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Design and performance of an embankment supported using low strength geogrids and vibro concrete columns

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ABSTRACT: The new toll plaza for the Second Severn Crossing was constructed about 2.5km west of the Crossing on low lying land adjacent to the estuary. An embankment generally 2.5 to 3.5m high with a maximum height of some 6.0m was constructed over highly compressible peat and clay soils. An innovative foundation system comprising a load transfer platform, incorporating low strength geogrids at the base of the embankment, and vibro concrete columns was used to support the embankment. The initial site tests, detailed design and construction of the ground treatment works and the embankment are described. The results of monitoring of the permanent works construction are also discussed.

INTRODUCTION

The Second Severn Crossing provides a second motorway link between South Wales and England across the River Severn estuary. The concession for constructing and operating the crossing was let to Severn River Crossing plc, a consortium led by John Laing Construction and GTM Entrepose. The designer was Sir William Halcrow & Partners Ltd in joint venture with Societe d'Etudes et d'Equipements d'Entreprises. The new 5.2km long crossing, sited about 5km downstream of the existing Severn Bridge comprises 2.2km long viaducts from both the Avon and Gwent shores and a 912m long cable stayed main bridge spanning some 456m across 'The Shoots' navigation channel. In addition to those structures a new toll plaza, covering an area of 16 hectares approximately was constructed some 2.5km west of the Crossing on the low lying Gwent levels, ref Figure 1. Ground levels at the toll plaza site were generally between about 4 and 5mOD. To alleviate the risk of flooding from extreme tidal events, ground levels were generally raised by between 2.5 and 3.5m increasing at the west end of the toll plaza to 6m maximum height as the proposed new motorway climbs to cross the London to South Wales main railway line.

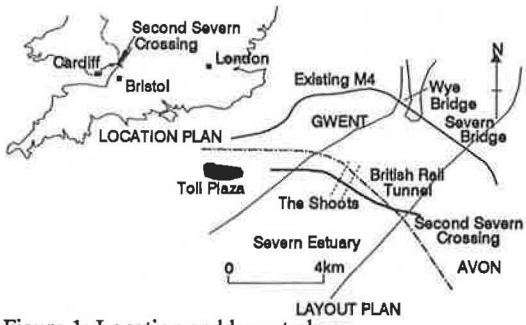

Figure 1: Location and layout plans

GROUND & GROUNDWATER CONDITIONS

The ground investigation at the toll plaza site included eleven boreholes sunk by cable percussion and rotary coring techniques to some 10m maximum depth, forty static cone penetration tests and a comprehensive range of laboratory testing for the determination of soils classification, shear strength and consolidation properties. The exploratory holes proved some 4.4 to 6.4m thickness of highly compressible peat and estuarine clay soils as shown on Figure 2. The upper stratum of estuarine clay was typically 2 to 3m thick and a firm to stiff desiccated crust existed at the surface. The deposits comprised silt and clay with up to 50 per cent sand and were mainly soft to very soft. These soils were underlain by amorphous peat containing up to 40 per cent clay, typically 2 to 3m thick, locally 4m thick. A lower stratum of estuarine clay less than 1m thick was found to underlie the peat. Sand and gravel deposits below were predominantly

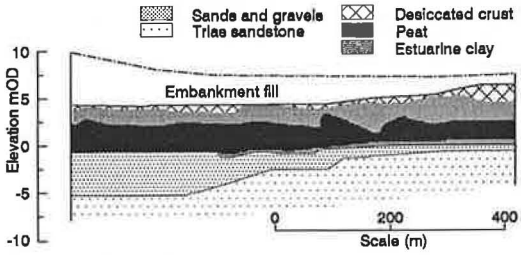

Figure 2: Geological section

well graded with some 40 to 70 per cent gravel except at the west end of the site where they were found to comprise a uniformly graded silty fine sand with less than less than 5 per cent gravel. Trias sandstone bedrock was proved to underlie the superficial deposits. A summary of the engineering design parameters derived from in situ and laboratory testing are presented in Table 1. The groundwater table at the site was monitored and was found to fluctuate between the sands and gravels and the base of the desiccated crust.

