

Case history of a geogrid-reinforced embankment supported on vibro concrete columns

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ABSTRACT: A modern concept for foundation of a carriageway crossing deep deposits of weak organic soils was applied for the first time in Poland. The constraints to the project caused by poor ground conditions and a high groundwater table were successfully overcome using combination of two geotechnical techniques: Vibro Concrete Columns and geogrids, resulting in the so-called load transfer platform design. A geogrid reinforced embankment, of 1.5 m total thickness, was employed to transfer the highway loads via arching to a grid of VCCs with a gravel base, which further transfer the loads through the weak alluvial soils to the underlying sands. The design concept of this structure was to enhance the natural arching of granular material by incorporating two layers of geogrid reinforcement. Presented are considerations leading to application of this solution, details of the design and construction and precontract field investigations. The design eliminated the time uncertainty with regard to the time for consolidation of organic deposits. The embankment has performed so far as anticipated, with no evidence of movement or differential settlement. The applied method of construction offers an economic solution to the common problem of constructing in a short time embankments on soft ground where no significant settlements can be tolerated.

1 INTRODUCTION

A new carriageway in Gdańsk, completed in 1995, comprises a dual three lane urban highway and a tramway running within the central reserve. It provides a strategic east-west link of the city transportation system with the main road to Warsaw.

Over a distance of about 500 m (Fig. 1) this carriageway had to be constructed on poor ground conditions. The subsoil was especially bad along 173 m long section (A), where deep deposits of highly compressible soils existed. To control settlement, most of the carriageway dead and live loading had to be transferred down to the underlying sands. The general philosophy of the design applied for this section (Topolnicki, 1994) was to employ a geogrid-reinforced mattress to stimulate arching and load concentration on Vibro Concrete Columns (VCCs), which further transfer the loads through weak alluvial soils to the bearing sands.

The adjoining section (B) of the carriageway, some 330 m long, could be founded in a direct way because there were more favourable ground condi-

tions below the surface fill (cf. Fig. 2). Along this section two layers of geogrid, embedded in a well graded granular fill, were used to provide a sufficient base support for the carriageway (Werno et al., 1994). It has been anticipated, however, that section (B) may settle more than section (A). Therefore a transition zone had to be constructed to eliminate an abrupt change of the subgrade flexibility at the joint of two different foundation systems along the same carriageway. The transition zone was designed at the end of section (A) by means of gradual shortening of VCCs and appropriate modification of fill reinforcement (Topolnicki, 1994).

2 GROUND CONDITIONS

Ground conditions were investigated by means of boreholes, dynamic soundings and laboratory tests (Geoprojekt 1989), trial embankment (Werno et al. 1993) and georadar investigations (NGI&Geostab 1994). A general geological cross-section of the subsoil along the central axis of the carriageway is

