (PGA) is 0.021 g and the upper limit (maximum) PGA is 0.05 g.

Analysis of Source Material
• It was estimated that on average approximately 60% of the material that was to be excavated, would be very hard with UCS of 100 MPa and may not break down during processing of the fill materials. This material consisted mainly of Trachyandesite (average specific gravity of 2.6).
• On average, approximately 40% of the excavated material consisted of soft or decomposed rock, i.e. rock with a UCS of less than 100 MPa which would probably break down during processing.

• The natural fissure pattern in the rock was expected to result in approximately 80% of both hard and soft material to have a nominal diameter smaller than 400 mm.
• No significant post-construction settlement was expected, although it was considered essential to monitor any creep settlement for a period of approximately 6 months prior to proceeding with the runway surfacing over the deep Dry Gut fill zone.

Materials Performance and Method of Construction
• Prior to bulk placing of rockfill, a trial embankment test section was constructed, using different combinations of material types, sourced from the cut zones. The water quantity added was varied and different compaction efforts with the equipment available verifying the best placing methodology.
• It was proposed to place the material in relatively thin layers with basically two embankment zones, viz. an inner zone directly under the runway and an outer zone forming the outer embankment slopes. A third zone was also proposed between the natural valley slopes and the fill embankment and at the bottom of the Dry Gut valley as a drainage zone. The minimum fill layer thickness was determined by the maximum material particle size allowed, with the maximum particle size not exceeding two-thirds of the layer thickness.
• Mixing and blending of the rock material was achieved during the normal excavation and placing process. The composition of the source material was expected to vary significantly and was tested in the field at various source locations. A 60% of harder trachyandesite rock to 40% softer basalt rock was considered desirable.
• A wet fill construction process was proposed using between 80ℓ/m³ to 100ℓ/m³ as the anticipated water application to achieve the lowest void ratio during compaction, which would minimize settlement. The processing of the fill and water application was best determined during the trial embankment tests to determine the optimum compactive effort.
• A 20 tonne smooth drum vibrating roller was used for compaction with 10 roller passes being the optimum number to achieve the required compaction to compact an 800 mm layer to at least 80% relative density as a target density for all zones of the embankment.
• Placing took place over a wide front to facilitate a high production rate – approximately 15 000 m³ to 20 000 m³ per day utilizing a double shift 24 hour day production strategy.
• Selection and temporary stockpiling of the harder rock was required in order to have sufficient volume of this material for placing in the outer sloped embankment areas. The softer basalt rock was selected for mixing and placing in the central embankment zone. Refer to section below for the proposed material type classifications as shown on Drawing No. WPG-720-CI-0003-01-Rev.2.

Trial Embankment Construction and Testing
On establishment of the construction plant on the airfield site an initial trial embankment (ITE) was constructed in the bottom of the Dry Gut and tested extensively to determine the degree of compaction that could be attained using various construction procedures for the bulk fill. Consideration was given to the influence on compaction of layer thickness, addition of water into the rockfill and number of passes of the 20 tonne smooth-drum vibrating rollers.

The values adopted for each of the three variables, to cover the range of expected optimal values, were:
• Layer thicknesses before compaction: 600 mm, 800 mm and 1 000 mm
• Water added per cubic metre before compaction: 40 ℓ, 80 ℓ, 120 ℓ.
• Number of vibrating compaction passes: 4, 6, 8 and 10 passes
Surface levels of the ITE were accurately recorded at various stages of placement and compaction of each layer to determine the changes in surface level and assess effectiveness of the compaction.

Plate load tests were conducted on the upper surface of representative compacted layers and large diameter water-replacement density tests were conducted in the upper layer of the completed embankment. The purpose of the plate load tests was to assess their potential application for Quality Assurance purposes. The density tests are an important part of the ITE investigation as they obtain actual parameters applicable to the rockfill materials and their behavior.

Refer to photographs 10 to 15 included in ANNEXURE A for recordings of the ITE test procedures. The test results indicated the most effective construction application for the material sourced at that time, and considering an 800 mm thick layer compacted by 10 roller passes after 80 ℓ/m³ of water being added.

Grading of Rockfill Materials

On consideration of the materials available it was recommended that only three different categories of material be used for the construction of the rockfill embankment, viz. Material 1 in the bulk of the embankment, Material 2 on the exposed embankment slopes for added slope stability and protection against the elements and Material 3 as the drainage layer in the Dry Gut channel and on the contact between the valley sides and the rockfill embankment to provide the lowest resistance against flow should water enter the fill – refer to drawing WPG-720-CI-0003-01-Rev2.

All materials were to be targeted with a fines content of less that 5%, however dependent upon where the material was to be placed in the construction profile it was considered that material with a fines content no greater than 15% would still be suitable for use as general fill, as long as point-to-point contact would still be achieved for particles larger that 50 mm.

The target grading for all materials were to be "well graded" with respect to gravel content - where Coefficient of Uniformity (Cu) > 4, and Coefficient of Curvature (Cz) was between 1 and 3 for a well graded material. However, it was considered that "poorly graded" material would also be acceptable as long as the general grading was within the envisaged ranges.