Table 1: Summary of engineering properties

Design parameters median (moderately conservative)	Desiccated Clay	Estuarine Clay	Peat	Sands & Gravels
Moisture content (%)	24	53	215	14
Bulk density (Mg/m³)	1.83	1.78	1.03	1.92
Dry density (Mg/m³)	1.48	1.26	0.38	1.65
Particle density	2.62	2.66	1.95	2.64
Liquid limit (%)	52	54	262	NA
Plastic limit (%)	24	25	164	NA
Plasticity index (%)	29	28	98	NA
Organic content (%)	8	5	46	NA
Undrained shear strength (c_u (kN/m²))	(40)	(15)	(17)	NA
Effective shear strength (c' (kN/m²),ϕ' (deg))	(5,25)	(0,29)	(10,23)	(0,30)
Coeff of vol compress. (m_v (m²/MN))	0.20	0.57	2.00	NA
Coeff of consolidation (c_v (m²/yr))	2.5	2.0	12	NA
Coeff of secondary consolidation (c_{sec})	0.002	0.004	0.027	NA
Deformation modulus (MN/m²)	-	-	-	62(1)
Shear modulus (MN/m²)	-	-	-	27(1)

Notes: NA - Not applicable
(1) - Derived from back analyses of VCC load tests

GROUND TREATMENT OPTIONS

For embankment construction of up to 3.5m height using conventional fill, consolidation settlements of up to 0.65m were anticipated in the founding estuarine clay and peat soils. At the west end of the toll plaza, expected settlements of up to 1.0m were calculated. From laboratory consolidation test results and data published by Hobbs (1986) and Jones et al (1986) it was estimated that primary consolidation of these deposits might take between 6 months and 3 years. In addition secondary consolidation (long term creep) settlements of up to 0.2m over a 120 year design life were calculated. These ground conditions necessitated the implementation of ground improvement measures to mitigate primary consolidation settlements, reduce long term secondary consolidation settlements and to limit differential movements along the carriageway to acceptable levels. Design criteria of 100mm maximum post construction settlement over 120 years with an angular distortion due to differential movements not exceeding 1 in 1000 were adopted.

The conventional ground improvement/foundation systems which were considered viable for use at the toll plaza were

- vertical drains and surcharge
- excavation of the highly compressible soils and replacement with rockfill
- piling with a high strength geosynthetic at the base of the embankment to carry the embankment loads.

An innovative alternative foundation system fulfilling many of the advantages of a piled embankment was proposed by Keller Ltd and Netlon Limited. That system comprised

- vibro concrete columns (VCCs), with a maximum working load of 600kN, installed on a triangular grid of 2.7m maximum spacing founding in the sand and gravel deposits
- a load transfer platform at the base of the embankment, comprising 75mm down granular fill incorporating two layers of low strength geogrid (Tensar SS2) to promote arching in the granular fill to transfer the embankment loads into the columns. The outline design of the load transfer platform assumed that the platform would arch between the VCCs. The two layers of reinforcement would promote arching and support the material below the arch.

The estimated cost of these works was similar to the cost of the vertical drains and surcharge option and about half the cost of the other options. The VCC and load transfer platform system minimised the uncertainty with regard to the time for construction inherent with vertical drains and surcharge designed without the benefit of representative field trials. A smaller scale application of the general technique using piles and a geogrid reinforced load transfer platform had proven successful elsewhere, ref Card and Carter (1995). The system proposed for the toll plaza employing some

11,700 VCCs and a load transfer platform incorporating two layers of Tensar SS2 was a substantial extension to previous experience. Site tests were undertaken to

- o confirm the suitability of the proposed construction methods
- o prove the performance of the individual construction elements and of the composite system
- o establish parameters for use in detailed design
- o establish criteria for construction and monitoring of the permanent works

INITIAL SITE TESTS

The initial site tests included static and dynamic load testing of two VCCs and a 'patch test' modelling the proposed permanent works design. The 'patch test' represented the proposed design system and was undertaken in a trial area where ground conditions were considered typical of most of the area, ref Figure 3. The VCCs were constructed using a Vibrocat unit. Details of the process are given by Bell et al (1994).

