DESIGN AND CONSTRUCTION OF A GEOFOAM EMBANKMENT OVER SOFT GROUND

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ABSTRACT

Development of the new Service Centre required construction of an off-ramp from the existing road together with an internal road network. Together these will provide access to the Service Centre for traffic from the existing roads. The off-ramp and road network were constructed on fill embankments, up to 4m in height, overlying a relatively uniform but very significant thickness of soft clay. In addition, the project site had been subject to a complex history of loading and unloading as a result of previous development on the site and ground improvements adjacent to the site, as part of new Service Centre development. To mitigate the effects of post-construction settlement to acceptable levels, safe and cost-effective ground treatment measures were developed along the preferred road alignment.

This paper provides some background to the project and presents in detail the design and construction methodology adopted for the lightweight fill treatment works using geofoam. The new geofoam embankment had to be constructed within the vicinity of existing and new infrastructures, i.e. pile-supported bridge, existing roads and a new drainage structure.

1 INTRODUCTION

The new Service Centre development, referred to as Banksia Place Stage 1A project, near Brisbane Airport required construction of roadway embankments on soft ground so as to provide an access to the new Service Centre for traffic from the existing airport access roads, i.e. Moreton Drive and Nancy Bird Way. The foundation soils encountered beneath the site comprised very soft and loose alluvial deposits of very significant thickness up to approximately 20m. The site had experienced a complex history of loading and unloading, as a result of the previous extensive upgrade of the Brisbane Airport's road network and ground improvement which was undertaken as part of Banksia Stage 1A project. This improvement included 6 months of preloading with 3.5m surcharge and wick drains. Additionally, the new off ramp from the existing road had to be constructed within close proximity of the existing and new infrastructures which included an existing bridge supported on piles, the existing Brisbane Airport's road network, and drainage structures.

Due to the imposed geotechnical and physical constraints on the project site, construction of the embankments in these areas involved significant risks associated with embankment stability, post construction settlement and potential impact on the existing infrastructures which had tight tolerance in terms of the additional allowable lateral and vertical movements induced by new bulk earthworks.

To develop safe and cost-effective ground treatment measures, the preferred road alignment was divided into discrete zones with similar characteristics including proposed fill height and physical/imposed constraints. Different types of ground treatment method were proposed for each discrete zone in order to mitigate the effects of post-construction settlement to acceptable levels.

This paper outlines some background to the project and presents the design and construction methodology adopted for the lightweight fill treatment works using Geofoam (also known as Expanded Polystyrene, EPS). The Geofoam embankment was constructed on the east corner of Moreton Drive and Nancy Bird Way intersection and had to be constructed within 10m of the existing twin bridge. Figure 1 below shows the project location.

2 BACKGROUND

Brisbane Airport is located approximately 14km north-east of the Brisbane central business district (CBD) and is one of the most important industrial and business areas in the Australia Trade Coast precinct in the state of Queensland. The airport area has therefore been experiencing continuously rapid growth and significant development over the past years.

Together with the State Government's Gateway Upgrade Project that provided improved connectivity to the Australia Trade Coast precinct, an extensive upgrade of the road network around the Brisbane Airport was carried out to alleviate traffic congestion around the airport and to provide much-needed infrastructure for the future growth. A second access roadway to the terminals, that included Moreton Drive linking the Gateway Motorway with the Domestic Terminal and Nancy Bird Way serving the International Terminal, was constructed in 2009.

Since the opening of the new Moreton Drive together with its associated secondary roads, a considerable portion of the overall traffic travelling to and from the terminals has moved onto the new roads. In addition, the construction of the new Brisbane Airport’s roads provided an opportunity to provide a new Service Centre within the Banksia Place development precinct on the Eastern corner of the Moreton Drive and Nancy Bird Way intersection roundabout. The new Service Centre development consists of several buildings and restaurants in a central Service Centre, a canopied refuelling area, a carwash and servicing building, and associated driveways and parking areas. Access to the Service Centre is provided by a new off ramp from the Moreton Drive and internal roads to the Nancy Bird Way.

Snowy Mountains Engineering Corporation (SMEC) were commissioned by the Brisbane Airport Corporation Pty Ltd (BAC) to provide design service to facilitate construction of an off-ramp from Moreton Drive and internal road network for the Banksia Place Stage 1A Service Centre precinct.

