THE DESIGN OF PARTIAL DEPTH CEMENT DEEP MIXING (CDM) FOUNDATION FOR AN IMMERSED TUNNEL

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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 Korea's 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 was achieved by extensive soil improvement of the marine clay below the tunnel with cement deep mixing (CDM) and sand compaction piles (SCP). This paper focuses on the characteristics of the marine clay, the basic design principles, layout and constructional issues of the CDM soil improvement, and numerical and numerical analyses to predict settlement and stability.

Keywords: cement deep mixing, sand compaction piles, foundation, immersed tunnel

INTRODUCTION

The Busan-Geoje Fixed Link consists of two short cut & cover tunnel access ramps, a 3.2 km long immersed tunnel (Figure 1) and two long span cable stayed bridges. The tunnel consists of eighteen concrete elements each of 180 m length having a rectangular cross section of 26.5 m in width and 10 m in height. The tunnel elements are cast in a dry dock on shore and floated out to the sea, immersed in their final positions on a gravel bed in pre-excavated trench and connected to each other by a dual gasket system. The water depth was up to 50 m making the tunnel of the deepest yet constructed. After immersion, the tunnel is protected by backfill material at the sides and rock armour 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 EDPM water stops, Omega seals and concrete shear keys. The longitudinal displacement behaviour of the segmental tunnel structure in the varying ground conditions represents a complex soil-structure interaction problem which has been modelled with a FEM structural model representing the tunnel, the joints and with springs to represent the soil that are derived from the PLAXIS soil/foundation models. In order to ensure the water tightness of the joints which have limited opening capacity and to limit the forces in the shear keys, differential longitudinal settlements have to be limited. This is achieved by limiting the variations in subgrade modulus along the alignment and it in practice means that differential settlements are limited to a few tens of millimetres. Without soil improvement of the marine clay below the tunnel, long-term tunnel settlements of up to 400 mm were predicted when taking into account the likely variations in the trench profile, backfilling and uncertainties in the soil properties. Such large total settlements led to large variations in the sub-grade reaction along the alignment, which in turn resulted in significant differential settlements, unacceptable joint opening and large shear key loading. It was therefore decided to carry out extensive soil improvement with cement deep mixing (CDM) (CDIT, 1999) and sand compaction piles (SCP) (Kitazume, 2005) after studying a number of foundation alternatives.

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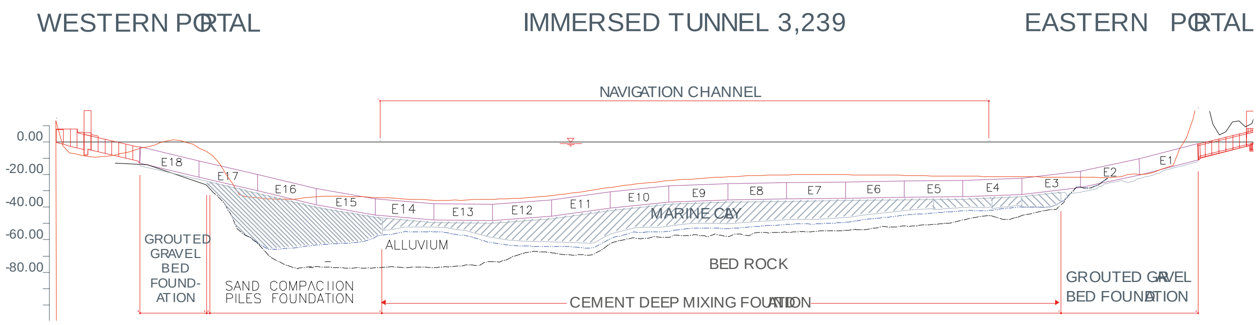

Figure 1 Longitudinal section of the tunnel

For the central part of the tunnel over 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 it was decided to improve the this with a partial depth CDM solution (Figure 1 & 2). The tunnel alignment also requires a short underwater embankment at the Western section of the alignment, where the clay below has a depth of up to 30 m, in this area ground improvement has been carried out with full-depth sand compaction pile treatment (SCP), the rational for the choice of the two different solutions is discussed below.