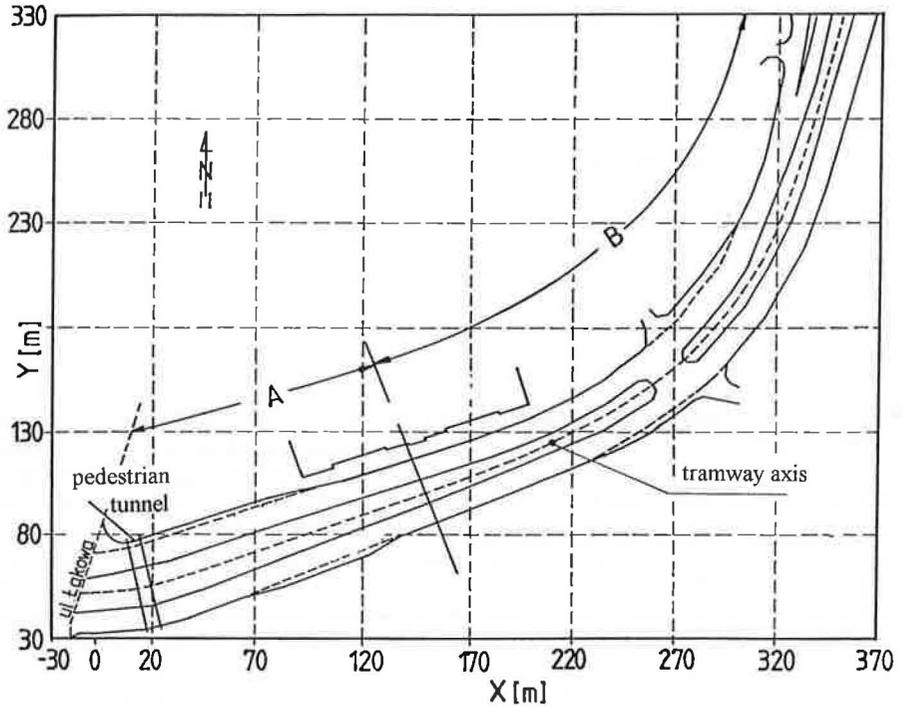


Fig. 1 Layout of the 500 m long section of the east-west carriageway in Gdańsk (section A: geogrid-reinforced embankment supported on Vibro Concrete Columns, ca. 173 m long, including 10 m for the transition zone; section B: geogrid-reinforced subgrade placed on existing fill, ca. 330 m long)

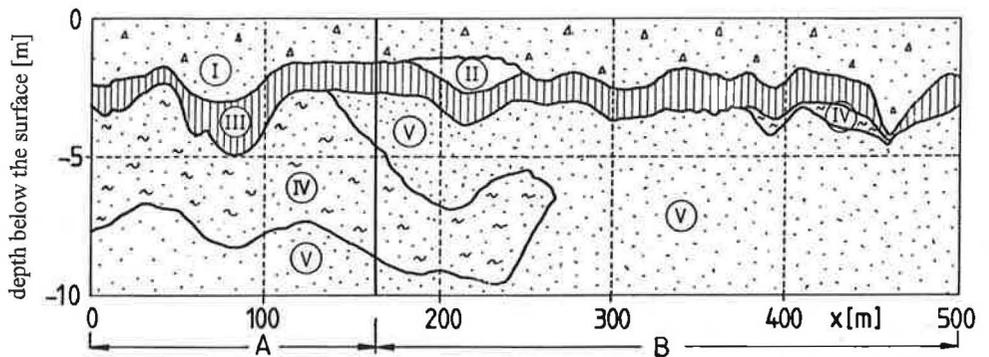


Fig. 2 Simplified geological cross-section of the subsoil along the central axis of the carriageway (Werno, 1994). Description: I - uncontrolled fill, II - sandy clay, III - peat, IV - organic clay, V - sand.

illustrated in Fig. 2. The partially loose fill (I) consists of sand, gravel, brick and concrete debris and humus. It has a variable thickness between 1.6 and 4 m and covers the whole route alignment. Below the fill there is a continuous layer of slightly decomposed

wet peat (III), 0.5 to 2.5 m thick, occasionally overlaid by medium stiff to stiff pockets of sandy clay (II). Below there is plastic organic clay layer (IV) of thickness between 3.3 and 5 m, overlying a basal layer of fine and medium sand (V) containing in

upper parts organic matters (especially in section B). The organic clay is discontinuous and inhomogeneous, mixed with silt, sand and peat. It underlies almost the whole peat stratum in section (A). Thus peat and organic clay build together a thick deposit of very compressible soils. Geotechnical parameters of both organic soils are summarized in Tab. 1. Groundwater level was found to vary between 0.8 and 2.8 m below the surface, depending on the season.