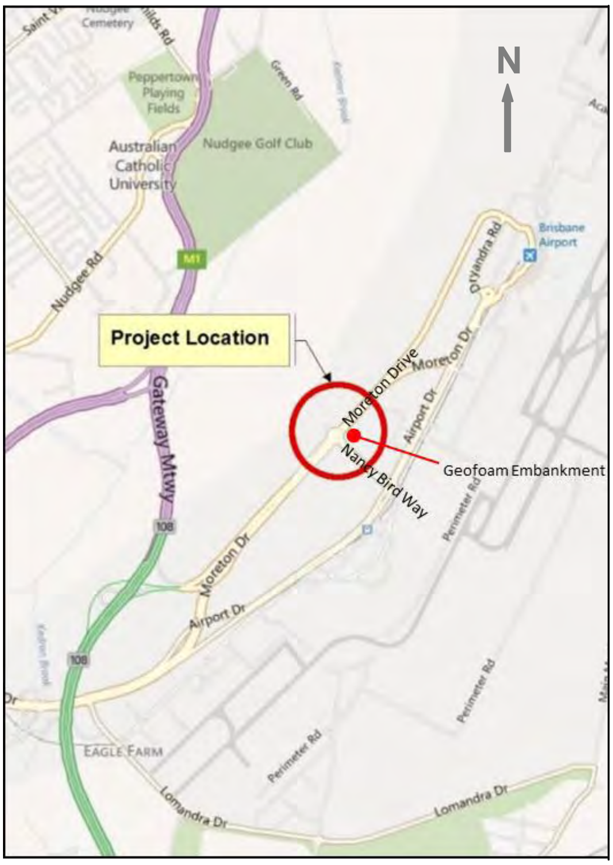

Figure 1: Project location.

As part of the Stage 1A Service Centre development, the site had been undergoing a bulk earthworks strategy to surcharge the Service Centre area at the time of the detailed design proposal. These works were anticipated to be for a period of six months. Upon completion of the bulk earthworks, construction of the off-ramp and internal roads commenced with a planned completion date of February 2014.

The main challenges for the project were the construction of the road embankment on soft ground, requiring safe and economical ground improvement technology for timely delivery of the project, together with a minimal impact of new bulk earthworks on the existing and new infrastructures adjacent to the project.

In particular, the area adjacent to the east corner of Moreton Drive and Nancy Bird Way intersection required construction of the road embankment within the vicinity of the existing twin bridges supported on piles. The bridges had extremely tight tolerance with regard to the additional allowable movements caused by the new bulk earthworks. The soil movement induced by the new bulk earthworks was expected to generate additional lateral forces imposing on the existing bridge piles. According to a preliminary assessment, preloading with 3.5m surcharge and wick drains will give rise to excessive ground movements underneath and adjacent to the embankment which have an adverse impact on the existing bridge piles. Therefore, it decided to adopt lightweight fill (geofoam) together with preloading and wick drains in order to mitigate the ground movement to an allowable range as well as minimise the impact on the nearby bridge structures.

3 GROUND CONDITIONS

3.1 SUBSURFACE CONDITIONS

The site of Banksia Place is located close to the mouth of the Brisbane River, and the subsurface geology generally consists of a sequence of alluvial, fluvial, estuarine and deltaic sediments deposited during the Holocene and Pleistocene ages, overlying Tertiary sediments of the Petrie Formation which typically consists of mudstone, siltstone and sandstone, and basalt.

The Holocene alluvial deposits are typically divided into the Upper Holocene alluvium and Lower Holocene alluvium. Upper Holocene alluvium was formed during the most recent rise in sea level, comprising very soft to firm, grey and brown clay of typically high plasticity, with interlayered silts or sands with some organic content. The underlying Lower Holocene alluvium was deposited in deeper water, comprising dark grey clay of high plasticity, with thin lenses of sandy soils. The clays are typically very soft in consistency, grading with depth to become firm and extending to significant depths, in excess of 20m in places. In some areas of the site, a variably thick sand layer is locally present at the interface between the Upper Holocene alluvium and the Lower Holocene alluvium. The underlying Pleistocene deposits consist of clays, sandy clays or clayey sands. The clays are typically stiff to hard, overconsolidated and of high plasticity. The sandy soils are typically of a medium dense to dense condition.

Groundwater was found at a depth of between 0.2m and 2.2m below natural ground surface level. The depth to groundwater is expected to vary with seasonal changes, tidal changes, and groundwater levels in the local area adjacent to Brisbane River.

3.2 GEOTECHNICAL CHARACTERISTICS

The geotechnical characteristics of the highly compressible soft clays, i.e. Upper and Lower Holocene clays, encountered on the site were evaluated on the basis of the results obtained from a suite of laboratory testing and field investigations, primarily comprising cone penetration tests and boreholes. Key geotechnical characteristics associated with index, deformation and strengths are summarised in Figure 2.