The VCC load tests were performed in accordance with the ICE Specification for Piling (1988) with step loading to a maximum load of 1.65MN. The interpreted results of the tests are summarised in Figure 4. The test results were analysed using 'stability plots' after Chin (1978). The analyses indicated that the total ultimate resistance of the VCCs to be about 1.85MN with ultimate end bearing and ultimate skin friction components of 0.96MN and 0.89MN respectively. The load tests were also analysed using axi-symmetric finite element analyses. From those analyses the deformation properties of the founding sand and gravel deposits were back figured for use in detailed design, ref Table 1.

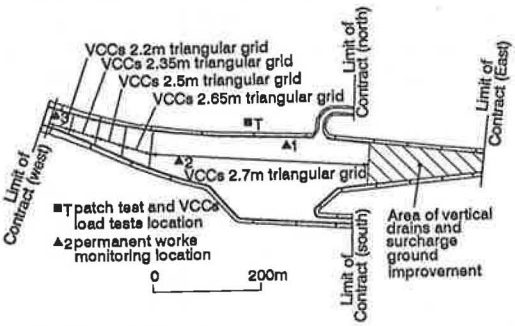

Figure 3: Toll plaza layout plan summarising ground improvement works

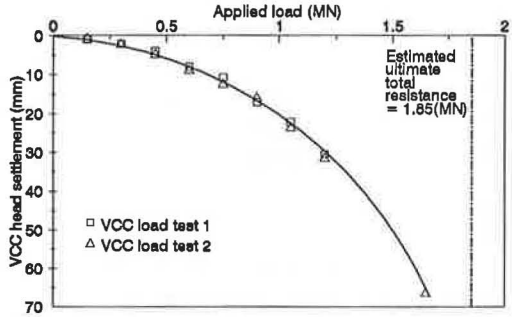

Figure 4: VCC load test results

The configuration of the patch test is shown on Figure 5. The trial covered an area of 7.2m by 7.2m at the centre of which four VCCs were installed on a 2.5m square grid. The load transfer platform was constructed at the ground surface and comprised a 0.6m thickness of well graded, 75mm down granular fill incorporating two orthogonally laid layers of Tensar SS2 geogrid. The grading of the fill was selected to minimise the potential for damage to the geogrids during construction and to develop an 'interlock' between the fill particles and the geogrid. Some 0.5m of rock fill (Class 1C DTp Specification for Highway Works (1991)) was placed above the load transfer platform to promote arching within the fill. Kentledge loading of 60kN/m^2 was applied to model the embankment loading. The kentledge was provided by concrete blocks placed on 0.5m thickness of rockfill over the load transfer platform. The system limited the possibility of the arch forming above the transfer platform. It was, therefore, a more severe test than that which would occur in practice.

The deflections of the VCCs were monitored using settlement markers installed at their heads. Displacements of the geogrid layers were monitored at seventeen markers installed in a 'H' pattern between the VCCs on the lower and upper geogrid layers. The settlement observations are presented on Figure 6.

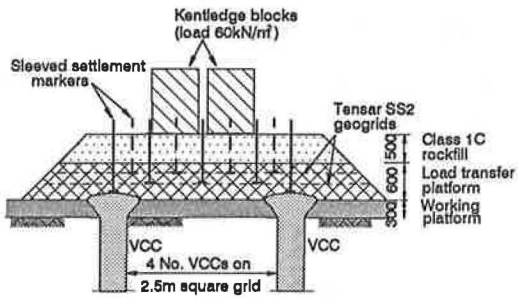

Figure 5: Patch test set up

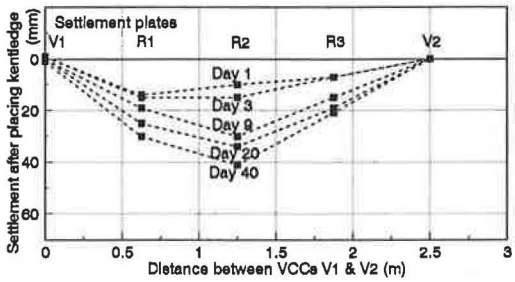

Figure 6: Patch test - geogrid profile

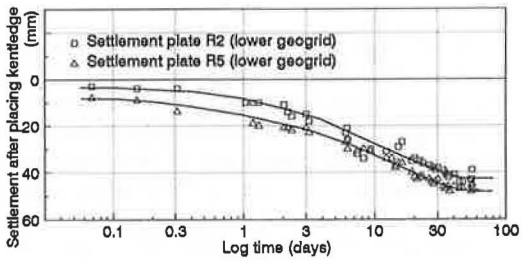

Figure 7: Patch test - monitored settlements

Settlements at the heads of the VCCs were minimal. Maximum settlements of between 40 and 50mm were recorded mid-span between the columns after kentledge placement. These were completed within about 40 days with no evidence of continuing creep settlements as shown on Figure 7.