Material 1 – the bulk of the rockfill embankment and composed of a blend of the uncontrolled blasted hard rock and the softer rock materials that were excavated – "well graded" material.

Material 2 – consisted of the hard rock material only – "well graded" material. A suitable volume of this type of material was stockpiled separately to ensure the required volume was available for the construction of the outer section of the bench slopes. Material 2 was placed in a 4 m to 5 m thick zone on the outer surface of the bench slopes from the bottom

to the top of the rockfill embankment. The target grading of Material 2 was such that 15% of particle sizes less than 19 mm were excluded and that point-to-point contact was retained for particle sizes greater than 50 mm.

Material 3 - the drainage layer which was required in the base of the Dry Gut channel and as a drainage interface layer to continue up the valley sides – "poorly graded" material.

A graphical display of the proposed grading limits for the rockfill material is shown in the figure below.

Rockfill Embankment Slope Stability

A detailed analyses was performed to evaluate the possibility of effecting cost savings by steepening the embankment slopes. The US Corps of Engineers approach for dams was used, which defines 1.5 as an allowable FOS for embankment slope analysis.

Without the availability of conclusive laboratory tests at the initial stages of the project to guide the selection of shear strength parameters to use in the slope stability analysis, empirical methods were used to derive acceptable shear strength parameters. Research has shown that the following relationship can be used to determine the shear strength parameter φ (with c = 0 kPa) for rockfill, gravel and sand:

$$\varphi = \varphi_0 - \Delta \varphi \log \left(\frac{\sigma_3}{1 \text{ atmosphere}} \right), \text{ where:}$$

φ : the friction angle
φ_0 : the friction angle at 1 atmosphere pressure (101.3 kPa)
$\Delta \varphi$: the correction for confining pressure variation
σ_3 : confining pressure in the fill

From several laboratory test results on materials from various dams the typical φ_0 and $\Delta \varphi$ ranges can be determined as summarised in the table below:

Material	Relative density (%)	φ_0 (°)	$\Delta \varphi$ (°)
Rockfill	100	55	10
	50	45	8
Gravel	100	51	8
	50	41	3

For the case of high internal confining pressures (say fill height > 50 m) it can be shown that φ may vary between 41° and 45° for rockfill at between 50% and 100% relative density, while the corresponding values for gravel may vary between 40° and 43°. Normally the compaction of rockfill should be at least between 75% and 80% of the bulk relative density. The composition of the rockfill that was used to construct the Dry Gut embankment contained a substantial amount of gravel content, hence φ = 42° was proposed as an acceptable shear strength parameter for deeper seated failures. A bulk rockfill density of 2 150 kg/m³ was used.

However, where c = 0 kPa in slope stability analysis, shallow slope failures with low factors of safety are the most critical, as was the case for the bench slopes selected. In this case the confining pressures were much lower and for the rockfill φ may vary between 47° and 52° (44° and 48° for gravel). It was therefore
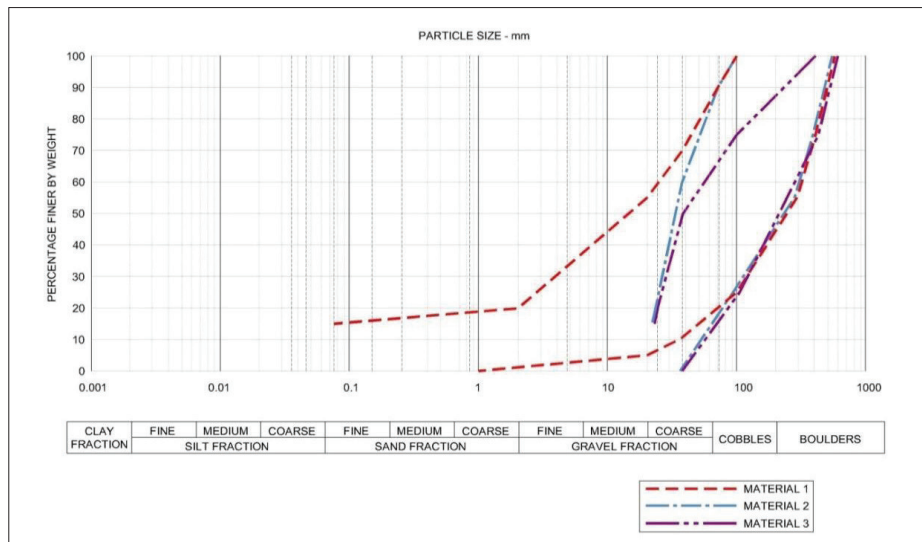
proposed that φ = 46° to φ = 49° should be used in such analysis and still with the values and ranges as recorded in the table above.

The proposed rockfill embankment slope geometry was as follows:
• benches 10 m high at a slope of 1:1.36 with a 4 m wide horizontal surface between benches, which equates to an average (relative) slope of 1:1.76
• maximum embankment height 110 m.