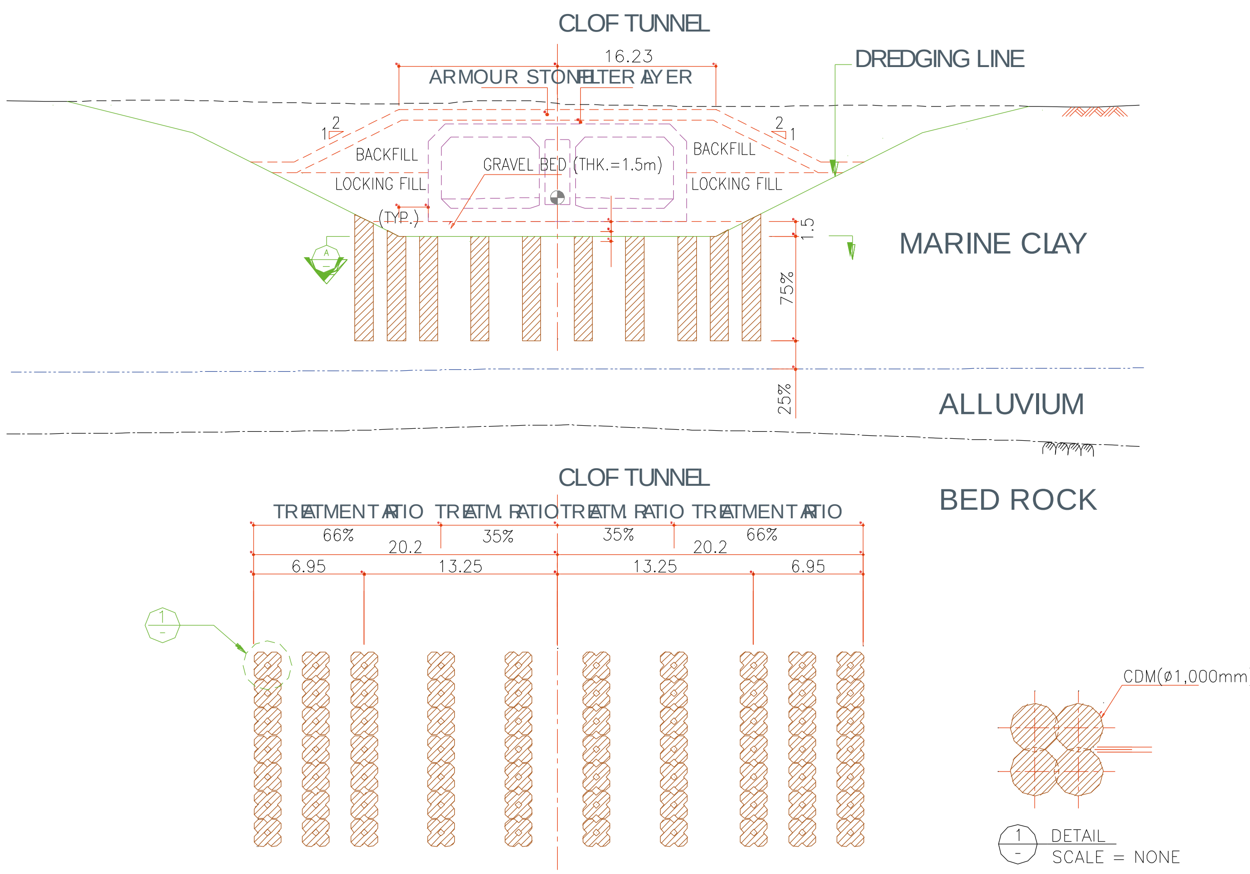

Figure 2 Section and plan view of typical layout of the CDM foundation for tunnel elements in a trench.

GROUND INVESTIGATIONS

The ground conditions along the tunnel alignment consisted of marine clay overlying a stiff alluvium which in turn overlies bedrock. The stratigraphy is illustrated on the long section (Figure 1) with the majority of the tunnel length being founded in a trench within the thick layer of marine clay. The marine clay is from the Holocene period and it is normally to slightly over consolidated. As the properties of the marine clay where of particular importance for the foundation and tunnel design, an extensive ground investigation and laboratory testing programmes was carried out (Steenfelt et. al., 2008) to investigate its properties. The field tests comprised 50 boreholes with soil sampling, field vane and SPT tests and 60 CPT soundings with pore pressure measurements. The results of the classification tests showed that the clay exhibits high to extremely high plasticity, the results are summarised below in Table 1.