Tab. 1 Main geotechnical parameters of compressible layers (Geoprojekt, 1989)

Soil	γ_0 kN/m ³	w_{mean} %	N %	M_0 MPa
peat	10.1-12.4 (11.5)	258	29-77	0.3 - 0.98 (0.58)
organic clay	13.2 -17.8 (15.8)	63	5-15	0.5 - 3.8 (1.8)

Soil	τ_{max} kPa	ϕ_u deg.	c_u kPa	k_{10} [m/s] from - to
peat	33-49 (19)	10.5	5.7	$3.8 \cdot 10^{-9}$ $2.3 \cdot 10^{-7}$
organic clay	33-63 (36)	9.2	9.2	$1.6 \cdot 10^{-8}$ $2.7 \cdot 10^{-8}$

where: γ_0 - bulk density, w - water content, N - loss on ignition, M_0 - oedometric modulus in the stress range between 0.1 and 0.2 MPa, τ - maximum vane shear strength, ϕ_u - undrained friction angle, c_u - undrained cohesion, k_{10} - coeff. of permeability at 10⁰ C, () - mean value, respectively.

A trial embankment of base area ca. 40x54m and 3 m tall, constructed at section (A) above a 5.5 m thick organic deposit covered by 2.5 m thick fill, indicated high compressibility potential of peat and organic clay strata. The settlements recorded at the ground surface within one year after construction ranged from 28 to 44 cm, with no indication of stabilization (Werno et al. 1993). Owing to these observations deep ground treatment was recommended for section (A) to eliminate primary settlements and to reduce long term settlements and differential movements to acceptable levels.

In the next step extensive georadar investigations were additionally performed by NGI and Geostab (1994) to determine the necessary length of section (A). The obtained three-dimensional picture of the upper subsoil stratification allowed final shortening

of section (A) to 173 m, including 10 m for the transition zone.

3 DESIGN AND CONSTRUCTION OF FOUNDATION SUPPORT SYSTEM ALONG SECTION (A)

The experience of the piled embankment gained from applications in Malaysia (Ooi et al. 1987) and few recent applications in the UK (e.g. Bell et al. 1994, Card and Carter 1995) were used as guides in the present design.

The general philosophy of the considered foundation support system is to transfer most of the dead and live loads via arching to a grid of piles or columns, which further transfer the loads through weak deposits to a bearing stratum. To enhance the natural arching, which is insufficient in low embankments, layers of stiff geogrid reinforcement are placed within the granular material at appropriate vertical spacing above the piles. Due to a strong interlocking interaction of a well graded angular granular material with the geogrids a flexurally stiff load transfer platform is build which enhances soil arching. The thickness of this reinforced mattress and the spacing of piles are selected so that uniform support is provided at the top of the mattress. The angle of loadspread in a geogrid reinforced cohesionless soil can conservatively be taken as 45 degrees (Guido et al. 1987). Thus the geogrids are designed to carry only the weight of the granular material in the conical zone below the arch. The mechanical interlocking developed between the grid and the aggregate also provides effective separation between the granular layer and the weak subsoil whilst allowing natural movement of groundwater at the interface.

The outlined design philosophy has been adopted for the section (A) and resulted in the design shown in Figs. 3 and 5. The Vibro Concrete Columns, typically 5.7 to 6.5 m in length and 450 kN working load, were executed from the existing surface with a mean elevation of about +1.2 m above the sea level. The arrangement followed generally a grid of equilateral triangles with a side length of 2.1 m, corresponding to the area of 3.8 m² per one column. A the part where the embankment was underlied by a thicker fill layer the spacing was 2.28 m, equivalent to the area of 4.5 m². The VCC construction technique (cf. Keller 1990) employed an electric bottom feed vibrator which penetrated into the bearing sand stratum where a gravel plug was firstly made under air pressure and vibro-compacting action