On completion of monitoring the patch test embankment was dismantled and the geogrids inspected for evidence of distortion. The geogrids were found to be in good condition with only limited localised minor damage, apparently the result of construction. One of the VCCs was exhumed to 3.5m depth and revealed the column to be of relatively uniform construction in both the peat and estuarine clay deposit with a nominal shaft diameter of 430mm and an enlarged head of some 550mm nominal diameter.

GEOGRID PROPERTIES

The design of the load transfer platform requires the geogrid to perform two functions

- o restraint of the granular fill particles so that the arching mechanism is set up within the platform layer
- o support of the granular fill below the arch

The form of the geogrid gives it the ability to interlock with and to restrain the granular fill particles thus enabling the peak shear strength of the aggregate to be developed. That performance related property of geogrids is described by Guido et al (1987) and was demonstrated by the performance of the patch test described above.

The support of the granular fill below the arch requires long term tensile strength from the geogrid. The long term strength of the Tensar SS2 geogrid was derived from creep tests carried out on samples 5 ribs wide and 3 ribs long, tested at a range of loadings at constant temperature (20°C ± 1°C). The results of the tests in terms of load versus strain were analysed to assess the long term, 120 years, strengths of the geogrid to be adopted on this project. Those strengths are presented in Table 2 together with the quality control tests data for the geogrid. These data show the geogrid to possesses different strengths in orthogonal directions.

Table 2: Properties of Tensar SS2 geogrid

Property	Transverse Direction	Longitudinal Direction
Quality control strength at 20°C (95% lower confidence limit)	31.5kN/m	17.5kN/m
Quality control peak strain at 20°C	≈11 per cent	≈12 per cent
Long term (120 year) strength at 10°C	9.5kN/m	5.2kN/m

DETAILED DESIGN

Following the successful outcome of the 'patch test', the detailed design of the permanent works was undertaken by Halcrow-SEEE in consultation with Keller and Netlon. A traffic surcharge load of 20kN/m^2 was adopted in the design in accordance with Department of Transport Departmental Standard BD 37/88. Based on the ground investigation data, the results of the VCC load tests and employing an overall factor of safety of 3.0, a design load of 0.6MN maximum was adopted for individual VCCs. The spacing of the columns was initially sized to maximise their loading and checked in relation to the embankment cover to negate the risk of reflective settlements using data presented in Azam et al (1990). The spacing between VCCs was typically 2.7m but reduced with increasing embankment height as shown on Figure 3.

Finite element analyses covering the range of VCC spacings and embankment heights were undertaken using the soil parameters derived from the ground investigation data and the initial site tests to determine the loads in the load transfer platform and the proposed geogrid layers. A range of typical geotechnical parameters for granular fill forming the load transfer platform and the embankment rockfill based on published information was used in the analyses. The

Table 3: Summary of finite element analyses results

Property	Range of analyses results
Tensile force in load transfer platform	5.3 to 9.3kN/m run max
Compressive stress at top of load transfer platform above head of VCCs	160 to 200kN/m² max
Compressive stress at original ground level mid span between VCCs	minimal
Settlement at top of load transfer platform above head of VCCs	10 to 25mm(1)
Settlement at top of load transfer platform mid span between VCCs	15 to 30mm(1)
Differential settlement, load transfer platform top	up to 5mm(1)
Notes: (1) typical sand and gravel deposits	

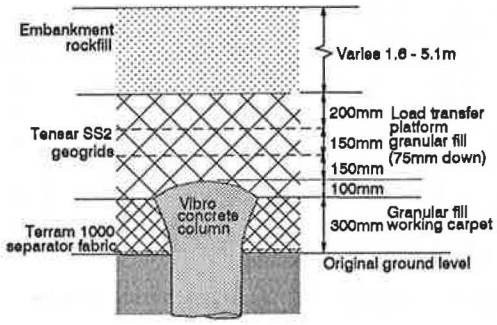

Figure 8: Permanent works construction

system was modelled as an axi-symmetric analysis with an equivalent radius for a triangular spacing determined from Smith (1978). The results of those analyses are summarised in Table 3. Along the sides of the embankment plane strain finite element analyses were also performed to assist in the determination of edge loadings and load transfer platform design.