Table 1 Classification Test Summary for Marine Clay

Property	Value
natural moisture content	89 %
liquid limit	96 %
sat unit Weight	14.7 kN/m^3
initial void ratio	2.4
Compression index Cc	1.25
Compression index Crc	0.091

The undrained strength properties and the stress history of the clay were assessed from the results of CPT and triaxial tests together with the SHANSEP ((Stress History And Normalized Soil Engineering Properties), (Ladd & Foott, 1974) as the most reliable information available was the CPTs and a small number of high quality triaxial tests. The undrained strength was derived after establishing a site specific cone factor (N_{kt}) from the triaxial tests and the undrained strength/stress relationship was determined to be:

$$c_u = 0.26\sigma'_{v0}((\sigma'_{v0}+\Delta\sigma)/\sigma'_{v0})0.76$$

With the preconsolidation stress $\Delta\sigma$ of the marine clay typically ranging between 10 and 30 kPa. The strength profile derived is illustrated in Figure 3.

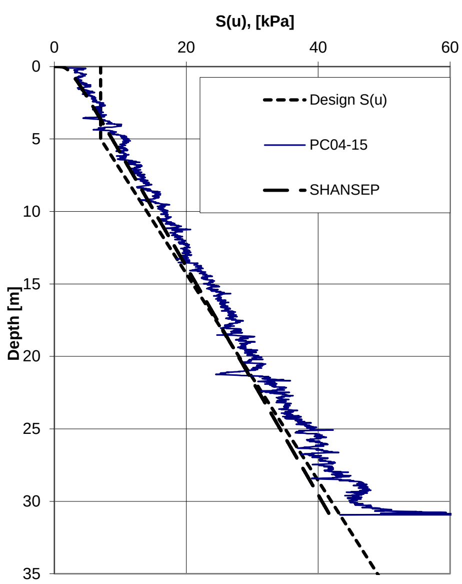

Figure 3 Undrained Strength Profile derived from CPT and triaxial tests using SHANSEP approach, CPT PC04-15 from the central section of alignment

The effective strength parameters φ' = 25º and c' = 3 kPa were derived from consolidated undrained triaxial tests and due consideration of 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.

FOUNDATION CONCEPTS

The normal foundation concept for immersed tunnels is a direct foundation on the underlying strata with a gravel bed or jetted sand foundation, as the foundation loading from the tunnel placed in a dredged trench is low, typically only a few kPa greater than the in-situ stress prior to dredging of the trench. This foundation concept was found to result in unacceptable settlements as while the predicted settlements for the main section of tunnel in trench without soil improvement were typically 100 mm with the theoretical trench profile. Once the tolerances in the trench profile which were considered, these were +/- 1 m vertically and +/- 3 m horizontally together with the consequent variations in the backfilling, the consolidation settlements increased significantly with 300 to 400 mm were predicted. Such settlements were considered to be larger than acceptable, as they could lead to unacceptable joint openings between the tunnel segments or between the tunnel elements. A number of possible foundation concepts were studied in order to reduce the settlements, these included founding the tunnel on piles, vertical drains together with preloading and soil improvement. Each of these concepts had advantages and disadvantages with practical difficulties considering that the foundation was to be installed at a maximum water depth of 47 m. From these studies a combination of soil improvement with cement deep mixing (CDM) for the main section of the tunnel in the dredged trench was chosen together with sand compaction piles (SCP) for the sub-sea embankment section. The rational for their selection is discussed below.
The CDM foundation was chosen for a number of reasons, these included the availability of equipment and the experience of local contractors with the technique and that it could, with a partial depth treatment and a relatively low treatment ratio provide a robust foundation solution cost effectively. The design and application of the CDM technique departed significantly from Japanese and Korean practice where full depth treatment is normally utilised with high strength columns providing an essentially rigid foundation solution with the loads transferred to the underlying stiff stratum. Usually a block is treated which is able to resist earthquake loading as well as service loads. The CDM solution adopted was a partial depth foundation, with the columns typically going to approximately two thirds of the depth of the compressible layer together with low strength columns as illustrated in Figure 2, with columns typically being 10 m in length to treat a layer of clay of 15 m thickness. This provided a flexible foundation solution that suited the inherently flexible immersed tunnel with its frequent joints. Such a partial depth solution is common in Sweden for the lime/cement column soil improvement of embankments and the concept is also used for piled raft foundations where it provides an efficient solution as both the columns/piles and the soil each take a share of the loading. The concept is described by Massarsch et al, (2007) and Fioravante et al (2005), amongst others. The design concept for the CDM is shown below in Figure 3 after (Allén, 2005) with the methods described by Allén used as a first assessment of the settlements which were later verified by FEM modelling.