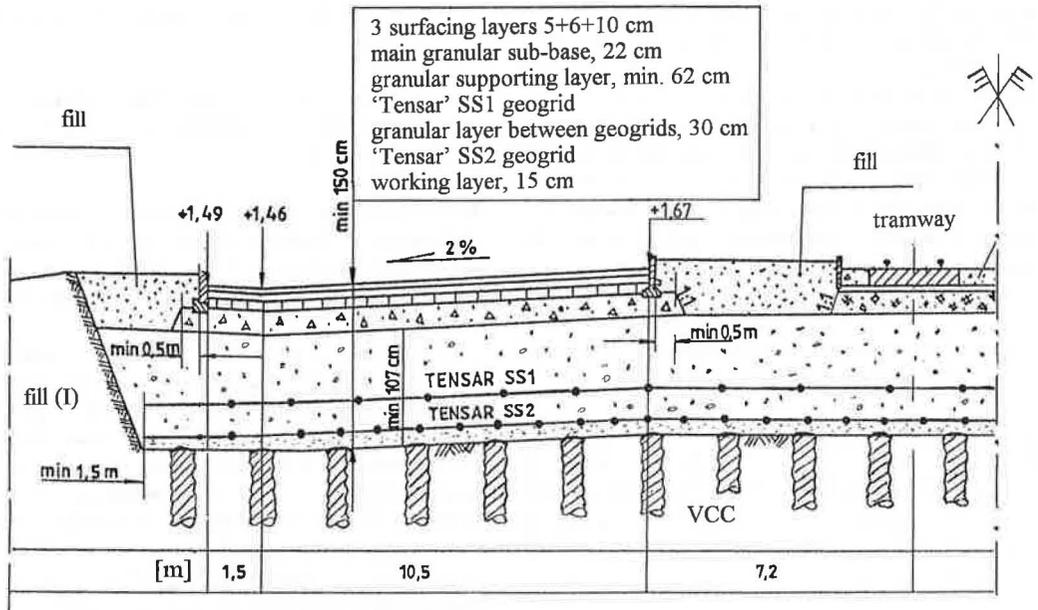


Fig. 3 Layout of the applied foundation support system in section (A)



Fig. 4 The right side of the carriageway after installation of Vibro Concrete Columns and removal of the existing fill

of the poker. This provided high load bearing capacity in spite of a relatively small penetration of the poker into the sand layer. The poker was subsequently filled with a low slump concrete and gradually raised and partly pushed down to form a vibro-compacted concrete column throughout the weak strata, up to the depth of about 1.5 m below the final pavement level. After few days the existing fill was removed and too high VCCs were carefully trimmed to the required level (Fig. 4). Along the whole section (A) a total of about 12500 running meters of VCCs were installed beneath the embankment.

Before placing the first geogrid, covering an area of about 6700 m², a 15 cm thick layer of compacted medium to coarse sand was laid over the VCC caps to provide a working layer and to prevent the lower geogrid from mechanical damage above the concrete caps. The load transfer platform was constructed using two layers of 'Tensar' geogrids, placed at a distance of 30 cm within a typical sub-base of 63 mm maximum size well graded material. For the lower layer 'Tensar' SS2 and for the upper layer 'Tensar' SS1 were used. The grids were laid in orthogonal

directions, with the ribs being parallel to the road centre line. The adjacent rolls were joint with a simple overlap of 50 cm. The upper grid was designed to carry the weight of fill under the assumed arch, which is a pyramid with a base dimension of 60 cm and a height of 30 cm. For both geogrids a performance limit strain of 5% was accepted. The foundation soil was assumed to be capable of carrying the initial 15 cm of fill below the first grid layer. Thus the design height of the load transfer platform was 75 cm (i.e. 15+2*30cm), enabling a maximum gap spacing of 150 cm between two adjacent columns.

In this kind of foundation support system the embankment height plays an important role in the design. It has to ensure that the arch within the load transfer platform is loaded and stable and has to prevent reflection of the column heads at the top surface. In the present case the total thickness of the embankment had to be not less than 150 cm, including roadbase and surfacing construction.

A cross-section through the transition zone linking two different foundation support designs along the same carriageway is shown in Fig. 5.