From the results of the finite element analyses the geogrids were selected. Based on the calculated tensile loads in the load transfer platform, the long term (120 year) strengths of the geogrid and adopting a factor of safety of 1.4 as recommended by Netlon the tensile strength required for the load transfer platform was achieved using two layers of Tensar SS2 geogrid laid orthogonally to each other, ref Table 2 for the properties of Tensar SS2 geogrid.

CONSTRUCTION

Figure 8 illustrates the permanent works construction. The specification for the installation of the VCCs was based on ICE Specification for Piling (1988). About 11,700 VCCs were installed to tolerances of 100mm in plan and 1 in 50 vertical alignment. Construction of the VCCs commenced with the positioning of the poker over the column plan position. The poker was then charged with concrete and made to penetrate the ground by means of its vibratory action, produced by an eccentric weight in the poker rotated by an electric motor, its self weight and a pull-down force applied through the Vibrocat base machine. Once the weaker clay and peat soils were penetrated the founding sand and gravel was identified by a reduced rate of penetration and an increased energy demand from the vibrating poker monitored by the operator using 'incab' instrumentation, ref construction monitoring below. The construction of the VCC shafts commenced with pumping concrete, supplied from a local ready mix supplier, through the delivery pipe forming part of the poker to form a bulbous base to the columns in the sand and gravel deposits. The poker was then withdrawn through the peat and clay soils at a constant rate consistent with the rate of concrete injection to negate the risk of a neck forming in the column. Repenetration of the top 0.5m approximately of the VCCs was carried out to form an expanded head profile as shown on Figure 8. The columns were formed of 25N/mm² minimum strength low slump concrete meeting the requirements for Class 3 soils in accordance with BRE Digest 363 (1991). The columns were typically 5 to 6m in length and penetrated the founding sand and gravel soils to between 0.5 and 1.0m depth. Using two Vibrocat units construction of the VCCs took about 5 months to construct.

Construction of the load transfer platform followed the installation of the VCCs. The platform was constructed of well graded 75mm down crushed rock fill. The lower and central parts of the platform were compacted using a light weight vibrating roller (Bomag BW6), though vibration was not used in compacting the lower fill to minimise the risk of damaging the heads of the unreinforced columns. A heavier, Bomag BW10 vibrating roller was used to compact the upper fill. The two Tensar SS2 geogrid layers were laid orthogonally. Because the maintenance of the tensile properties of the geogrid layers was imperative, adjacent geogrid rolls were joined employing a three grid overlap (75mm approximately) and stitching, with a polymer braid, around coincident ribs.

At the embankment edges the geogrid layers were taken to the face, wrapped up the 1 in 2 slope and anchored back into the embankment over a 5m length, just above the load transfer platform. That detail provided continuity of the load transfer platform at the embankment face and minimised the potential for differential movements at the embankment toe and distress to the slope face. At the interface with the area treated by vertical drains and surcharge the load transfer platform was extended 5m beyond the VCCs.

