Foundation of an immersed tunnel on marine clay improved by cement deep mixing and sand compaction piles

Amélioration des vases de fondation d'un tunnel immergé par "CDM" et "SCP"

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ABSTRACT
A 3.2 km long immersed road tunnel forms part of the Busan-Geoje Fixed Link in South Korea, which connects Koreas second largest city Busan with the island of Geoje. To ensure the water tightness of the joints of the segmental tunnel structure and to limit the forces in the shear keys, the differential settlements have to be limited. This is achieved by extensive soil improvement of the marine clay below the tunnel with cement deep mixing (CDM) and sand compaction piles (SCP). This paper is concerned with the characteristics of the marine clay, the basic design principles, layout and constructional issues of the soil improvement as well as the numerical settlement and stability analyses.

RÉSUMÉ
Un tunnel routier long de 3.2 km fait partie du "Busan-Geoje Fixed Link", en CORÉE DU SUD, qui connecte la deuxième ville du Pays, Busan à l'île de Geoje. Afin d'assurer l'étanchéité des joints et de limiter les efforts dans les clés de cisaillement entre les caissons constituant les éléments de tunnel, les tassements différentiels doivent être minimiser. Cela est obtenu par un vaste programme d'amélioration de sol des vases sous-jacentes au moyen des techniques d'amélioration par "Mixage Profond de Ciment" (CDM, pour Cement Deep mixing) et par "Colonnes Ballastées de Sable" (SCP, pour Sand Compaction Piles). Le présent papier s'attache aux caractéristiques de la vase, aux principes de base de la conception, au plan d'implantation et aux problèmes de réalisation de la consolidation projetée, ainsi qu'aux analyses numériques de tassement et de stabilité.

Keywords : immersed tunnel, foundation design, soil improvement, cement deep mixing, sand compaction piles, marine clay

1 INTRODUCTION

The tunnel of the Busan-Geoje Fixed Link (Odgaard et al. 2006) consists of two short cut & cover tunnel access ramps and a 3.2 km long immersed tunnel (Figure 1). The 180 m long concrete tunnel elements of the immersed tunnel have a 26.5 m wide and 10 m high rectangular cross-section and are cast in a dry dock on shore. They are floated out to the sea, immersed to their final positions on a gravel bed in pre-excavated trenches in water depths of up to 50 m and connected to each other. After immersion, the tunnel is protected by backfill material at the sides and on top. Each tunnel element consists of eight 22.5 m long segments. The joints between the segments and between the tunnel elements are equipped with rubber gaskets and concrete shear keys. The displacement behaviour of the segmental tunnel structure in varying ground conditions represents a complex soil-structure interaction problem. In order to ensure the water tightness of the joints and to limit the forces in the shear keys, the differential longitudinal settlements have to be limited. Without soil improvement of the marine clay below the tunnel, long-term tunnel settlements of up to 35 cm and too large differential settlements were predicted by calculations in an early stage of the project. It was therefore decided to carry out extensive soil improvement with cement deep mixing (CDM) and sand compaction piles (SCP). At a length of 2.2 km, where the tunnel is placed in a trench, the clay below has a depth of up to 15 m and is improved with partial depth CDM columns (Figure 2). The tunnel alignment also requires a shorter underwater embankment in the western part, where the clay below has a depth of up to 30 m and is improved with full-depth SCP (Figure 3).

2 GROUND CONDITIONS

As the soft marine clay requires special attention and necessitates special foundation solutions, extensive ground investigations and laboratory testing programmes have been carried out (Steenfelt et al. 2008). The field tests comprised 50 boreholes with soil sampling, field vane and SPT tests and 60 CPT soundings with pore pressure measurements. Classification tests have shown that the plasticity of the clay is high to extremely high. The range of the liquid limit is 68 - 117 % (average 96 %) and the range of the plasticity index is 30 - 85 % (average 63 %). The saturated unit weight varies between 11.5 and 16.2 kN/m^3 (average 14.7 kN/m^3) and the void ratio is

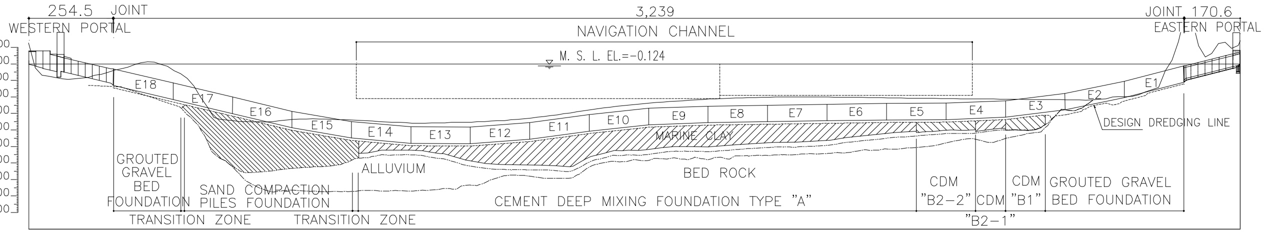

Figure 1. Longitudinal section of the tunnel.