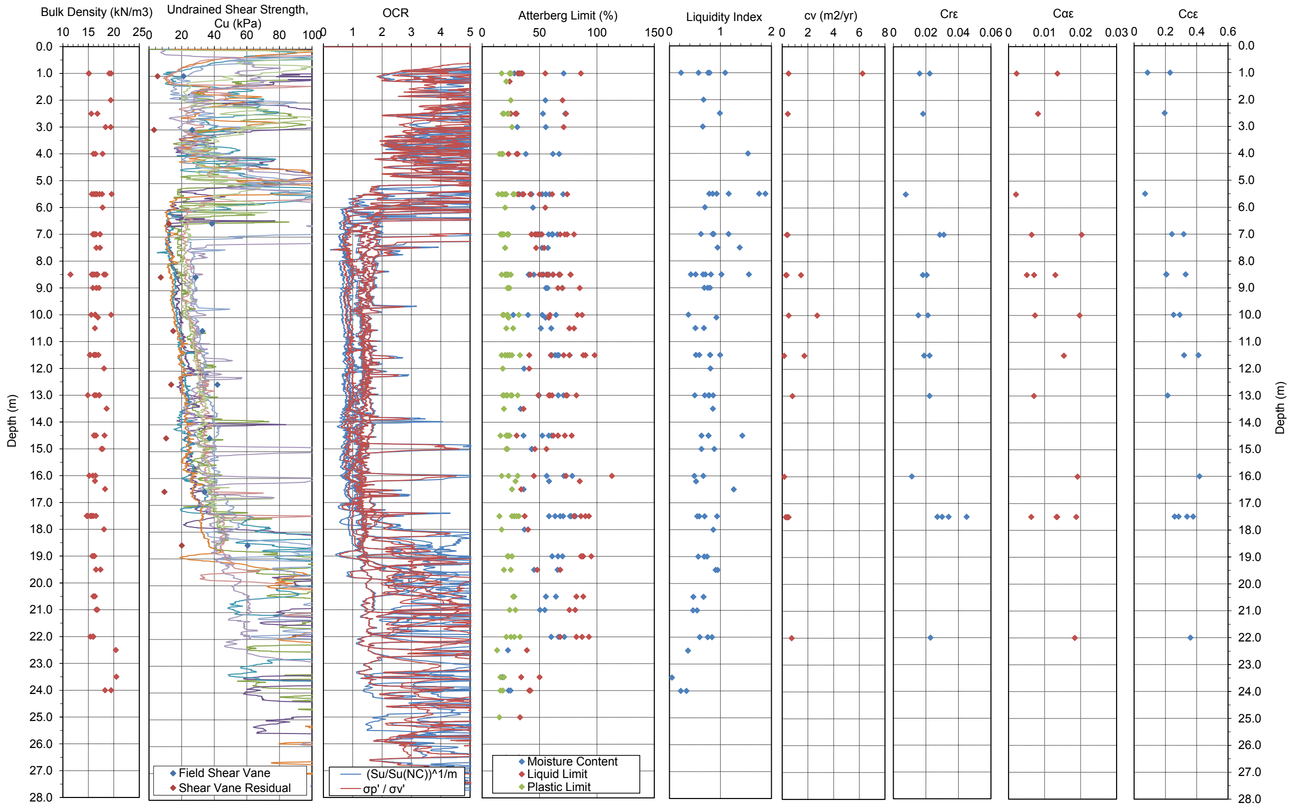

Figure 2: Geotechnical characteristics of subsoils derived based on laboratory and field testing.

Index characteristics were derived using moisture content and Atterberg limits for soil classification. Holocene alluvial soils are typically of a high plasticity with water content (*w*) of 20-85%, liquid limit of 23-99%, and plasticity limit of 17-33%. For the purposes of settlement estimation, consolidation characteristics including modified compression and recompression indices, $C_{c\varepsilon}$ ($=C_c/(1+e_0)$) and $C_{r\varepsilon}$ ($=C_r/(1+e_0)$), and modified creep index, $C_{\alpha\varepsilon}$ ($=C_\alpha/(1+e_0)$), were utilised, where C_c = coefficient of compression, C_r = coefficient of recompression, C_α = creep index, and e_0 = initial void ratio. The rate of consolidation was defined by the coefficients of consolidation in the vertical and horizontal directions, C_v and C_h. The typical range of geotechnical parameters for both Upper and Lower Holocene alluvial soils are presented in Figure 2. The Upper Holocene layer is highly compressible and relatively rapid consolidation whilst the Lower Holocene layer is expected to consolidate very slowly, taking many years to complete consolidation depending on its thickness and drainage path length. Strength characteristics were determined in terms of undrained strength (S_u) and drained effective strengths (c' and ϕ'). The interpreted S_u for the

Upper Holocene layer shows a great scattering range due to thin layers of silts or sands interbedded at various depths, whilst the Lower Holocene clay shows a trend of a gradual increase in undrained shear strength with depth varying from 10kPa to 20kPa.