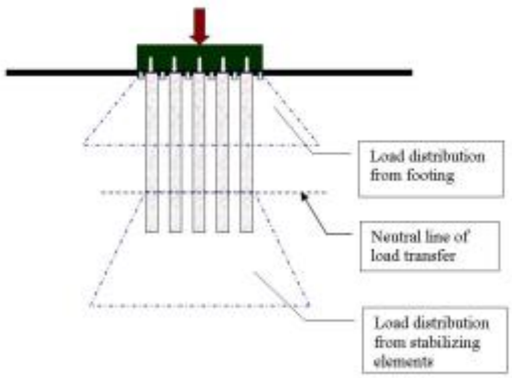

Figure 4 Floating Foundation Concept after Allén 2005

The reason for using SCP at the embankment section was that the SCP treated soil is ductile and it could deform under the high seismic loading that prevailed in the area and the inertia from the heavy embankment placed on top of the treated soil. In comparison, a CDM solution would have had to resist the seismic loading and to prevent progressive failure, it would have required a high treatment ratio and high strength columns. The SCP treatment however required a high treatment ratio due stability considerations during the embankment construction and preloading. The risks of embankment construction on the marine clay are illustrated in Chung et. al. (2006), where failure of a breakwater close to the construction site of the immersed tunnel is discussed.
The tunnel was founded on a gravel bed formed by berms placed in rows to a very high accuracy in order to form an even distribution of load on the tunnel. This necessitated the construction of a special jack up barge and tremmie pipe gravel placement system.

CEMENT DEEP MIXING (CDM) FOUNDATION

In the main section of the tunnel where the tunnel is placed in a trench (from tunnel element TE3 to TE14, shown in Figure 1) the clay was strengthened with walls formed by contiguous columns of wet mixed cement and clay, the column layout is illustrated in Figure 2. The CDM columns were constructed using the wet mixing method, where cement slurry is mixed with the clay, the mixing shafts shown in Figure 4 (left) are first drilled down 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 offshore CDM production is illustrated in Figure 4 (right). The CDM walls reach depths of up to 60 m below sea level (Figure 1) and some of the largest available CDM installation barges were used for the work.
One of the issues to be considered with many soil improvement techniques and also with CDM is that it results in a significant amount of heaved soil, and if left in place would have left a compressible layer above the top of the columns. This was removed by dredging after a series of trials to verify that the dredging would not damage the top of the columns.

Figure 5: Mixing blades of the deep mixing machine (left) and offshore CDM production barge (right).

The equivalent unit weight of the tunnel is only slightly greater than water (10.06 kN/m^3) and thus the loading from the tunnel at foundation level is low, typically 5 kPa. The major loading of the foundation comes from the backfill adjacent to the tunnel as this formed by rock fill covered by rock armour with a depth up to 12 m, this gives foundation loading of approximately 100 kPa adjacent to the tunnel. The treatment of the clay was therefore concentrated where the loads were largest with a low treatment ratio of 35 % used beneath the centre section of the tunnel and a larger treatment ratio of 66 % used beneath the backfill and the edges of the tunnel.

The chosen partial depth treatment of the marine clay enabled a load share between the soil and the CDM columns. With the major part of the foundation loads transferred to a lower level in the marine clay where its strength and stiffness properties 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 of load-sharing, where the foundation system is “self-adjusting”, with a gradual transfer of load between the foundation and the columns taking place until equilibrium is achieved. The depth of treatment can be adapted along the tunnel alignment to accommodate for differences in geotechnical conditions and to accommodate the 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 base slab of the tunnel.