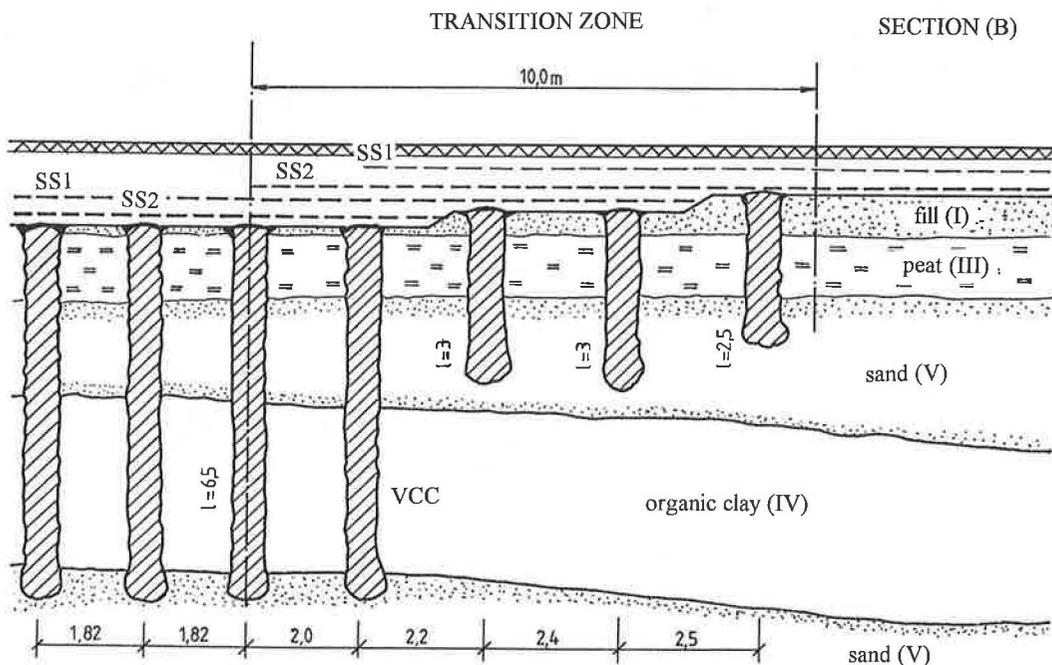


Fig. 5 Design of the transition zone at the end of section (A)

In the transition zone of 10 m length the VCCs were made gradually shorter to allow for smooth change between two foundation support systems. The layers of geogrid reinforcement, used in basic designs for section (A) and (B), were extended to the transition zone and overlapped to provide additional strengthening to the granular mattress.

4 PRECONTRACT ASSURANCE TESTING

Assurance field investigations were conducted to check the bearing capacity of VCCs and deformation of the embankment surface in relation to column spacing. At a selected testing site in section (A) six 'normal' VCCs and one 'combined'