4 DESIGN AND CONSTRUCTION

4.1 DESIGN CONSIDERATIONS AND METHODOLOGY

The design of a geofoam roadway embankment over soft soil was carried out in accordance with National Cooperative Highway Research Program (NCHRP) Report 529, web document 65 (Stark et al., 2004a; Stark et al., 2004b) and Transport Research Laboratory (TRL) Contract Report 356 (Sanders and Seedhouse, 1994) that provide the basis of the design framework for a geofoam application in roadway embankments.

The geofoam embankment primarily consists of foundation soil, fill mass including EPS (Expanded Polystyrene)-block geofoam and earth fill (fill cover), and pavement system, as shown in Figure 3. The design requires not only individual design of three major components of the embankment but also external and internal stabilities of the overall embankment taking into account interaction between its components. External stability of the embankment focuses on interaction of the embankment incorporating a mixture of EPS-block geofoam and normal density fills with foundation soil, whilst internal stability considers stability within the embankment itself. Both external and internal stability checks require design considerations for Serviceability Limit State (SLS) and Ultimate Limit State (ULS).

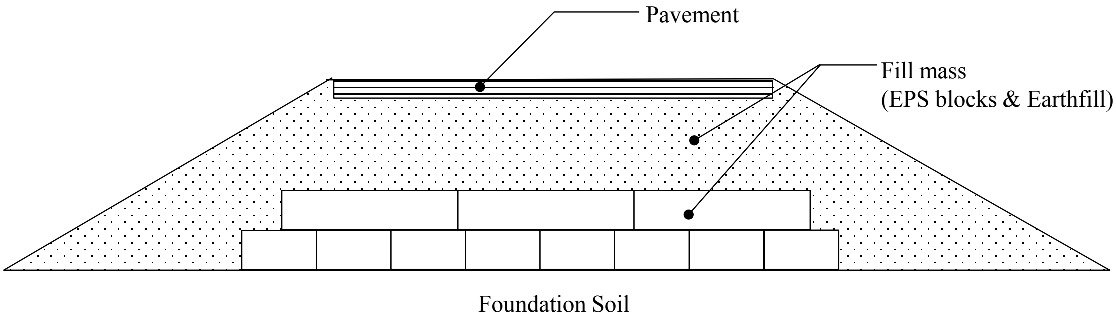

Figure 3: Typical cross section of an EPS-block geofoam embankment.

An overview of the geofoam embankment design procedure is summarised below:

- Gather background investigation information including ground conditions at the project site, maximum design flood levels for a return period similar to the design life of the embankment, temporary or permanent design loads applied to the embankment, and the locations and details of adjacent structures and services.
- Establish representative ground model including subsurface profile and geotechnical parameters.
- Undertake design checks of external (global) stability of the overall embankment that includes total and differential settlement of the soft foundation soil (SLS) and stability of the overall embankment (ULS) in relation to bearing capacity and slope stability under various loading conditions (e.g. hydrostatic uplift (flotation), lateral loading due to water and wind, and seismic loading) to the greatest extent practicable for the design life of the embankment.
- Undertake design checks of internal stability of the embankment including immediate and time-dependent (creep) settlement of the EPS-block geofoam mass due to overlying fill and pavement and stability under water, wind and seismic loadings.
- Undertake design of pavement in order to select suitable pavement type and arrangement.
- Estimate post construction settlement (PCS) that are likely to comprise any remaining primary consolidation settlement of the foundation soils due to dissipation of excess pore water pressure generated as a result of loading and secondary (creep) settlement of the foundation soils as a result of internal compression of the soil structure itself.

4.2 DESIGN CRITERIA

The project was required to fulfil the following design criteria for the geofoam embankment in terms of settlement and stability.