The partial depth treatment resulted in quite low stresses in the columns, much less than would result from full depth columns with end bearing in the stiff alluvium below due to the load sharing. This enabled low strength columns to be used with a design unconfined compressive strength of 500 kPa compared to the normal Korean/Japanese CDM practice of strengths greater than 3 MPa. This allowed significant reductions in the amount of cement used in comparison to normal practice and the consequent savings in costs.

The quantity of treated soil, i.e. columns was approximately half a million cubic metres, illustrating that even small reductions in cement quantities could bring significant saving for the project. The avoidance of having the columns in "end bearing" also provided significant savings, as the column construction could proceed much more quickly without the need to penetrate the stiff alluvium and ensure "end bearing" was achieved.

DESIGN and MODELLING

The self-weight of the tunnel and backfill adjacent to the tunnel together with the downdrag resulting from friction between the backfill and the walls of the tunnel represent the permanent service load. In addition, accidental loads had also to be considered in the foundation design, this included a flooded tunnel and a sunken ship on top of the tunnel. The sunken ship load was the most onerous load, which was considered as a load of 30 MN applied uniformly over one tunnel segment, i.e. 30 / 22.5 = 1.33 MN/m of tunnel length.

In order to verify the foundation design and develop an optimised solution for the treatment ratio and column lengths a large number of settlement predictions and stability analyses with elasto-plastic finite element calculations were carried out. From these analyses settlements and equivalent soil springs for input into the structural models were determined from 2D plane strain analyses of the tunnel cross-sections with the finite element program PLAXIS.

