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The Design of the Øresund Tunnel Anchored Ramps

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Summary

The anchored reinforced concrete approach ramps to the Øresund Tunnel provide the transitions from the earthworks cuttings to the tunnel portal. They carry both motorway and railway traffic.

The ramps are required to withstand the uplift forces from the hydrostatic pressures associated with the surrounding water table. The ramps, in conjunction with a watertight geomembrane, are also required to exclude water from the approaches.

The anchored ramp design was proposed at any early stage of the design and build tender preparation as a substitute for the gravity structures originally envisaged. This paper describes the main features of the design and how they evolved during detailed design.

1. Introduction

The 16km long Øresund Link between Denmark and Sweden comprises an immersed tube tunnel, a man made island and a series of bridges. The link carries a dual lane motorway and a twin track railway. The ramps described in this paper are part of the immersed tube tunnel contract which runs between a Peninsula of reclaimed land near Copenhagen airport and the man made island, known as Peberholm. The ramp structures extend for about 400 metres at each end of the tunnel to support the motorway and railway approaches.

The design and build tunnel contract was won by Øresund Tunnel Contractors (a joint venture of 5 major European contractors comprising NCC AB of Sweden (leader), John Laing Construction Ltd of England, Dumez-GTM SA of France, Boskalis Westminster Dredging bv of The Netherlands and E Pihl & Son A/S of Denmark). Symonds Travers Morgan were the designers during both the tender and detailed design stage. Site work started in September 1996 and is due for completion in 1999, prior to the installation of finishing works which will allow the link to open in 2000.

Early in the tender period it was found that the gravity ramp structures shown in the Owners' illustrative design would require significant excavation and subsequent replacement with concrete and ballast material. An anchored ramp solution offered savings in both material and time. The major design task was to find suitable details to provide a cost effective stable watertight approach structure. At the time of writing this paper the design is almost complete with the Owners' review still on-going.

2. General Description

2.1 Geology

The ground conditions are generally favourable for construction. From sea bed level downwards the sequence of the existing geology is marine deposits, glacial deposits and finally limestone to significant depth. The marine deposits occur in thin layers and are removed where they might affect any structures. Glacial deposits contain mainly clay till with some gravel, stone and sand. The depth of material is generally within the 2 to 5 metre range. The underlying limestone is characterised by silicification and by subvertical jointing. Near its top, there are glacially disturbed layers to a depth of 2 to 4 metres.

The founding levels of the Peninsula segments vary from about -3m from mean sea level (MSL) at the top of the ramps to -9m at the Portal entrance. The foundations are either within glacially disturbed limestone, glacial deposits, or limestone fill. On the Island the levels vary from -8m MSL at the Portal entrance to -3m at the top of the ramps. The foundations are within disturbed or undisturbed limestone, the glacial deposits or limestone fill placed as part of the Island reclamation works.