Kasper, T., Jackson, P. G., Rotwitt, C. & Massarsch, K. R. 2009. Foundation of an Immersed Tunnel on Marine Clay Improved by Deep Mixing and Sand Compaction Piles 17th International Conference on Soil Mechanics and Geotechnical Engineering, Alexandria, Egypt. Vol. 3, pp. 2415-2418.

between 1.99 and 3.24 (average 2.44). The strength properties of the clay were assessed from the results of CPT and triaxial tests. Based on the SHANSEP approach, the undrained strength can be described as $c_u = 0.26 \cdot \sigma'_{v0}((\sigma'_{v0}+\Delta\sigma)/\sigma'_{v0})^{0.76}$ with the preconsolidation stress $\Delta\sigma$ of the marine clay typically ranging between 10 and 30 kPa (max. 65 kPa). The effective strength parameters $\varphi' = 25^\circ$ and $c' = 3$ kPa were derived from consolidated undrained triaxial tests and agree with the general experience from the area. Oedometer tests have shown ranges of the primary compression index C_c and recompression index C_{rc} of 0.83-1.82 (average 1.25) and 0.041-0.133 (average 0.091), respectively. The hydraulic permeability of the clay is in the order of 10^{-9} m/s.

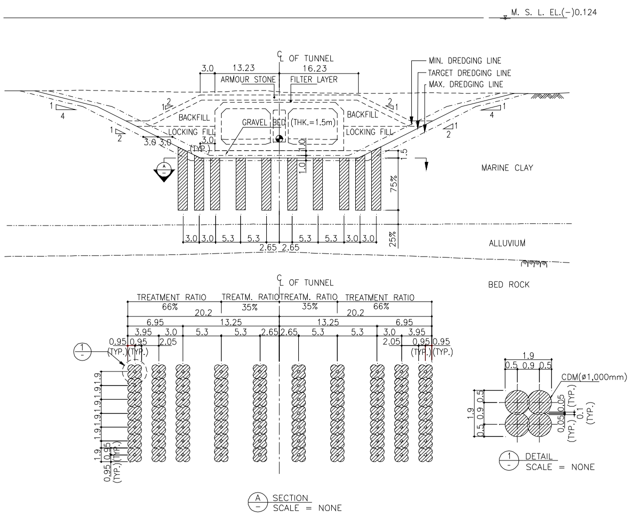

Figure 2. Typical CDM foundation layout at tunnel elements E3 to E14.

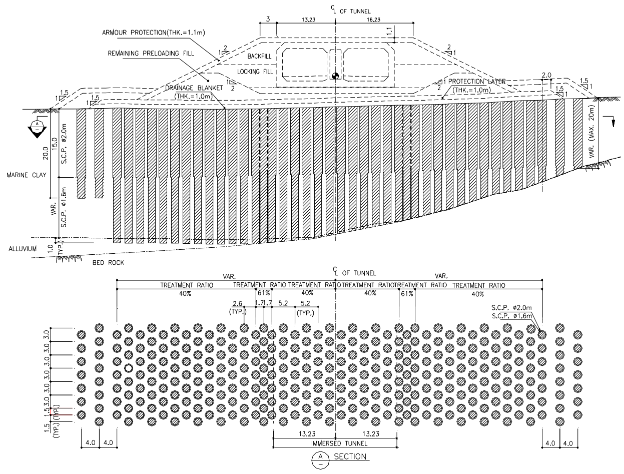

Figure 3. Typical SCP foundation layout at tunnel elements E15 to E17.

3 FOUNDATION SOLUTIONS

The conventional foundation solution for immersed tunnels is a simple foundation with a gravel bed placed on the underlying soil. However, this foundation concept was found to be insufficiently robust mainly due to the sensitivity to changes in load caused by construction tolerances and due to creep related settlements, which are difficult to predict. To make the foundation more robust, soil improvement with cement deep mixing (CDM) and sand compaction piles (SCP) was chosen. The reason for using SCP in the embankment section is to allow quick drainage and thus to achieve quick strength gain of the marine clay. This avoids stability problems of the embankment and allows sufficiently quick embankment construction between typhoon seasons.