- Design life: 20 years
- Bearing capacity of embankment: FoS = 3
- External stability – Slope stability of embankment: FoS = 1.5
- External/internal stability – Seismic stability of embankment: FoS = 1.2
- External stability – Hydrostatic uplift: FoS = 1.2
- Internal stability – Load bearing (minimum required elastic limit stresses for EPS under pavement systems / within the EPS block): FoS = 1.2
- Post construction settlement of 150mm over 20 years within a structural zone
- Differential settlement gradient within the transition zone less than 1(vertical):200(horizontal)

The area of lightweight fill treatment was specified as a structural zone which was defined as a length not less than 25m within the approach to any structures such as existing bridge and culverts. This structural zone required strict control of the post construction settlement during its design life, whilst the non-structural zone allows the post construction settlement up to 225mm over 20 years.

4.3 GROUND MODEL AND GEOTECHNICAL DESIGN PARAMETERS

A representative ground model for the particular area of lightweight fill was developed based on existing geotechnical information. The geotechnical design parameters adopted for design of geofoam embankment were derived considering the potential variability of the ground conditions. The coefficient of consolidation in the horizontal direction (C_h) was assumed to be $2C_v$, where C_v is coefficient of consolidation in the vertical direction.

The embankment and surcharge fill primarily consist of a silty gravelly sandy clay mixture without any organic material, rocks, stones or any other hard materials that can retain on 75mm sieve size or any other unsuitable material that cannot support pavement.

The design groundwater level was assumed to be present at the ground surface level with hydrostatic pore water pressure distribution. For stability check against hydrostatic uplift, a maximum design flood level was assumed to correspond to finished road surface level.

Table 1: Geotechnical design parameters.

Material	Depth (top of layer) (m)	γ (kN/m^3)	c' (kPa)	ϕ' (°)	E' (kPa)	ν	S_u (kPa)	OCR	$C_{c\varepsilon}$	$C_{r\varepsilon}$	$C_{\alpha\varepsilon}$	C_v (m^2/yr)
Surcharge Fill	-	20	5	30	50000	0.3	-	-	-	-	-	-
Crust	0	18	5	30	8000	0.3	-	-	-	-	-	-
Upper Holocene Clay	0.7	16	2	24	2000	0.3	20	2.1	0.25	0.038	0.01	10
Holocene Sand	2.3	18	0	30	10000	0.3	-	-	-	-	-	-
Lower Holocene Clay	7.0	16	2	24	1000	0.3	Max (10 , 1.54z +0.77)	1.3	0.3	0.045	0.015	2.5
Pleistocene Clay	16.5	17	5	30	15000	0.3	90	2.5	0.04	0.006	0.002	15

4.4 DESIGN PROCEDURE

Design of EPS-block geofoam embankment was carried out with the following steps:

Step 1 – Selection of EPS block type and embankment arrangement

RMAX EPS (expanded polystyrene) geofoam Grade H was initially selected and finally verified during design process. It has the following physical properties: density 24kg/m^3 (equivalent to approximately 1% weight of normal earth fill), nominal compressive strength σ_{c10} = 135kPa at 10% strain, allowable compressive stress σ_{c2} = 100kPa at

2% strain and σ_{c1} = 48kPa at 1% strain, and long term water absorption by total immersion ≤ 3%. The standard commercially available size for the RMAX EPS geofoam for fill use is 5000mm x 1220mm x 600mm blocks.

For cost-effective design, the geofoam embankment arrangement was optimised in order to minimise the volume of EPS-block geofoam, which are generally more expensive than normal fill materials. Two layers of EPS blocks with a total thickness of 1.2m and 1.8m thick overburden consisting of earth fill and pavement were considered to form the geofoam roadway embankment, up to 3m in height. A crest width of the embankment of approximate 11.0m was considered with 1(vertical):2(horizontal) battered side slopes. The depth of soil covering the sides was considered not to be less than 250 mm, measured normally to the planes bounding the installed blocks.

Step 2 – External stability evaluation

Total settlement of an EPS-block geofoam embankment consists of immediate or elastic settlements of both the fill mass and the foundation soil, primary and secondary (creep) settlement of the foundation soil, and long-term creep of the EPS-block geofoam. Immediate or elastic settlements of both the fill mass and the foundation and creep of the EPS-block geofoam assemblage were ignored due to their negligible potential impact on the final pavement construction. On the basis of settlement calculations using one-dimensional consolidation theory, the estimated post construction settlement (comprising primary and secondary settlements) over 20 years was 270mm which exceeded the criteria when compared with the allowable settlement for structural zones (i.e. 150mm). A number of alternatives for foundation improvements were also assessed, including a geofoam embankment with helical screw piles. Prior to placement of EPS-block geofoam, six months preloading of 3.5m surcharge with prefabricated wick drains spacing of 1.0m in triangular pattern was eventually concluded to be most feasible and cost-effective in reducing post construction settlement to an acceptable level, particularly in relation to secondary creep settlements. A maximum primary settlement of 800mm was calculated within the preload area after 6 months of treatment, and a further 100mm post construction settlement over 20 years.