The first phase of the slope stability evaluation revolved around the stability of the individual bench slopes (Material 2), for which the following parameters were used:
• Bulk rockfill density   2 150 kg/m^3
• Shear strength   φ = 46° to 49°, c = 0 kPa
• Bench slope   1:1.36
• Bench height   10 m to (15 m provisional)
• Allowable min. Factor of Safety (FS)   1.5

The results of this slope stability evaluation are summarised in Table 1 below.

TABLE 1 Stability of Bench Slope

Bench Height (m)	Friction Angle (φ°)	Factor of Safety	Failure Surface Distance from Edge (m)
10	46	1.46	0.5
10	46	1.49	1.0
10	47	1.50	0.5
10	49	1.61	0.5
10	49	1.65	1.0
11	46	1.45	0.5
11	47	1.50	0.5
11	49	1061	0.5
12	46	1.45	0.5
12	47	1.50	0.5
12	49	1.61	0.5
13	46	1.44	0.5
13	47	1.49	0.5
13	49	1.60	0.5
15	46	1.44	0.5
15	47	1.49	0.5
15	49	1.60	0.5

It follows from these results that it would be possible to increase the bench height to at least 12 m and still maintain a FS of 1.50 at φ = 47°. A bench height of 15 m may even be acceptable.

The second phase of the slope stability evaluation concerned the stability of the overall slope (Material 1), for which the following parameters were used:
• Bulk rockfill density   2 150 kg/m^3
• Shear strength   φ = 46° to 49°, 0 = 50 kPa
• Overall slope   1:1.36 (for 10 m bench) to 1:1.627 (for 15 m bench)
• Bench height   10 m to (15 m provisional)
• Alloable min. FS   1.5

Stability table not recorded in this paper buy it followed that an overall slope of 1:1.76 (for a 10 m bench height) and φ = 40° to 42°, c = 0 kPa, the rockfill embankment slope would be stable (FS>1.5) for all the cases that were evaluated. For an overall slope of 1:1.627 (for a 15 m bench height) the embankment slope would also be stable (FS > 1.5) for all the cases analysed, although a cohesion (c) of a minimum of 10 kPa was required in some cases.

The third phase of the slope stability evaluation considered the stability of the overall embankment slope (Material 3) without the Reinforced Earth system. The results of this slope stability evaluation are summarised in Table 2, with the same parameters as indicated above.

TABLE 2 Stability of Overall Embankment Slope Without Reinforced Earth

Slope Height (m)	Reinforced Earth (m)	Slope	Friction Angle (φ)(°)	Cohesion (c) (kPa)	Factor of Safety	Failure Surf. Dist. from Edge (m)
110	0	1:1.76	40	0	1.50	4
110	0	1:1.76	40	10	1.63	4
110	0	1:1.76	42	0	1.60	4
80	0	1:1.76	40	0	1.51	4
80	0	1:1.76	42	0	1.62	4
50	0	1:1.76	40	0	1.52	4
50	0	1:1.76	42	0	1.64	4
30	0	1:1.76	40	0	1.54	4
30	0	1:1.76	42	0	1.66	4
20	0	1:1.76	40	0	1.58	4
20	0	1:1.76	42	0	1.69	4

From the outcome presented in the Deterministic Seismic Hazard Analysis indications were that the island has a low seismic hazard potential and the predicted mean Peak Ground Acceleration (PGA) is 0.021 g and the upper limit (maximum) PGA is 0.05 g.

Normally (from a probabilistic analysis) an Operating Base Earthquake (OBE) with 144 years recurrence interval, a Maximum Design Earthquake (MDE) with 475 years recurrence interval and Maximum Credible Earthquake (MCE) with 10 000 years recurrence interval will be defined to obtain the PGA for design. However with a deterministic analysis it is not possible to define these earthquakes. The 0.05 g was therefore proposed as the absolute maximum PGA to be expected and should therefore be taken as the MCE, while the 0.021 g was taken as the MDE or the designs of the airport buildings.

In terms of dam designs the safety is evaluated to achieve a FS of at least 1.5 for application of the MDE, while the MCE is defined as an extreme event for which a lower FS can be accepted (1.2). Due to the low value of 0.05 g, the dam may only be designed for a FS >1.5 at 0.05 g, without giving attention to the MCE.

For the rockfill slopes some stability calculations were performed for both these values. For the 10 m high bench slope the following became clear with a pseudo static analysis:

Slope	φ	FS		Failure Surf. Dist. from Edge (m)	FS	Slope	FS	
		0.021 g	0.05 g		0 g		0.021 g	0.05 g
1:1.36	46	1.38	1.30	0.5	1.46	1:1.5	1.52	1.43
1:1.36	46	1.42	1.34	1.0	1.49	1:1.5	1.56	1.46
1:1.36	47	1.43	1.35	0.5	1.50	1:1.5	1.57	1.48
1:1.36	49	1.54	1.45	0.5	1.61	1:1.5	1.69	1.59
1:1.36	49	1.58	1.49	1.0	1.65			

In the above table the φ = 46° applies to a rockfill with high gravel content, while φ = 47° to 49° applies to rockfill with low gravel content (hence the use of low gravel content material on the exposed embankment slopes). The implications were therefore very simple; by ensuring the use of low gravel content rockfill on the outer slope surfaces, the FS = 1.43 to 1.58 which is acceptable (0.021 g), and for an extreme event (MCE with 0.05 g) FS = 1.35 to 1.49 which is also acceptable. Material selection was therefore most essential to ensure low gravel content rockfill was used on the exposed embankment slopes (Material 2 – reference table 1 above) otherwise the slope was to have been flattened to 1:1.5 for use of the high gravel content materials.