3.1 *Cement deep mixing (CDM)*

Where the tunnel is placed in a trench (from tunnel element E3 to E14, cp. Figure 1) the clay is strengthened with walls formed by contiguous columns of mixed cement and clay (Figure 2). This ground treatment concept has previously been used in Korea and is in wide-spread use in Japan (CDIT 1999) and in northern Europe for settlement control of structures founded on soft clay deposits. The CDM columns are constructed using the wet mixing method, where cement slurry is mixed with the clay. The mixing shafts shown in Figure 4 are first drilled with the rotating blades into the soil to the desired depth. The cement slurry is injected in the withdrawal stage, where the vertical speed of the machine and the flow rate of the cement slurry are kept constant. Four 1 m diameter columns with overlaps of 10 cm form a 1.9 m square column pattern. To form continuous walls, these square columns are placed with 10 cm overlaps in rows (Figure 2). The CDM walls reach depths of up to 60 m below sea level (cp. Figure 1). The heaved soil in the trench due to CDM construction is dredged away before tunnel construction takes place.

Figure 4. Mixing blades of the deep mixing machine and offshore CDM production barge.

The equivalent unit weight of the tunnel is only some 12 % higher than water. The treatment of the clay is concentrated where the loads are largest. Thus a treatment ratio of 35 % is used beneath the centre of the tunnel, which is less than the treatment ratio of 66 % beneath the backfill and the edges of the tunnel. The chosen partial depth treatment of the marine clay enables a load share between the soil and the CDM columns. The major part of the foundation loads is transferred to a depth where the stiffness and strength properties of the marine clay are significantly better. An important advantage of the load-sharing concept is that the contact pressure below the foundation can be controlled, thereby assuring shearing resistance in the case of lateral static, dynamic or cyclic loading of the tunnel. Another benefit is that the foundation system becomes “self-adjusting”, whereby a gradual transfer of load between the clay and the stabilizing elements takes place until equilibrium is achieved. The depth of treatment can be adapted along the tunnel alignment to accommodate for differences in geotechnical conditions and specific requirements at transitions between the different foundation types. A full-depth solution would induce a high risk of uneven load distribution and stress peaks in the CDM walls, which would in turn lead to undesired local stress concentrations in the bottom slab of the tunnel.

3.2 *Sand compaction piles (SCP)*

Sand compaction piles (Kitazume 2005) are used where the tunnel is constructed on an embankment (from tunnel element E15 to E17 according to Figure 1). Sand compaction piles have been chosen in order to allow quick consolidation and strength gain of the clay and thus sufficiently quick embankment construction between typhoon seasons, and to introduce a favourable ductile behaviour of the foundation under earthquake excitation. The risks of embankment construction on very soft marine clay are illustrated by Chung et al. (2006), who discuss the failure of a breakwater close to the construction site of the immersed tunnel.

Design and optimization criteria of the SCP foundation are

- Sufficient preloading of the SCP by a preloading embankment
- Sufficient safety against failure of the preloading embankment during construction
- Sufficient safety against failure in the operational phase considering the effect of wave and earthquake impact
- Control of the tunnel settlements
- Limitation of the irrecoverable displacements of the embankment / tunnel due to earthquake impact.

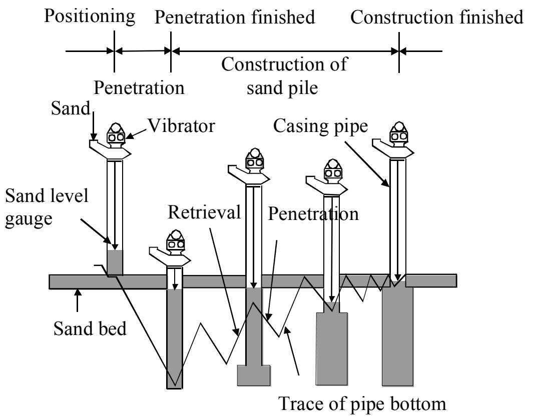

Figure 5. SCP construction procedure according to Kitazume (2005).

As illustrated in Figure 5, casing pipes are driven into the ground by vertical vibratory excitation from the vibro-hammer mounted on top of the casing. During penetration, the casing is filled with sand. After reaching the prescribed depth, the casing pipe is pulled about 1 m to feed the sand into the ground with the help of compressed air. The sand is compacted by the vertical vibratory excitation and the degree of compaction is controlled such that the required diameter of the sand pile is achieved. This procedure continues until the sand piles are completed. The depth of the casing pipes and the sand levels in the casing pipes are continuously monitored. The SCP have a diameter of 2 m and reach depths of up to 65 m below sea level (cp. Figure 1). The chosen treatment ratio is 40 %. However, the treatment ratio was increased to 61 % below the edges of the tunnel to provide support against downdrag resulting from side friction caused by placement of the fill material.