Bearing capacity of the foundation soil and static and seismic stability of the overall geofoam embankment were shown to be satisfactory with the design criteira. For external stability checks, the overburden loads due to the weight ofthe fill mass (EPS blocks and earth fill), pavement and traffic loads was modelled as equivalent pressure that was vertically applied to the surface of the foundation soil. The shear strengths of the EPS blocks, earth fill and pavement were threfore ignored. Use of equivalent pressure instead of modelling the complex behaviour of the EPS-block geofoam embankment has been used in estimating slope stability through the soft soil foundation (Public Works Research Institute, 1992). An example of the external stability check for the simplified EPS-block geofoam embankment is presented in Figure 4.

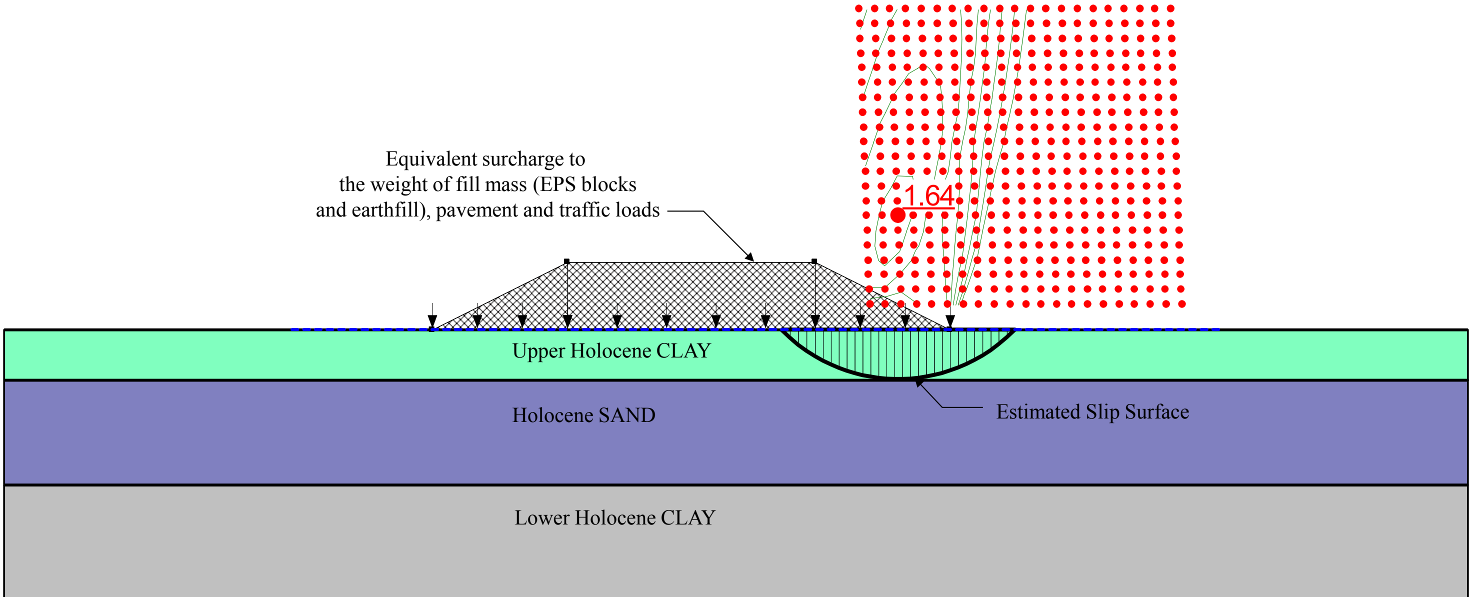

Figure 4: External stability check of the simplified EPS-block geofoam embankment.