For the overall slope at 1:1.76 and 0.021 g a similar condition was clear, ie. for the 110 m, 80 m, 50 m, and 30 m heights the FS = 1.43 to 1.47 for φ = 40° (gravel), but for φ = 42° (low gravel) FS = 1.53 to 1.58, where the latter was acceptable. For 0.05 g the FS is lower but acceptable.

The lower FOS results were for the areas closer to the edges of the fill embankment where according to our interpretation the live loads are not applicable. Only working loads were used, because ultimate state design was not required. Further to this a surcharge load of 10 kN/sq m was applied at the surface.

The shear strength parameters of the material excavated were retested once exposed in the initial stages of the excavations to verify the above mentioned analysis. The following tests and observations were conducted on the initial trial embankment procedure and repeated for any change in material composition to verify the material characteristics and performance (initially at a frequency of approximately 50 000m³ intervals, and once a level of consistency was achieved every 100 000m³ during the construction progression:

• Plate load tests
• Large volume density tests
• Grading analysis
• Compaction effort against settlement measurements
• During construction assessments of settlement and any movements to the external embankment terraces were physically monitored after filling to platform level RL 220 and reported at regular intervals thereafter.

Settlement
It is very difficult to quantitatively predict the settlement of a rockfill embankment and therefore the experience gained at different rockfill dams in Southern Africa, was considered together with international experience documented on concrete faced rockfill dams (CFRD). The similarity with a CFRD is that the rockfill of the Dry Gut fill will be substantially dry for the lifetime of the structure. Rockfill dams are normally designed for a lifespan of at least 120 years and these structures are designed and constructed for minimum post-construction settlement.

The embankment settlement is being determined by regular assessment of installed settlement monitoring equipment and geodetic survey measurements during the construction phase. This data together with numerical analysis provide confidence in the construction methods used and in determining the final construction levels to accommodate the projected settlement and hence ensure that the upper surface of the fill remains within the prescribed tolerances throughout its design life.

Monitoring of Rockfill Settlement and Compaction Control
In order to accurately assess the rate of settlement of the rockfill a series of tests are being conducted during the construction with regular monitoring of stage lift maximum layerworks depths. Wash-out trial tests are also conducted at various stages to monitor the optimum water demand required to achieve interlocking of the rock fragments.

Samples of the blasted rock materials are regularly tested for strength and durability to ensure that the best quality rock is selected for construction of the exposed embankment faces. Other tests for quality control records included mass density tests, plate bearing tests, water absorption and porosity, Atterberg limits, UCS, point load tests, pH and water soluble Sulphates to confirm that there is no potential for degradation of the source materials. A fully equipped materials laboratory was established on site to accommodate calibration of equipment and the standard method of testing of all in-situ materials and the monitoring of construction workmanship and materials performance requirements, all in accordance with the specifications of UK Highways Works, Series 600 (631 and 644) together with BS 1377-Part 1 and US Corps of Engineers EM 1110-2-2302. Testing frequencies for process control are conducted in accordance with the following specifications:

• Particle size distribution: dumped prior to compaction and compacted: 1 test per 50 000 m³ material placed.
• Bulk density and relative density of compacted material: 1 test per 100 000 m³ material placed.

Reference was made to the following publications:
• Part 5 – Large Scale Density and Gradation Tests in Compacted Rockfills, AB Engineering Inc., Littleton, Colorado, USA
• Rockfill Embankment Trials.

Stormwater Drainage
Due to the difficulty of constructing drainage collection channels at the interface of the toe of the fill with the natural rock valley sides consideration was made for directing all stormwater runoff from the airfield footprint away from the fill matrix and channel the outlets to convenient locations along natural contours and into the neighbouring water courses.

Also of note was that an open channel drain was excavated into the southern face of the Dry Gut redirecting the stormwater flows from the Dry Gut catchment into the neighbouring valley, thus resulting in the omission of the previously proposed Dry Gut culvert and attenuation dam.

Balance of Earthworks, Design Drawings and Volume Calculations
Taking cognizance of the design criteria, the anticipated source material characteristics and specified construction process, Detail Design drawings of the Airfield Earthworks were developed. The surfaces of the embankment profiles and volume calculations were generated from Model Maker TOT files of the site survey data. In view of the initial uncertainty of the ultimate performance of the source materials a sensitivity analysis was computed using various combinations of the design criteria.