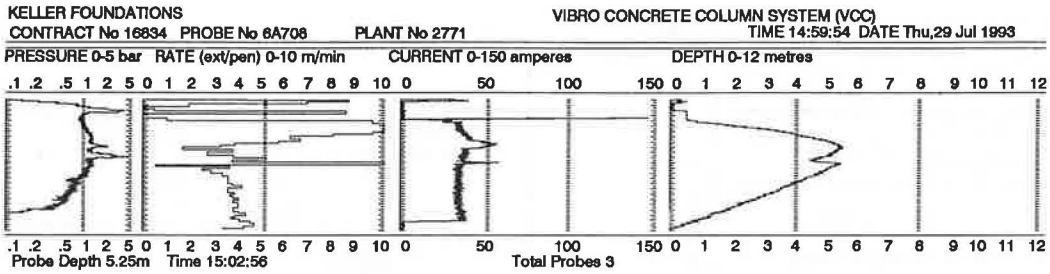

Figure 9: Typical VCC 'incab' monitoring print out

CONSTRUCTION MONITORING

The construction of the VCCs was monitored by 'incab' instrumentation which recorded the depth of penetration of the Vibrocat poker, rate of penetration, energy consumption (ammeter output) and the volume of concrete used. The hard copy print outs, ref Figure 9, of those data for each column constructed were analysed on site as the works progressed to maintain control of the process and to provide a checking system for potentially defective columns in terms of their construction and founding depth. The concrete for the columns was slump tested and cubes were made for 7 and 28 day strength testing. During the initial phase of construction, 1 in 20 VCCs were dynamically load tested. Following demonstration of consistency of construction and load test results this quality control was relaxed in stages to a minimum 1 in 100 columns. In total some 150 tests were carried out. Five working column static load tests with a maximum load of 1.5 x working load (0.9MN) were also undertaken. In all cases the VCCs were found to have higher carrying capacities than the initial test VCCs described above.

Three areas of the toll plaza embankment were instrumented to monitor the performance of the load transfer platform and the rock fill embankment, ref Figure 3. At each location, fifteen settlement markers in two orthogonal lines were installed both at the heads of and mid span between the VCCs as shown on Figure 10. The construction of the settlement markers is shown on Figure 11. Given the relatively small magnitude of the anticipated movements precise levelling was employed to measure the settlements. Four hydraulic pressure cells were also installed at each of the monitoring sites, two above the heads of VCCs and two at original ground level below the load transfer platform, ref Figures 10 and 12. At each monitoring site an additional 1m height of fill was placed above finished road level to represent traffic loading.