Due to its extremely light weight, the EPS-block geofoam embankment is vulnerable to buoyancy and uplift resulting from seasonal flooding, tidal effects on water table and high winds. Common methods to counteract buoyancy include the use of sufficient overburden or mechanical constraints such as helical screw piles or tension piles. The potential for hydrostatic uplift (flotation) of the entire geofoam embankment under an extreme flood event was assessed with a maximum design flood level assumed to be at the finished road surface level. The factor of safety against buoyancy of the embankment is calculated as the ratio of the overburden weight available from the geofoam embankment to the uplift water force at the base of the embankment. As the geoform is extremely light, sufficient overburden is allowed over the EPS blocks to achieve the required factor of safety. For a road embankment of normal width and moderate height, stability due to lateral loading induced by wind or water is unlikely to be a problem for the completed embankment.

Step 3 – Internal stability evaluation

Internal seismic stability assessment considered failure modes occurring within the geofoam embankment along the following interfaces: 1) between the overburden fill and the EPS blocks, 2) between two consecutive EPS blocks layers and 3) between the EPS blocks and the foundation soil. The internal stability of the embankment was assessed to meet the design criteria with an assumed horizontal seismic coefficient of 0.2.

An EPS geofoam product was selected to have the required properties to support the overlying earth fill, pavement and traffic loads. Such product, when subjected to the estimated loads, will remain to be within the elastic range with an elastic limit stress equivalent to the compressive stress at 1% strain. The analysis indicated that RMAX EPS (expanded polystyrene) geofoam Grade H was appropriate, satisfying the required EPS elastic limit stress for placement underneath the fill and pavement.

Step 4 – Final embankment design

An EPS-block geofoam embankment is inherently susceptible to damage resulting from many potential causes, e.g. petrol and chemical attack, heat and flame, ultra violet light and so on. Geofoam can be protected from most of these risks by being covered with a layer of earth fill and pavement materials. However, these cover materials may be permeable to petrol and other fluids. In the design, the EPS-block geofoam assemblage was encapsulated with impermeable and chemical resistance membrane of 0.75mm in thickness (EnviroLiner 6030) and a 150mm thick lightly reinforced concrete slab (C20/25) was casted in place in order to provide an effective barrier against any chemical spills which could dissolve the EPS-block geofoam. This slab is deemed to be beneficial in improving the distribution of wheel loads. A typical cross section of the EPS-block geofoam embankment is presented in Figure 5 with detailed arrangement of relevant components.