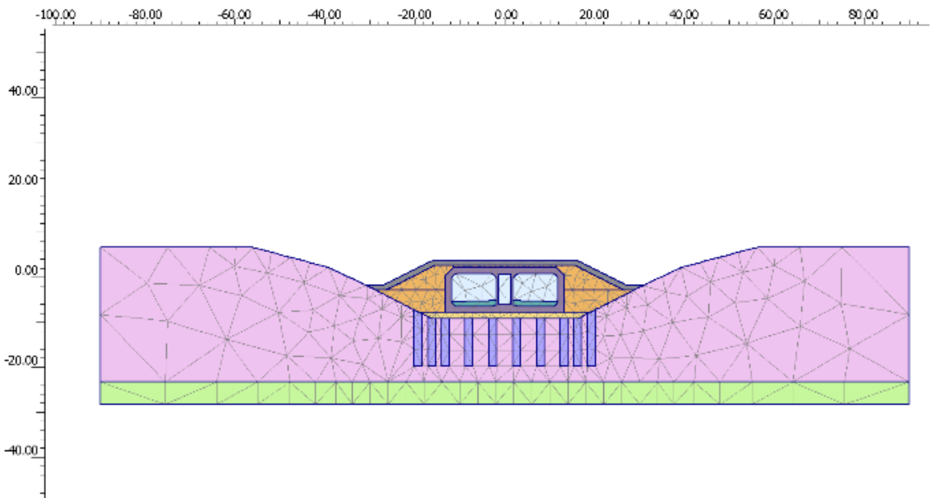

Figure 6 A typical FE model for the trench section with CDM foundation

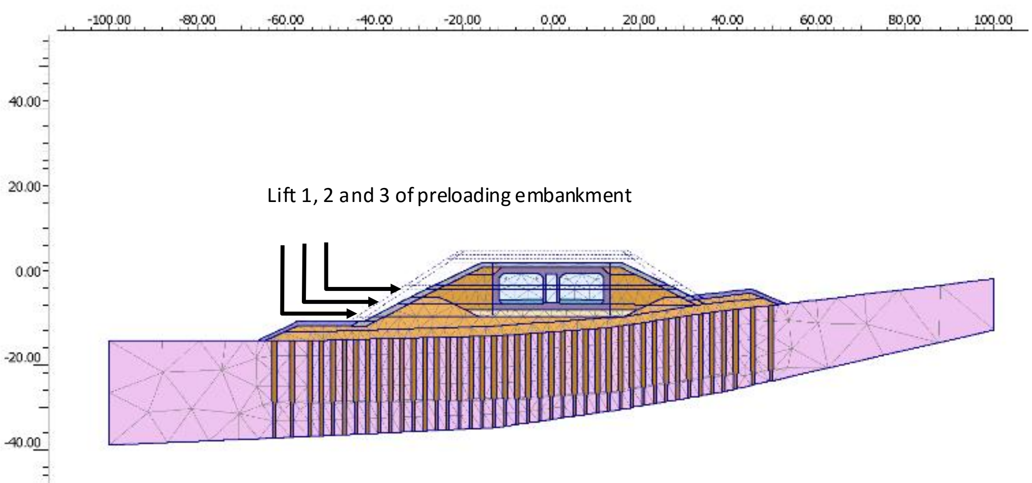

Figure 7 Typical Sections of SCP embankment

Two examples of the 2D finite element models are presented above, one for the tunnel in the trench section with a CDM foundation (Figure 6), and one for the tunnel at the highest embankment section on a SCP foundation (Figure 7). The base of the models are fixed in both directions, while the vertical model boundaries are fixed in the horizontal direction. To account for the actual 3D geometry, the CDM columns (walls) and sand compaction piles were modelled with their equivalent widths (per linear metre). Higher-order 15-node triangular elements were used to achieve a good quality of the results. The marine clay and the sand compaction piles are modelled with an advanced elasto-plastic model called the Hardening soil model (Schanz et. al. 2000). The parameters for the clay have been derived from the results of laboratory tests as discussed above, while the parameters for the CDM columns and are best estimates based on the literature and empirical relationships. Also for the for the backfill and rock armour the material parameters were based on empirical relationships and experience. The material parameters are summarised in Table 1.

Table 2: Material models and parameters.

	Marine clay	Alluvium	CDM Columns	SCP	Gravel bed	Fill + Armour
Material model	Hardening soil model	Mohr-Coulomb	Linear elastic	Hardening soil model	Mohr-Coulomb	Mohr-Coulomb
γ (kN/m^3)	14.7	20	20	20	20	20
E^{ref} (MPa)	1.2	50	80	39	50	50
ν (-)	0.2	0.25	0.2	0.3	0.2	0.2
E^{ref}_{oed} (MPa)	0.6			33		
E^{ref}_{ur} (MPa)	8			120		
m (-)	1			0.5		
p^{ref} (kPa)	100			100		
c' (kPa)	3	0		0	0	0
ϕ' (º)	25	35		33	36	40
k (m/s)	$1{\cdot}10^{-9}$	$1{\cdot}10^{-5}$	$1{\cdot}10^{-9}$	$1{\cdot}10^{-4}$	$1{\cdot}10^{-1}$	$1{\cdot}10^{-1}$

The construction sequence was 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 hydrostatic 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, with the subsequent relaxation of the clay allowed. Depending on the purpose of the model, this was either modelled as fully drained or a full consolidation model to allow the drainage time to be considered.
- To consider the installation of CDM and SCP, the material properties were 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. For the SCP this is considered to be conservative as the horizontal stress are increased during SCP installation but for the CDM this is probably not conservative.
- Stability was a significant aspect of the SCP design due to the loading from the embankment and preloading. For the CDM design, stability was not a concern as the trench stability was not a significant concern.
- Modelling of tunnel construction by activation of finite elements and change of material properties.

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 were modelled as undrained, drained or consolidation processes. The displacements were reset to zero at the start of the last stage after placement of the tunnel and backfill in order to determine the tunnel settlements during service. Also in the last stage after placing the tunnel and backfill, interface elements representing wall friction angles of $\delta = 2/3\ \varphi'$ were imposed around the tunnel to represent the downdrag from the backfill. Other load cases were also considered such as trench sedimentation, a sunken ship, the load from a dragged anchor impact and to determine equivalent subgrade reaction moduli (load-displacement curves) of the tunnel foundation for input into the structural models used for the tunnel structural design. Furthermore, strength reduction analyses (Brikgreve, 1991) were carried out for the different stages of the models to determine the corresponding safety factors. A required safety factor of 2.1 was used for the slope stability of the tunnel protection material in the trench section and 1.6 for the stability of the highest embankment section (undrained, just after tunnel construction). The trench section is exposed to the impact from large waves and the stability of this has been investigated in Kasper et.al. (2008) and Kasper et.al. (2008a). The predicted tunnel settlements are in the order of 100 mm in the trench section and 50 mm at the embankment section (shown in Figure 8). The finite element analyses allowed to the optimisation of the CDM and SCP layout (i.e. treatment ratio, number and depth of the reinforcing elements) and to investigate the effect of soil parameter and geometry variations. The analyses were used to verify that the loading of the reinforcing elements was acceptable and allowed the construction sequence of the preloading embankment on SCP to be optimised.