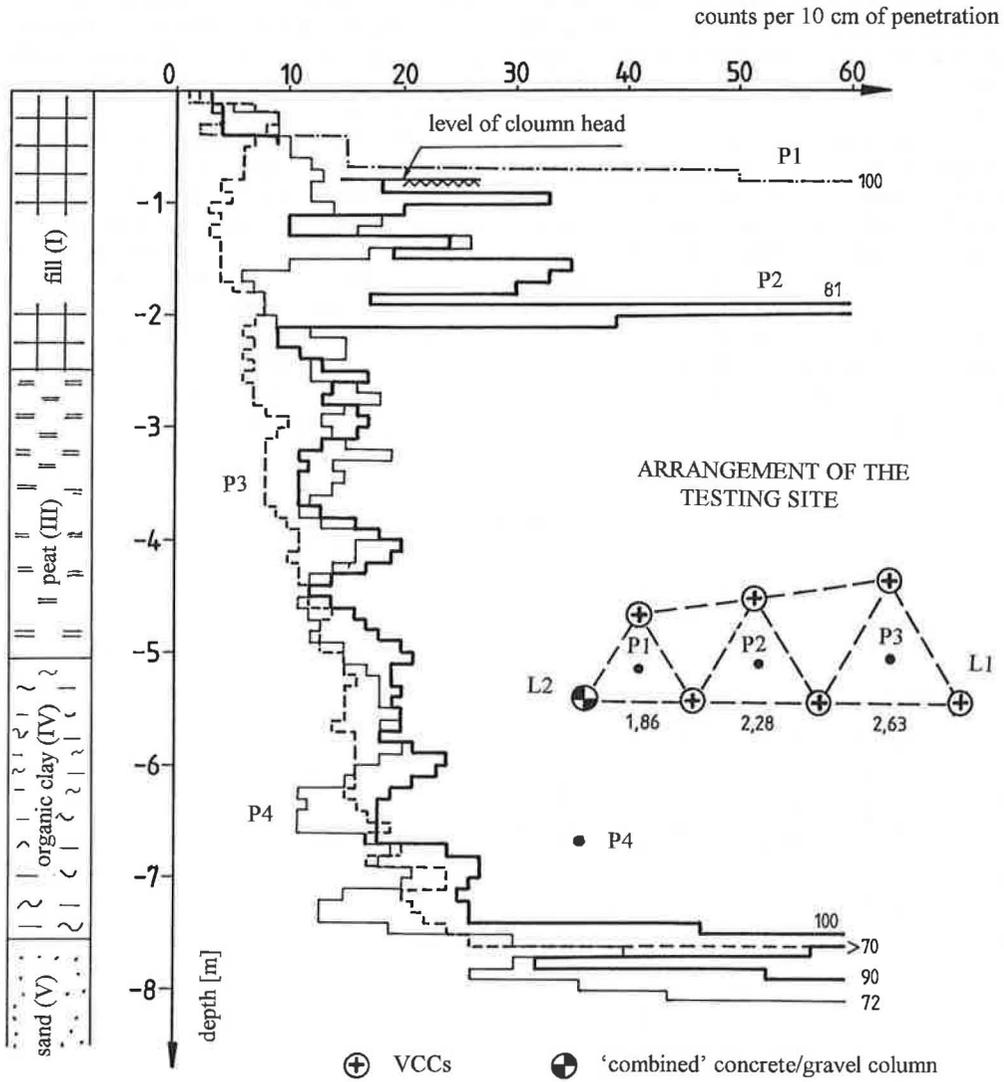


Fig. 6 Dynamic penetration tests conducted at a testing site using light ITB-ZW sounding probe (P1, P2, P3, P4 - locations of dynamic penetration tests L1, L2, P2 - locations of static loading tests)

vibro concrete/gravel column were installed following the arrangement shown in Fig. 6. Four dynamic penetration tests were subsequently performed at the locations P1 to P4 to investigate the effect of VCCs construction technique on the subsoil. It has been found that there was a strong strengthening effect in the fill and in sand layer but almost no improvement in peat and organic clay. Consequently, if only gravel would have been used instead of concrete, as it is the case with a classical vibroreplacement technique, there could be insufficient horizontal soil reaction along the columns. This in turn could lead to excessive deformations of the embankment.

At the testing site three static loading tests were

conducted at the locations L1, L2 and P2 shown in Fig. 6. The column at L1 was a 'normal' VCC, equipped with a concrete cap at the ground surface to accommodate a hydraulic jack. At L2 special 'combined' column was constructed, with a gravel part within the 2.5 m thick fill layer above the concrete shaft. The load on this column was applied through a concrete plate 90×90×23 cm, casted at the ground surface. After completion of tests L1 and L2 the ground surface at the testing site was lowered 1.5 m and all columns were trimmed at the bottom of excavation. The geogrid reinforced embankment was subsequently constructed following the design outlined in section 3, but using 'Tensar' SS2 for both grid layers.