The hydraulic pressure cell observations shown on

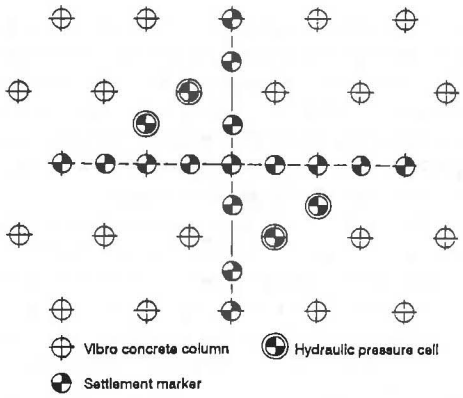

Figure 10: Monitoring instrumentation layout plan

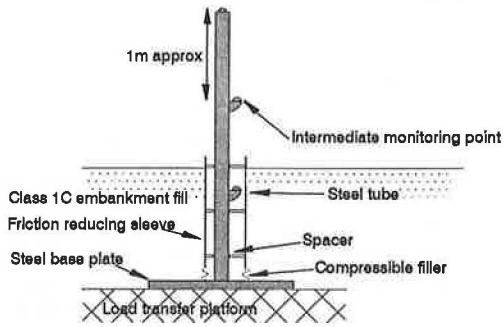

Figure 11: Details of settlement markers

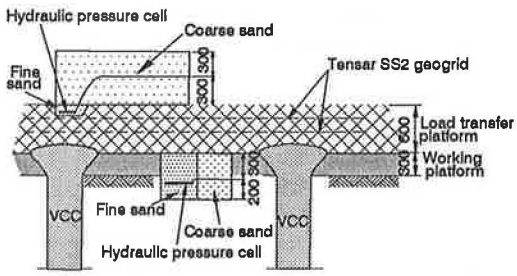

Figure 12: Hydraulic pressure cells monitoring layout

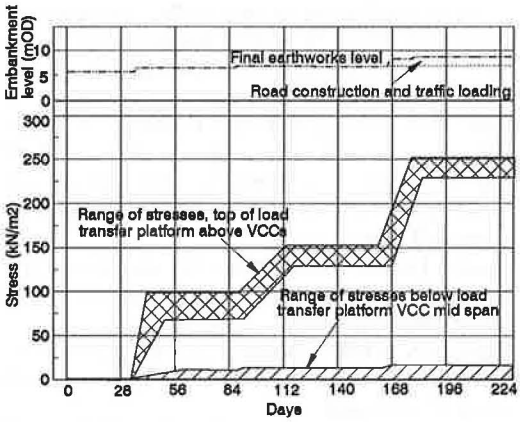

Figure 13: Typical monitored stresses above and below load transfer platform

Figure 13 are typical of those monitored at the sites and show increases in stress above the VCCs with each phase of construction. A maximum applied stress of 250kN/m^2 was measured consistent with the design analyses predictions. The hydraulic pressure cells installed below the load transfer platform showed increases in stress of some 5 to 10kN/m^2 with the initial phase of embankment construction. Small increases in stress were observed with subsequent phases of construction consistent with increased loading imposed as a result of settlement of the VCCs and the embankment. These results show that arching developed within the load transfer platform enabling the transfer of the embankment loads to the VCCs consistent with the design methodology.

Settlement observations at monitoring locations 1 and 2 (founding soils of sand and gravel) for construction up to final earthworks levels indicated a narrow range of displacements of the load transfer platform of some 5 to 10mm and 8 to 15mm above the VCCs and at mid span between them respectively as shown on Figure 14. Those settlements are consistent with the loads being carried in skin friction based on information published by Tomlinson (1994). The differential movements typically amounted to less than 5mm between the heads of the VCCs and mid span between columns consistent with small movements leading to the development of arching within the platform.

Placing fill to model the road construction and traffic loading caused settlements to increase to between 15 and 35mm maximum above the VCCs. At mid span the settlement increased to between 19 and 42mm. The differential settlement within the load transfer platform between the VCCs and mid span was generally less than 5mm. This suggests that the cause

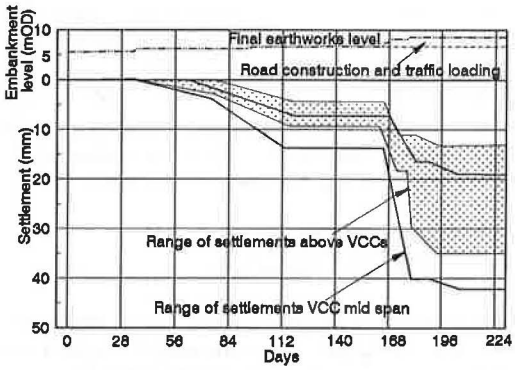

Figure 14: Typical settlements top of load transfer platform at monitoring locations 1 and 2

of the increased settlement was displacement of the VCCs into the founding stratum consistent with the development of end bearing resistance. This displacement resulted in a small increase in load on the compressible soils of the order of 5kN/m^2 as shown on Figure 13.

At monitoring location 3 in the west of the site, the VCCs were founded in silty sand soils. Settlements of between 35 and 75mm occurred, greater than anticipated. These larger settlements, compared to those monitored at locations 1 and 2, would seem to be the consequence of the lower bearing capacity of the soils in that location and the less than expected penetration of the VCCs into founding soils. Such movements were however readily accommodated given the 'flexible' nature of the load transfer platform and the embankment. The differential settlements between the VCCs and mid span were generally less than 5mm similar to those recorded at locations 1 and 2.

Level surveys of the toll plaza over the 12 months following the completion of road construction have revealed no movements of the embankment supported on VCCs and the load transfer platform.

CONCLUSIONS

An innovative system of ground improvement comprising VCCs and a load transfer platform incorporating low strength geogrids was employed to support the embankment for the new toll plaza at the Second Severn Crossing. The system provided an effective, financially competitive solution for the construction of an embankment on highly compressible peat and clay soils with negligible risk to the contract programme. The design methods adopted based on a patch test trial and finite element analyses proved

reliable on the basis of the monitoring results. The combined qualities and close co-operation of the experienced geotechnical designer, specialist geotechnical processes contractor and geogrid manufacture were of primary importance in the development and successful implementation of the ground treatment works at the toll plaza site.

ACKNOWLEDGEMENTS

The concession company for this project was Severn River Crossing plc, a consortium led by John Laing Construction in joint venture with GTM Entrepose. The designer was a joint venture between Sir William Halcrow & Partners Ltd and Societe d'Etudes et d'Equipements d'Entreprises. The toll plaza embankment support system was designed in consultation with Keller Ltd and Netlon Limited. The authors express their thanks to their colleagues in these organisations.

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