In view of the critical path programme, fixing the final vertical alignment of the runway was considered vital considering the lead time required in setting up the procedures of the necessary flight path sensitivity analysis and the follow-on requirements of developing early submissions to the Regulator for final approvals. This process has presently been programmed on the basis of our final design alignment configuration and balance of earthworks. As such any later adjustment to this alignment will have serious time delay consequences to the Construction programme.

The earthworks volume sensitivity analysis clearly showed that the slightest variation in the material performance had a marked effect on the material balance.

Refer to Annexure B for Detail Design drawings of the Airfield Bulk Earthworks, Runway Longsection and Cross Sections.

Airside/ Aviation
The airport design is challenging and requiring the following innovative design considerations:
• Remoteness and the distance to the island airport requires:
  - innovative design to ensure that an aircraft can safely land with sufficient fuel to return to the originating airport.
  - careful consideration of selection of navigation aids and operating procedures to ensure that the likelihood of an aborted landing is contained within an absolute minimum landing space.
• Low number of aircraft movements due to island's population size. This presents challenges in:
  - Cost effective design to ensure that only necessary infrastructure is provided.
  - Pavement Quality Concrete runway to consider increased flexural strength with possible reduction in pavement thickness.
  - Managing operation costs.
  - Maintaining technical and operational expertise, since the work force may largely be part time.
• The terrain on the island makes positioning of the airport runway difficult, especially to keep costs and environmental impact in bounds. The trade-offs include:
  - Runway position and length (e.g. using displaced thresholds, shortened approach lights, etc.).
  - Orientation of runway (to make use of prevailing wind).

- Obstacles forcing unconventional angled (offset) aircraft approaches.
• The design and installation of all Airport Ground Lighting, Navigational Aids and Airport Traffic Control Equipment.

Airport Buildings

The design of the Airport Terminal Buildings and their setting in the landscape attempts to capture and convey the unique spirit of the island in built form. The new buildings have been designed with the following considerations:
• Proportions, design principles and materials used in the rich heritage of historical building styles mainly from the Georgian, Regency and Victorian periods found on the island.
• Stone ramparts, canons, thick battered walls and gun embrasures harking back to the military history of the island.
• The red, blue, grey and black volcanic oxides on Prosperous Bay Plain has been considered to pigment precast & off-shutter concrete and plaster. The use of natural materials from the site will anchor the Terminal Buildings within their context. Volcanic ash, pumice, oxides and rock formations form the very fabric of the island.
• Enclosed garden courtyards consist of lichens and endangered plants on either side of the main entrance. Existing rocks with coloured lichens are to be harvested from the site prior to building commencement to be used within the proposed garden courtyards.
• Local artwork will be featured in the Terminal Buildings in order to capture the spirit of the people of St. Helena and create a feeling of inclusivity amongst the Saints. These will consist of wall murals and specific commissions from local artists.
• Sustainable green building practices have been incorporated into the buildings. These include rainwater and grey-water harvesting in tanks buried below the garden courtyards; solar panels mounted on the roofs for water heating; passive energy principles including solar shading via roof overhangs and opening window sections as well as maximum daylight penetration via roof skylights.

CONSTRUCTION CHALLENGES

The biggest challenge in constructing the St Helena Airport has been creating and maintaining an efficient planning and logistics chain. There was no major construction plant or building materials on the island and virtually everything having to be shipped to the island. The Main Contractor has chartered a 2 500 tonne ocean-going vessel for the duration of the contract to accommodate their plant and materials supply requirements. The Contractor has set up a consolidation area in Walvis Bay from where the NP-Glory-4 sails to St Helena. It takes five days to reach the island and seven to return, owing to currents and winds, with a single shipping cycle taking up to 21 days to complete.

With no harbour on the island a temporary Jetty had to be constructed to accommodate the roll-on-roll-off facilities for the NP-Glory-4. Other early works establishments consisted of a temporary fuel facility (1,5 million litres), construction of the 14,5 km haul road over very harsh rocky and steep terrain, staff accommodation for 120 persons, borehole explorations in sourcing adequate groundwater for construction water and the construction of temporary water storage dams (4 x 2 million litre HDPE lined facilities), workshops, the establishment of a fully equipped, internationally accredited laboratory, and the erection of crushing and concrete batching plants. The social interaction and integration with the local Saints community was an issue that was looked at seriously before embarking on the contract with the Main Contractor employing about 270 Saints of their current 390 on-island work force. At the peak of production the work force will increase to about 450 persons.

Apart from the varying geographical features of the island, the airport site presents unique challenges owing to the setting and history of St Helena. There have been protected slave burial sites and archaeological finds to contend with, apart from the site being close to the breeding area of the Wirebird, which is indigenous to St Helena. It is also in close proximity to around 40 species of invertebrates that can only be found on the island. Once completed, the challenge remains for the Contractor and their subcontractors to clean up and remove all hazardous waste generated by the project, leaving the island as they found it when they arrived.