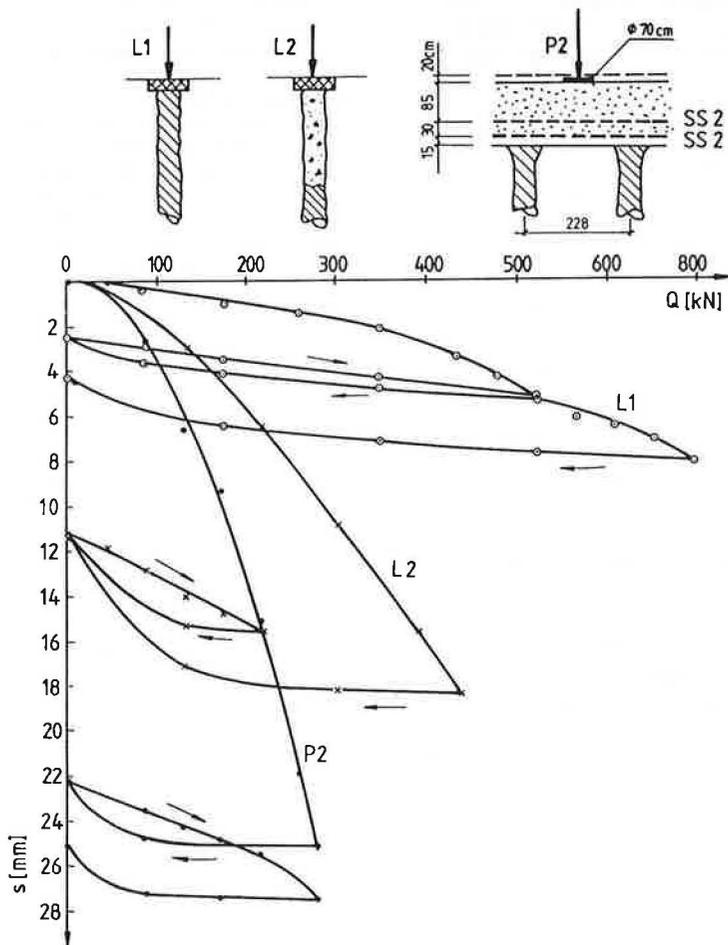


Fig. 7 Load against settlement relationships of all static loading tests

The third loading test was conducted at the midpoint P2 between the columns. The load was applied by means of a hydraulic jack placed on a steel plate of 70 cm diameter, embedded 20 cm below the ground surface. The position and dimension of the plate were determined to simulate stress distribution below a design wheel acting at the final road surface.

The results of all loading tests are depicted in Fig. 7. The load against settlement relationships include first loading cycle and a second cycle following full unloading. The admissible vertical load for a 'normal' VCC was estimated to be 450 kN at 3 mm displacement. Curves L2 and P2 demonstrate the settlement above and between the columns supporting the embankment. Within the range of static working loads applied by a single wheel, assumed not to exceed 55 kN, the total settlement at P2 is less than 0.8 mm. The expected differential settlements between L2 and P2 after few cycles of load application should not be greater than the accepted elastic deformation of the road surface, equal 0.7 mm.

5 CONCLUSIONS

The presented foundation support system provided an effective solution for construction of the carriageway over highly compressible peat and organic clay deposits with negligible risk to time schedule of the contract.

The philosophy of load transfer platform design applies to symmetrical or nearly symmetrical loading of VCCs. The reinforcement mechanism is based on the interlock between granular material and the geogrid, while a large portion of soil particles must be able to penetrate through the grid aperture. To achieve the assumed angle of loadspread within the granular mattress a well graded angular granular material is required.

The most important criteria are the grid spacing and the minimum embankment height so that the possibility of reflection of the column positions is minimized. For particular applications an optimization study of the construction cost should be performed with respect to the most effective relation between column spacing, embankment height and the number of reinforcing layers.

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