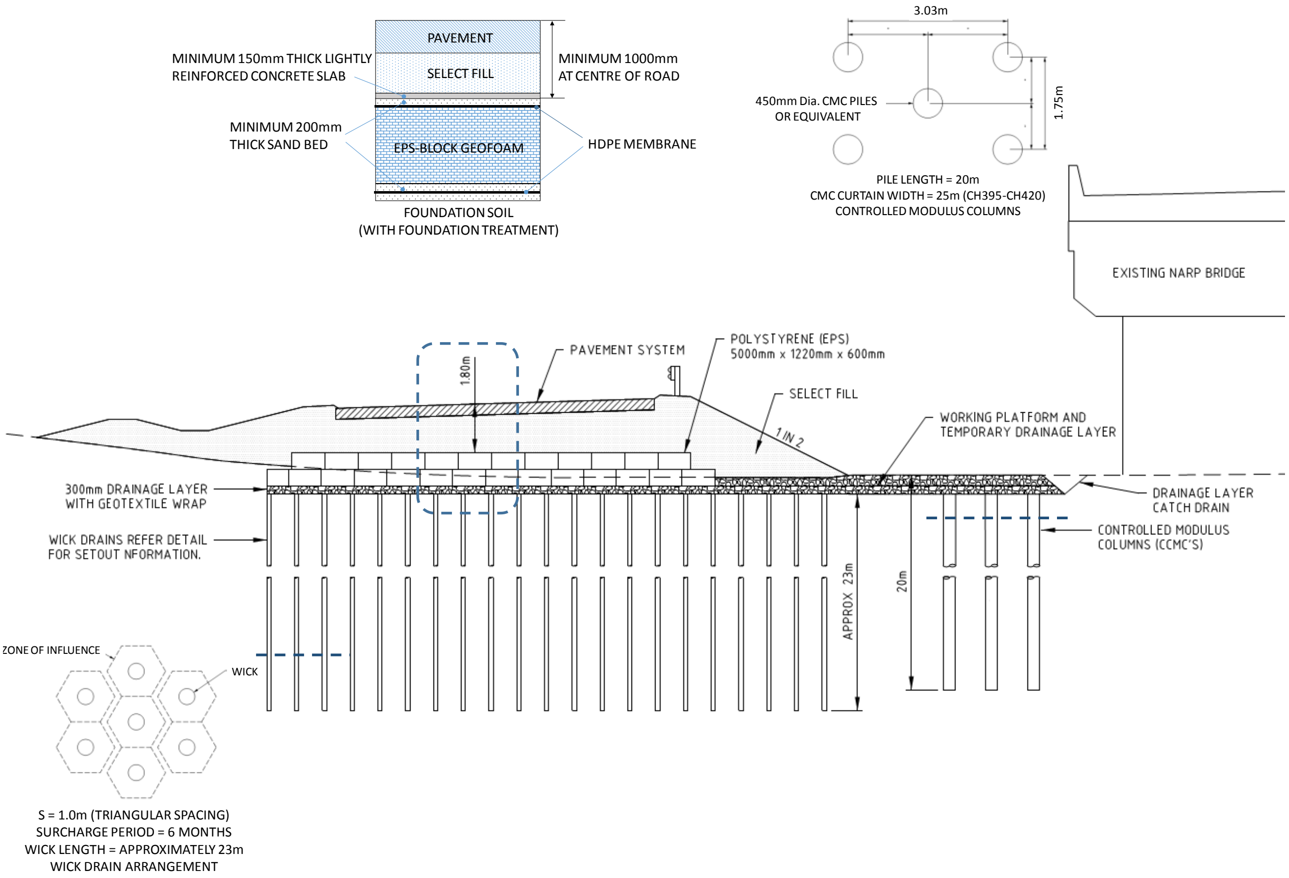

Figure 5: Typical cross section and arrangement of the proposed geofoam embankment.

Where the geofoam embankment adjoins an earth fill embankment, transitional arrangements were designed by adopting progressive reduction in the thickness of the EPS-block over the transition zone. Sufficient length of the transition zone was allowed to minimise differential settlements between different embankment zones and to achieve a settlement gradient of less than 1(vertical):200(horizontal).

4.5 CONSTRUCTION OF GEOFOAM EMBANKMENT

Prior to the placement of EPS-block geofoam to form the roadway embankment, long-term performance of the foundation soils were improved by means of six months preloading with 3.5m surcharge and wick drains (1.0m centre-to-centre spacing in a triangular pattern). The impact of preloading on the existing bridge piles was minimised by three rows of control modulus columns installed to a depth of 20m below ground level between the preload embankment and the bridge (see Figure 5 for details).

(a) Excavation to base level (b) Foundation preparation

(c) Geofoam installation (d) Protective membrane installation

(e) Earth fill cover placement (f) Pavement construction

Figure 6: Geofoam embankment construction.

The site for lightweight fill treatment was firstly prepared by removing the placed surcharge down to the proposed geofoam base. A basal geotextile was placed over the excavated surface on which a bedding layer of sand of a minimum of 200mm in thickness was placed and levelled to prepare a smooth and planar surface. A protective membrane sheet (HDPE) was laid over the bedding sand layer and then covered by another layrer of sand of a minimum of 200mm thickness. This upper sand layer was to allow increased friction between the HDPE membrane and the EPS-block geofoam. The first layer of EPS-block geofoam was placed on the sand bedding layer with staggered joint arrangement where voids and open joints were avoided. The subsequent layer of the geofoam blocks was placed in a similar fashion. The stepped sides of the geofoam generally required the blocks to be resized to suit the required geometry using a hot wire. To prevent displacement of the blocks during construction they were fixed together at intervals with polyurethane-based adhesive or mechanical fasteners. Once geofoam blocks were in place, they were covered and sealed by joining the protective HDPE membranes using extrusion welding as soon as possible. Minor damage to the membranes was repaired with the patch extending at least 200mm outside the damaged area. The overburden earth fill was placed and compacted under controlled and certified conditions. The pavement was finally constructed in accordance with usual practices and rules. Figure 6 shows stages of the geofoam embankment construction.

Any water at or near the ground surface was pumped off until the geofoam blocks are covered by overburden materials with sufficient weight to prevent floatation. A temporary culvert placed on the site during the preloading period was replaced with a permanent one prior to construction of pavement.

5 CONCLUSIONS

The application of lightweight fill (EPS-block Geofoam) was successfully implemented for construction of the road embankment on soft alluvial soils for the new Banksia Place Stage 1A Service Centre development. It has demonstrated that the geofoam embankment can significantly reduce risks associated with embankment construction on soft ground in terms of slope stability, settlement and impact on adjacent structures.

Furthermore, use of EPS-block geofoam allows rapid construction of full-height embankment in a short period of time without any difficulty in handling construction materials. The new Banksia Place Stage 1A Service Centre development was successfully completed and opened in July 2014 as planned.

6 REFERENCES

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