The risk awareness of the project execution is absolutely crucial for the success of the project, and both the St Helena Government and the project team run and share a comprehensive risk and opportunity register all in compliance with legislated CDM statutory regulations. This is vital to identify and mitigate any risks to the Health and Safety of personnel prior to commencement of any sector of the works. This register is updated regularly by the St Helena Government and the Contractor's project team in an effort to ensure that Health and Safety of personnel and the special features of the island are protected and that the ultimate goal of completing the contract on time and within budget is achieved.

Annexure A1
Initial Plant Establishment

PHOTO 1 Rupert's – NP Glory and Temporary Jetty

PHOTO 2 Rupert's – Temporary Fuel Facility (1.5 m litres)

PHOTO 3 Rupert's Bulk Fuel Facility

PHOTO 4 Bulk Fuel Facility – 6 million litres

PHOTO 5 Airport Access Road (14.5 km)

PHOTO 6 Airport Access Road

Annexure A2
Initial Trial Embankment (ITE)

PHOTO 7 Dry Gut Fill, Preparing Platform for ITE

PHOTO 8 Confirmation of Plate Load Test on ITE

PHOTO 9 Preparing Sample for Density Test on ITE

PHOTO 10 Sample at Laboratory after Grading

PHOTO 11 Start of Washout Test

PHOTO 12 Rock Particle Interlock

Annexure A3
General Production

PHOTO 13 Aerial View of Drilling Operation

PHOTO 14 Dry Gut – Start of 8 million m³ Rockfill

PHOTO 15 Panoramic View – Southern Approach at 300.72 m msl

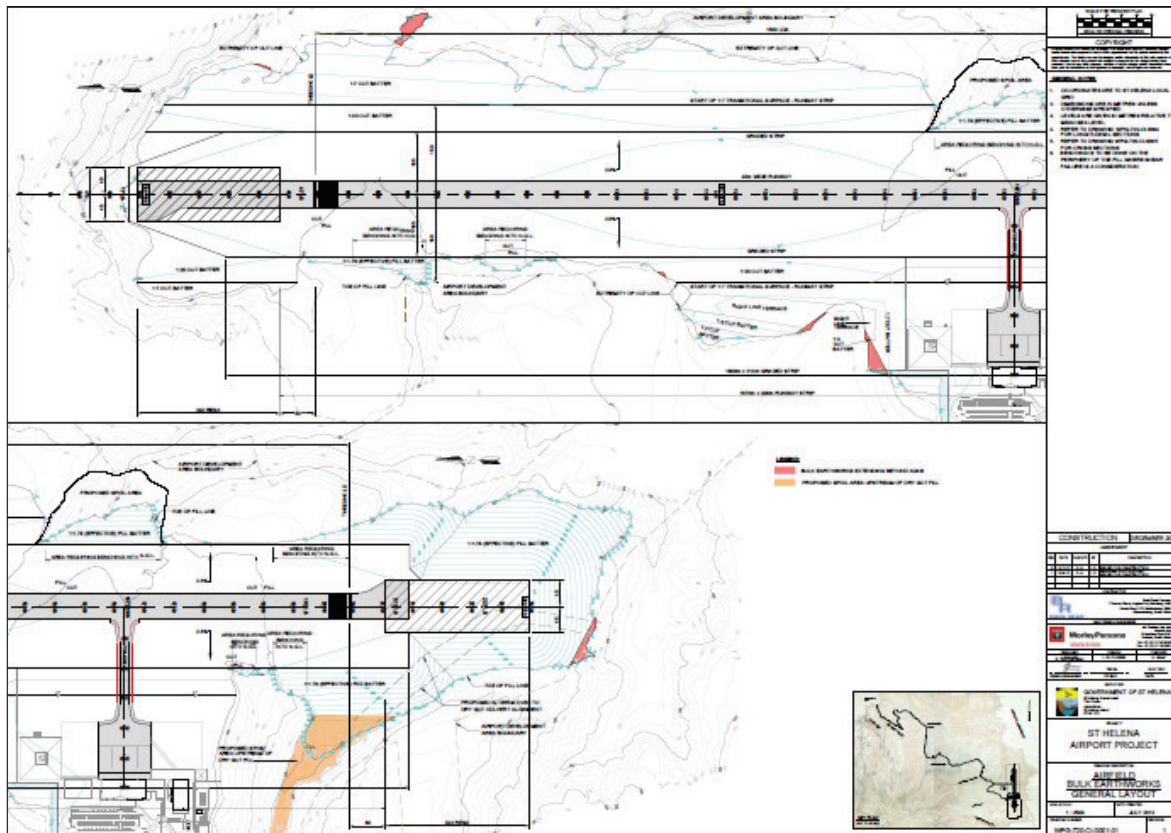PHOTO 16 10 m Terraced Dry Gut Rockfill – Total height 102 m
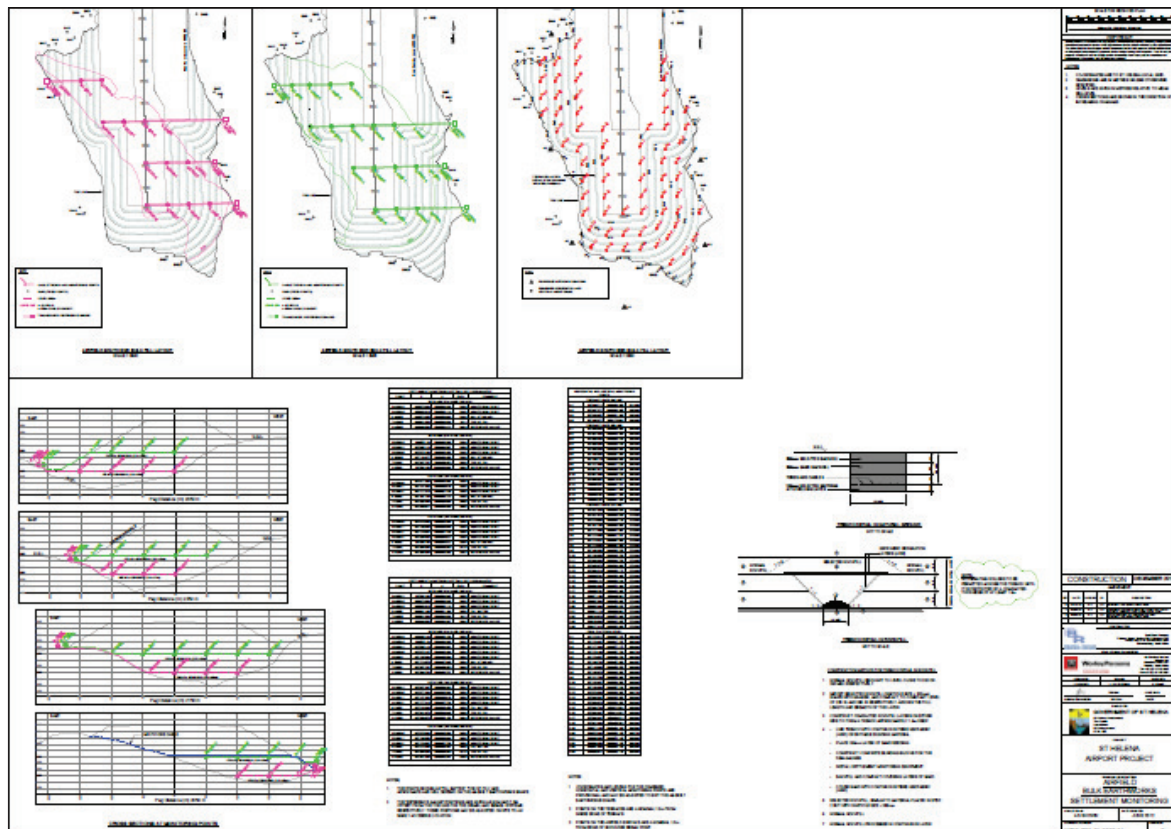