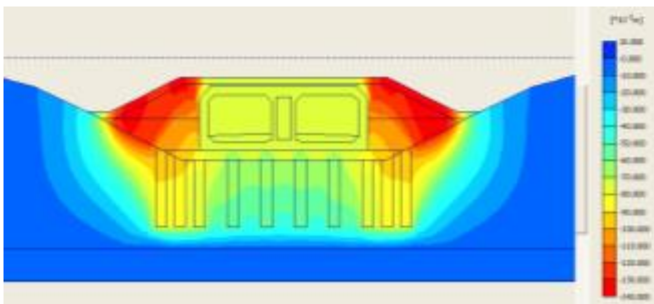

Figure 8: Example of settlements due to tunnel construction in the trench section for CDM foundation

The equivalent subgrade reaction moduli of the tunnel foundation and their variations were derived from the PLAXIS calculations and used as input for nonlinear soil springs in the structural models of the tunnel. This model, which also considers the joints of the segmental tunnel structure, has been used to verify that the behaviour of the tunnel and the resulting joint openings for different loading situations are satisfactory.

CONSTRUCTION and VERIFICATION

The installation of the approximately a half million cubic metres of CDM columns took approximately one year with two CDM barges. Prior to commencement of the column installation a considerable number of trials were carried out to verify the column strengths and their continuity. These included initial lab trials to determine the initial cement addition ratios and field trials with the intended CDM barge together with coring of the columns and SPTs to verify the strength of the installed columns. These trials took some time as the design strength (UCS) was 500 kPa, which was considerably lower than was normal practice in Korea and Japan. This allowed significantly lower cement addition rates to be used than was normal practice, however the use of lower than normal cement ratios took some time to satisfy the Construction Supervisor that good quality columns could be achieved.

The verification of the in-situ strength proved to be difficult as locating the top of the columns was difficult underwater and it was therefore difficult to judge where a borehole was located and what a particular core sample of SPT represented. There seems scope for the development of the in-situ verification process and in particular the use of CPTs may have proved useful.

Measurements of settlements were made throughout the construction period and for a period after construction, these showed that the settlements were well within the predictions. For the CDM foundation in a dredged trench, the predictions of settlement for the service state with a best estimate of the trench profile were approximately 60 mm, measurements up to two years after construction showed that the observed settlements were in the range of 30 mm to 50 mm. For the SCP embankment large settlements were observed during construction and preloading but these were well within the predictions. The final settlements after placing the tunnel were similar to the trench CDM foundation with approximately 50 mm being observed some two years after construction.

CONCLUSIONS

Total and differential settlements that were too large were predicted for the immersed tunnel when it was directly founded on a deep layer of marine clay. A number of foundation solutions were considered that included piling, vertical drains with pre-loading and soil improvement. The chosen soil improvement methods were cement deep mixing for the majority of the tunnel where the tunnel was in a trench and an SCP solution where it was located on a sub-sea embankment.

The Busan-Geoje Fixed link was a design and build (D&B) project as part of a concession agreement with the Preliminary and Detailed Design carried out by the Contractor's consultant COWI A/S based on a conceptual design by the Owner. The D&B project allowed the Contractor and designer the freedom to innovate and develop solutions that were not normally used in Korea. In particular it allowed the partial depth load share foundation concept that is frequently used in Sweden with lime/cement columns to be applied with the Japanese CDM method.

The performance of the CDM solution has been demonstrated in practice, with the predicted settlements being in the range of 50 to 100 mm and measured settlements being approximately 50 mm after construction was completed. The consequent performance of the tunnel joints has also been shown to be as expected with joint opening being well within the predictions.

The Busan-Geoje immersed tunnel opened in 2010 and has been a very successful project, the soil improvement being one of its major components without with it could not have been constructed.

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