4 MODELLING

The self-weight of the tunnel and backfill and the downdrag resulting from friction between the backfill and the walls of the tunnel represent the permanent, settlement inducing loads. One of the reasons why a tunnel forms part of the Fixed Link is to accommodate heavy ship traffic to a large nearby harbour. Sunken ship impact has been considered as a load of 30 MN applied uniformly over one tunnel segment, i.e. 30 / 22.5 = 1.33 kN/m. The design was supported and optimized by a large number of settlement predictions and stability analyses with elasto-plastic finite element calculations. Settlements and equivalent soil springs were determined by 2D plane strain analyses of tunnel cross-sections with the finite element program Plaxis.

Figure 6 and 7 show two examples of finite element models, one for the tunnel in the trench section on the left hand side and one for the tunnel at the highest embankment section on the right hand side. The base of the models is assumed to be fixed in both directions, while the vertical model boundaries are fixed in horizontal direction. To account for the actual 3D geometry, the sand compaction piles are modelled with equivalent widths. Higher-order 15-node triangular elements are used to achieve a good numerical quality of the calculations. The marine clay and the sand compaction piles are modelled with an advanced elasto-plastic model called hardening soil model (Schanz et al. 1999). The parameters for the clay have been derived from the results of laboratory tests, while the parameters for the sand compaction piles are best estimates for dense sand. The material parameters are summarised in Table 1.

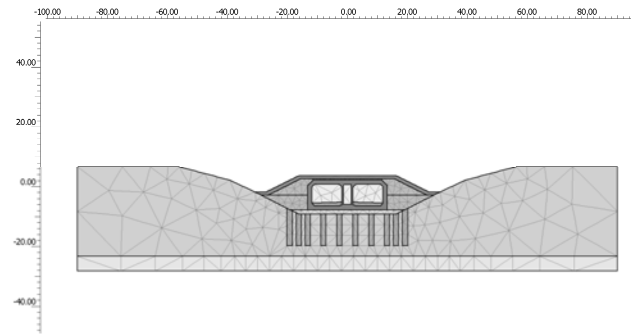

Figure 6. A typical FE model for the trench section with CDM foundation.

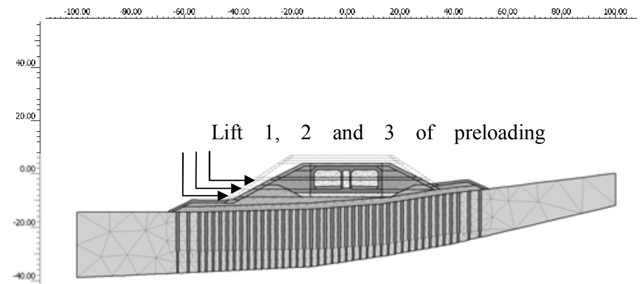

Figure 7. A typical FE model for the highest embankment section with SCP foundation.

Table 1. Material models and parameters.

Model	Marine clay HS	Alluvium MC	CDM LE	SCP HS	Gravel bed MC	Fill+Armour MC
γ (kN/m^3)	14.7	20	20	20	20	20
E_{50}^{ref} (MPa)	1.2	50	80	39	50	50
ν (-)	0.2	0.25	0.2	0.3	0.2	0.2
E_{oed}^{ref} (MPa)	0.6			33		
E_{ur}^{ref} (MPa)	8			120		
m (-)	1			0.5		
p^{ref} (kPa)	100			100		
c' (kPa)	3	0		0	0	0
φ' (°)	25	35		33	36	40
k (m/s)	$1{\cdot}10^{-9}$	$1{\cdot}10^{-5}$	$1{\cdot}10^{-9}$	$1{\cdot}10^{-4}$	$1{\cdot}10^{-1}$	$1{\cdot}10^{-1}$

HS - Hardening soil model, MC - Mohr-Coulomb, LE - Linear elastic

The construction sequence is modelled in several stages:

- Initial state. Initial stresses in the clay and alluvium are generated based on self-weight and Jaky's formula $K_0 = 1 - \sin \varphi'$. Pore pressures are calculated based on the sea level and the unit weight of water.
- In the trench section: Modelling of trench excavation by deactivation of the respective finite elements.
- To consider the installation of CDM and SCP, the material properties are changed in the respective finite elements. This simplified approach neglects the effect of CDM and SCP installation (remoulding and compaction) on stresses and consequently strength and is considered to be conservative.