PHOTO 17 Airport Terminal and Air Control Buildings
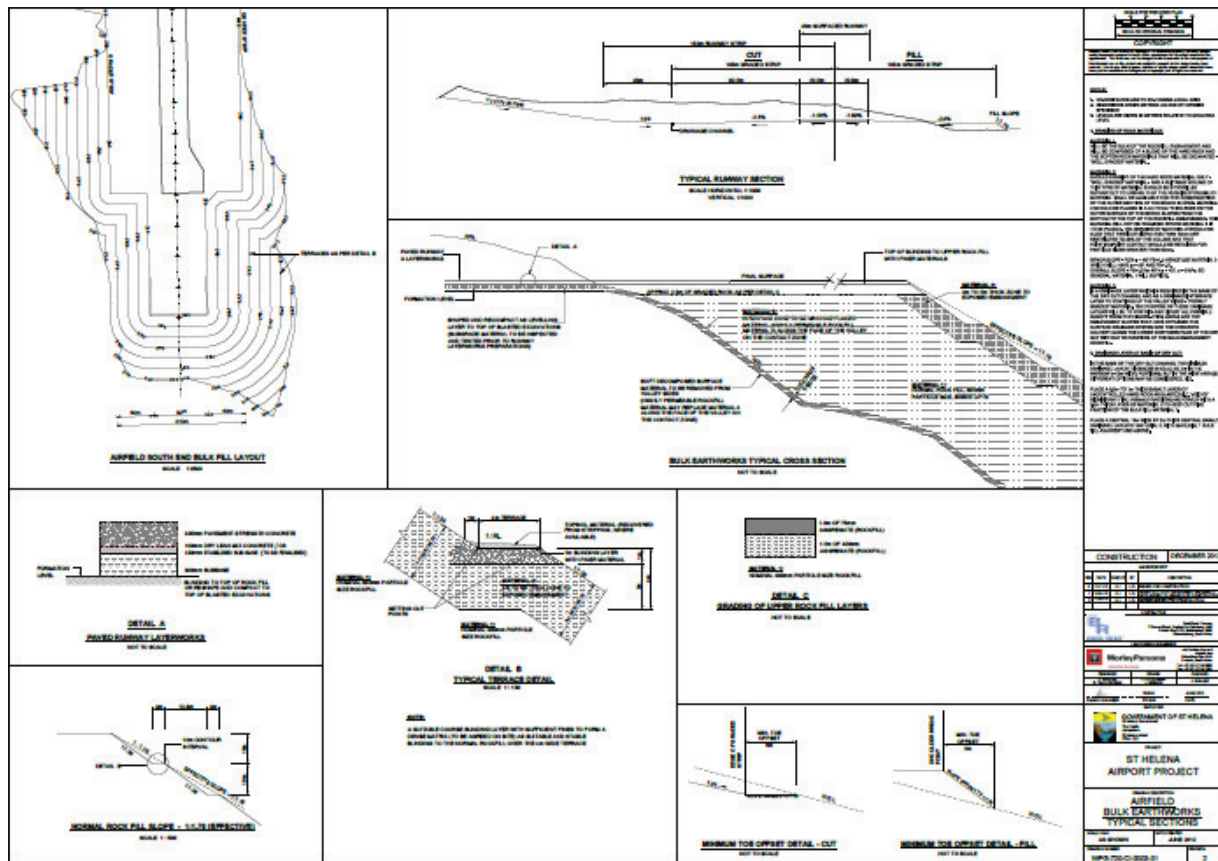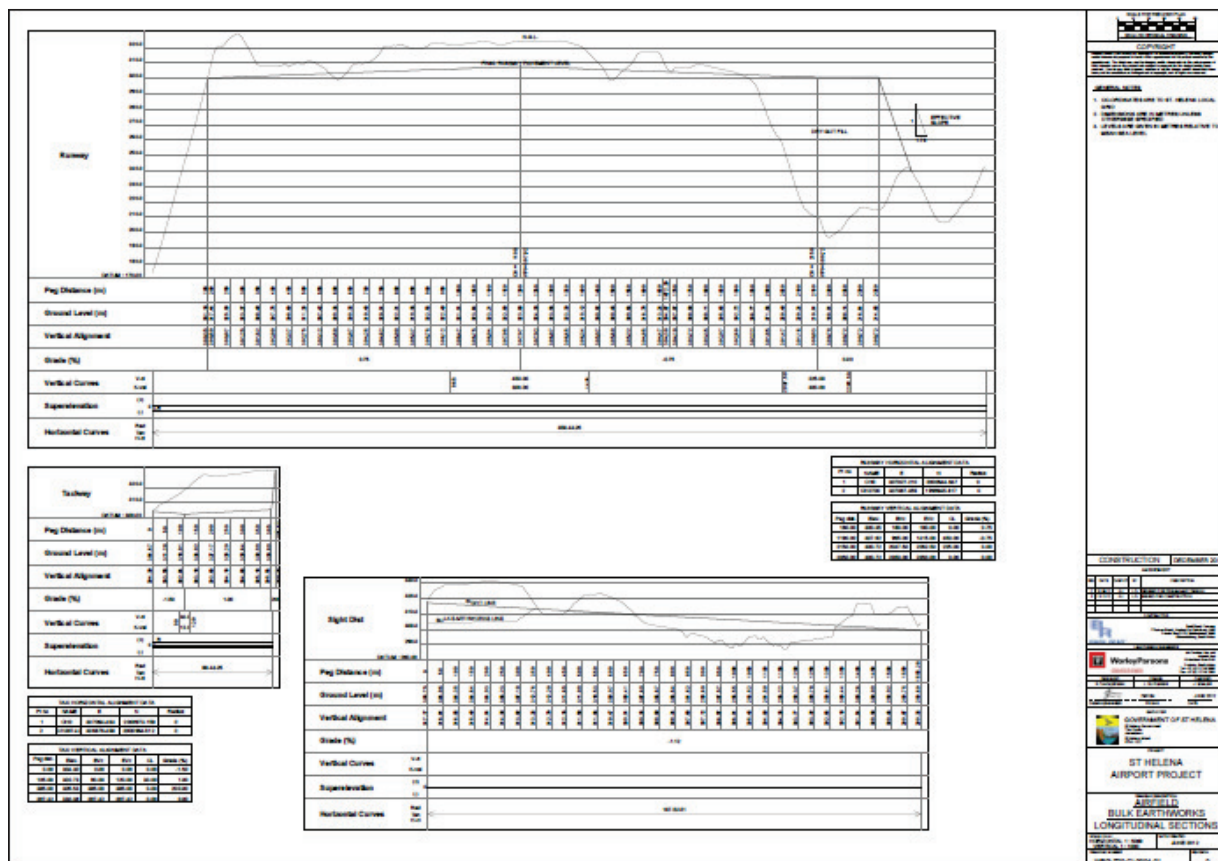

PHOTO 18 Airport Air Control building

Annexure B

WPG-720-CI-0001-01-REV 1 Airfield Bulk Earthworks General Layout

WPG-720-CI-0002-01-REV 2 Airfield Bulk Earthworks Settlement Monitoring

WPG-720-CI-0003-01-REV 2 Airfield Bulk Earthworks Typical Sections

WPG-720-CI-0004-01-REV 2 Airfield Bulk Earthworks Longitudinal Sections