2.2 Structural Layout

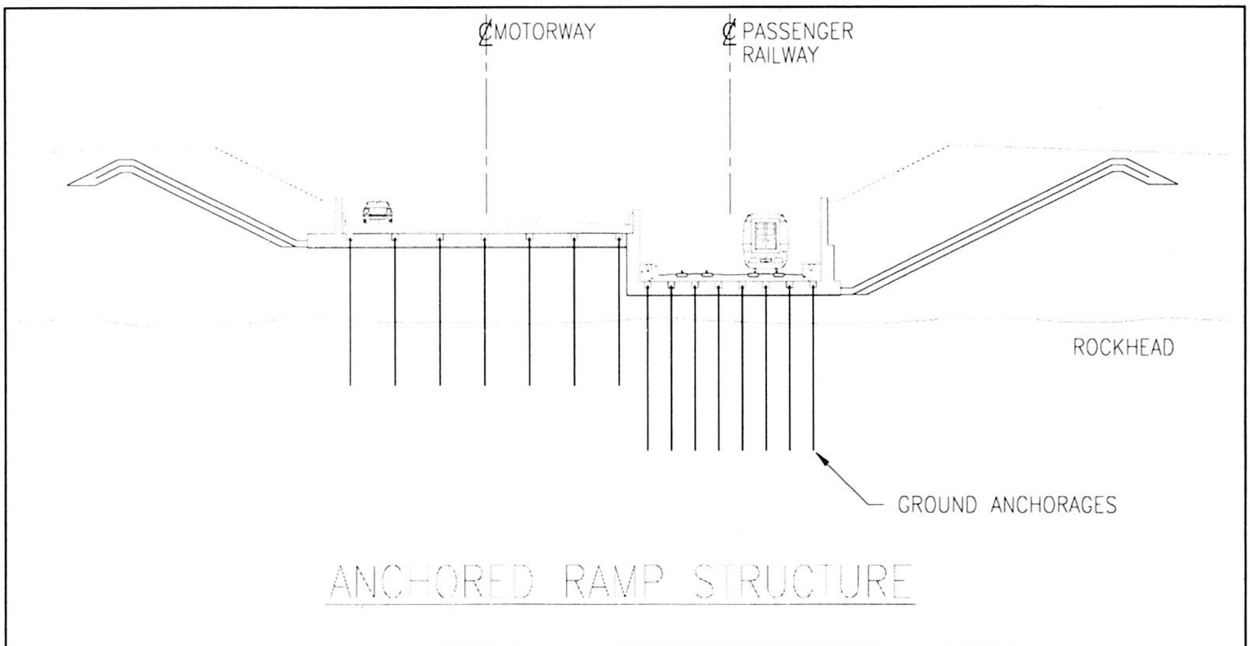

Figure 1: Typical Cross Section through ramps

The ramp base slabs and geomembrane are used to exclude groundwater from the approaches. They are subject to buoyancy forces which vary due to changes in the groundwater level predominantly caused by tidal movements, especially on the Island. Buoyancy effects are resisted by the weight of fill above the geomembrane and by the use of ground anchorages for the slabs. A total of approximately two thousand three hundred 36mm diameter grade 1080/1230 bar anchorages with lengths between ten and fifteen metres were specified for the ramps.

The vertical alignment of the segments follows the railway and motorway alignments. In cross-section the top of the motorway slabs is set horizontally below the motorway alignment by at least the full carriageway construction thickness. For the railway track the slab is set at a minimum of 800mm below the lowest rail level with a nominal crossfall to the south to provide drainage of the track ballast. The motorway is able to climb away from the tunnel at a steeper gradient than the railway. As a consequence the motorway ramps are always at a higher level than the railway ones and can be stopped sooner. A typical cross section is shown in Figure 1.

The ramps are divided into segments to mitigate the effects of shrinkage, thermal and ground movements and differential settlement with the adjoining segments. Various bay lengths were considered during the initial design for both the bases and walls. A typical length of 20 metres was eventually selected. Expansion joints in the base slabs subject to groundwater pressure are sealed with waterstops between segments. Waterstops in the transverse direction will be connected to the waterstops in the longitudinal direction via a crucifix section, to maintain a watertight seal. Shear keys are provided between adjacent sections to limit differential vertical movement between adjacent slabs and eliminate any possible harmful effects on the carriageway and railway construction.

3. Design

3.1 Loadings

A number of load cases and combinations based on the Eurocode were considered during the design of the segments. The individual loads are shown in Table 1 below.

Permanent Loads:	Variable Loads:	Accidental Loads:
Self weight	Nominal surcharge	Accidental traffic collision
Road pavement	Road traffic loads	Accidental train collision
Railway ballast	Train loads	Earthquake
Hydrostatic pressure	Water level variations	
Ground anchorage loads	Temperature loads	
Earth pressures	Snow loads	
Settlement		
Differential settlement		
Construction sequence		

Table 1: Loads used in the base slab and ground anchorage design

For base and anchorage vertical displacements, the following ground conditions were considered:

- Settlement due to de-watering.
- Swell due to unloading during excavation.
- Settlement under permanent dead load including anchorage loads.
- Upward movement due to turning off the de-watering system.
- Creep settlement under permanent dead load including anchorage loads.
- Movement due to variation in the buoyancy forces.
- Self settlement of fill areas

3.2 Base Design

Typical base slabs, representative of groups of slabs, were analysed as 2-D plane finite element models. The slabs were supported on springs to model the ground conditions anticipated from the available data and from the additional ground investigation undertaken during the contract. The founding material comprises one or more of disturbed and undisturbed limestone, glacial till and limestone fill. For each material, a range of spring stiffnesses was considered. Additionally, the effects of possible variations in ground stiffness under a particular slab were considered by assigning different support stiffnesses to adjacent supports or groups of supports. All loads were applied to the model including the ground anchorage effects and the variation in groundwater pressures. The section design was completed in accordance with Eurocode 2.

3.3 Ground Anchorage Design

3.3.1 Alternative Types of Ground Anchorage Considered

During the tender stage a variety of solutions for resisting uplift pressures were considered and it was found that ground anchorages would be the most economical solution. At detailed design stage the option of using passive or prestressed anchorages was investigated. The passive anchorages have significant advantages over the prestressed anchorages from constructional aspects, but have a penalty in their generally livelier load-deflection characteristics. The inherent advantages of the passive anchorages over prestressed ones include:

- anchorages will not require stressing
- prestressed anchorage head pockets eliminated,
- steel fixing, shuttering, finishing work simplified
- a potential source of leakage removed.

For passive anchorages, the slab was found to be more susceptible to lift off from the adjoining ground with subsequent loads carried only on the anchorages. The resulting behaviour under live load was therefore almost independent of the soil and depended mainly on the stiffness of the passive anchorages. In the general case, the size of the imposed downward load needs to be assessed against the buoyancy forces to assess how the soil-anchorage system will behave.