- In the embankment section: Modelling of the preloading embankment construction in 4 phases (lift 1, lift 2, lift 3 and subsequent partial excavation) by activation and deactivation of the respective finite elements.
- Modelling of tunnel construction by activation of finite elements and change of material properties.

When elements are (re-)activated, their stresses are initialised to 0. Depending on the purpose of the analysis and the result of interest (e.g. instantaneous settlements, final settlements or development of settlements in time), the individual stages are modelled as undrained, drained or consolidation processes. The displacements are reset to 0 at the beginning of the last stage in order to separate the tunnel settlements from the settlements in the previous stages. Also in the last stage, interface elements representing a wall friction angle of $\delta = 2/3\ \varphi'$ are considered around the tunnel. Additional loading cases are calculated at the end to consider loads such as trench sedimentation, sunken ship or dragged anchor impact or to determine equivalent subgrade reaction moduli (load-displacement curves) of the tunnel foundation. Furthermore, strength reduction analyses (Brinkgreve & Bakker 1991) are carried out for the different stages to determine the corresponding safety factors.

A satisfying safety factor of 2.1 has been determined for the slope stability of the tunnel protection material in the trench section. The safety factor for the stability of the highest embankment section on SCP is 1.5 (undrained, just after tunnel construction) and 1.9 (in the final situation after full consolidation). The trench section is exposed to possible impact from large waves. The stability of the open trench during construction with consideration of wave impact has been investigated by Kasper & Jackson (2008), while the stability of the tunnel under wave impact after construction has been investigated by Kasper et al. (2008).

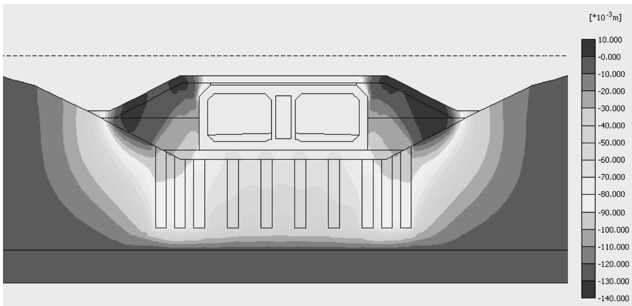

Figure 8. Example of settlements due to tunnel construction in the trench section with CDM foundation.

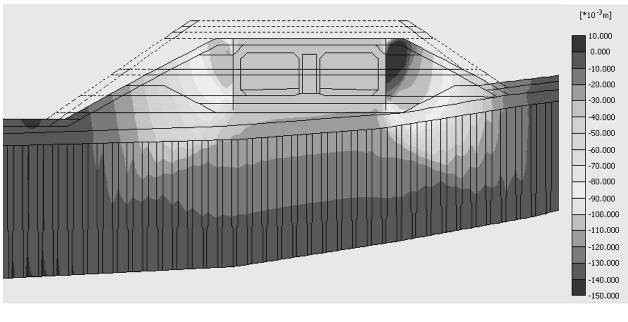

Figure 9. Example of settlements due to tunnel construction in the embankment section with SCP foundation.

The predicted average final tunnel settlements are in the order of 9 cm in the trench section and 5 cm in the embankment section (cp. examples in Figure 8 and Figure 9). Based on a large number of cross-sections and sensitivity studies, possible differential settlements were carefully evaluated and have been found acceptable. The finite element analyses allowed to optimize the CDM and SCP layout (i.e. position, number and depth of the reinforcing elements) and to investigate the effect of soil parameter and geometry variations. The analyses were used to verify the loading of the reinforcing elements and further allowed to optimize the geometry as well as construction sequence and periods of embankment preloading on SCP treated soil.

The equivalent subgrade reaction moduli of the tunnel foundation and their variations as obtained from the Plaxis calculations were used as input for soil springs in a 3D structural model of the tunnel. This model, which also considers the joints of the segmental tunnel structure, was used to verify that the behaviour of the tunnel and the resulting joint openings for different loading situations are satisfactory.

5 CONCLUSIONS

Too large total and differential settlements were predicted for the segmental structure of the immersed tunnel of the Busan-Geoje Fixed Link in case of a simple foundation on the marine clay. It was therefore decided to improve the clay with CDM and SCP. The soil improvement limits the tunnel settlements and provides the necessary robustness of the foundation. Numerical models were applied as a complement to traditional design methods and have been found to be a very useful tool for the design, optimization and verification of the chosen foundation solutions. The soil improvement was finished in 2007 and the first 5 elements were immersed in 2008.

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