The behaviour of the anchorages under train loads was a major consideration during detailed design, especially fatigue aspects. Based on the number of anchorages required to provide vertical equilibrium, movements were found to be typically in the order 10mm downwards under train loads. This magnitude of movement generates large stress changes within the anchorages and results in a severely limited number of fatigue loading cycles which they can withstand. In order to produce a satisfactory design, the number of passive anchorages would have needed increasing to approximately twice the number of prestressed anchorages required.

For prestressed anchorages, provided the precompression is greater than the subsequently applied upward loads, the soil stiffness affects the slab's behaviour. In weak ground, the spring stiffness of the anchorage plays a more important role in the behaviour and applied loads can cause a large change in stress in the ground anchorages. If the ground is good, as it generally is here, or even moderate the ground stiffness is dominant.

The final design adopted prestressed anchorages predominantly because they provided a more efficient solution against the train loads and the resultant risk of fatigue. The use of bar or strand tendons was also considered. For this scheme, where a large number of relatively lightly loaded and short ground anchorages were required, bars were considered the most practical and economical solution.

The design life of the anchorages, which will be either partially or totally within the groundwater range, is required by the Contract to be 100 years. The corrosion resistance of the tendon or the ability of the tendon system to exclude water and any harmful chemicals is a fundamental requirement in achieving the required design life. The selection of the tendon and corrosion protection system is therefore a major factor. The contract requirement adopts the approach of assuming that if the grout body of the anchorage is fully under compression then water will be kept away from the metal tendon within. A concern about this approach was that cracking can occur behind the end nut within the fixed anchorage with subsequent corrosion of the anchorage. The alternative principle of double corrosion protection was therefore adopted.

3.3.2 Detailed Analysis

A simple elastic model was established to investigate how the soil-structure system behaved under the design loading. This was used to assess the anchorage stressing loads such that the anchorages would not be overstressed due to subsequent upward loads and that lift-off of the slabs would not occur.

The soil and anchorages were modelled as elastic springs connected at the top by the slab, which was assumed to be infinitely stiff. Loads applied to the system cause deformations and result in changes in stress within the soil mass and the anchorages. The slab acts over a very large area compared to the depth of the anchorages, so one dimensional compression of the soil was assumed. The small deformations occurring in soils below the anchored zone were considered separately from the above using a simplified analysis. The effect of the increase in pore pressure resulting from the dewatering being switched off was modelled by applying an equivalent mechanical force to the system. The applied force equals the change in the 'effective stress' in the soil, which will change in volume accordingly (i.e. heave as pore pressures build up).

Allowance was made for the different behaviour of the system depending on whether the material behaves in a drained or undrained manner during anchorage stressing. If the soil behaves as 'undrained' during stressing, excess pore pressures will be set up which will slowly dissipate after lock-off leading to loss of anchorage force. This loss will not occur if the soil is 'drained' during stressing. The consequence is a difference in the final anchorage force with a possible overstress in the drained case but potential understress (i.e. possible lift-off) in the undrained case. In practice, the real behaviour will lie somewhere between the two extremes. The model takes this into account by allowing for different degrees of consolidation to apply during stressing, with further consolidation allowed to occur after lock-off if appropriate.

3.3.3 Calculation of Required Ground Anchorage Length

A ground anchorage consists of three main elements: a fixed length, a free length and the head. The free length was determined from the maximum of the following criteria:-

- A minimum length of 5 metres;
- Extending the distal end of the free length into sound limestone;
- Providing a total free and fixed length to mobilise sufficient mass of rock and soil to resist uplift.

For a 150mm diameter borehole and a conservative ultimate skin friction of 0.5 N/mm^2, the required fixed length was calculated to be 8 metres. However, it was considered that there was potential for reducing this length and so full scale pre-tests were carried out at locations near to the proposed alignment.

Six pull-out tests were carried out with fixed lengths ranging from 3 to 6 metres. To demonstrate that the fixed length was adequate, a pull-out load of three times the working load was used. The bar in the proposed works anchorage cannot accommodate this load, therefore nine 15.7mm diameter strands were used. In order to obtain comparable results with the proposed bar anchorage system, the same diameter corrugated sheath was used for the fixed length enclosure.

The ground anchorage installation and testing were carried out in accordance with DIN 4125. The tests demonstrated that a fixed length between 4 and 5 metres was capable of withstanding the ultimate pull-out force of 1876kN. It was decided to specify a minimum fixed length of 5 metres for the works ground anchorages to allow for variations in the limestone. To verify the pull-out tests and confirm the adequacy of the proposed ground anchorages for the main works, suitability tests were carried out in accordance with DIN 4125. These tests concluded that the proposed ground anchorages with a 5 metre free length and a 5 metre fixed length complied with the requirements of DIN 4125.

4. Conclusions

Anchored ramps can provide an extremely cost-effective solution to the problem of buoyancy effects where suitable ground conditions exist. The problem of heavy cyclical loading associated with railway loading requires special attention in developing an appropriate design. Each site with its unique ground and loading conditions requires careful consideration in developing a solution.

Design and build has benefits in stimulating considerations beyond the usual boundaries imposed on designers and contractors and permits investigation of real cost-effective design.