# The design and construction of temporary works for Limerick Immersed Tube Tunnel

## Design et construction des travaux temporaires du tunnel-tube immergé de Limerick

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ABSTRACT: An immersed tube tunnel has been constructed across the River Shannon downstream of Limerick. Enabling works either side of the river comprised two 130 x 30 m retained excavations. Ground conditions at the site consisted of up to 20 m of soft alluvial soils overlying thin and intermittent glacial deposits on top of limestone. The excavations were retained by combi-walls of 1420 mm tubes and pairs of sheet piles, with the maximum height between supports being 15 m. During construction of the southern section, considerable problems were experienced with the installation of the base slab underwater. Radical changes were therefore made to the already complex construction sequence for the northern section. This required very close cooperation between the construction team, the geotechnical designers and the structural designers. The aim of this paper is to show how this cooperation was achieved, thus enabling float-out of the first tunnel unit to take place on programme on 7th September 2008.

RÉSUMÉ : Un tunnel-tube immergé traversant la rivière Shannon a été construit en aval de Limerick. La réalisation des travaux ont nécessité deux excavations de retenue de 130x30m. Les conditions de sol du chantier consistaient en 20m de sols alluviaux mous couvrant des dépôts glaciaires minces et intermittents recouverts de calcaire. Les excavations étaient retenues par des combi-murs faits de tubes de 1420mm et de paires de palplanches avec une hauteur maximale de 15m entre les supports.Pendant la construction de la section Sud nous avons expérimenté des problèmes considérables quant à l'installation de la dalle de base immergée. Il a donc fallu engager des changements radicaux dans la construction déjà complexe de la section Nord. Tout ceci a nécessité une étroite coopération entre l'équipe de construction, les concepteurs géotechniques et les concepteurs structuraux. Le but de cet article est de montrer comment on a pu parvenir à cette coopération qui nous a permis de placer la première unité du tunnel le 7 Septembre 2008, comme programmé.

KEYWORDS: Tunnels, excavations, retaining walls, temporary works, alluvial clays

## 1 INTRODUCTION

Limerick Immersed Tube Tunnel forms part of the new Limerick Southern Orbital Road. It crosses the River Shannon about 3 km downstream of the city centre (Figure 1).


Figure 1. Location of tunnel

The tunnel is 675 m long, and is formed of five precast units each 100 m long, 25 m wide and 8 m high, with a cut and cover section at each end. The units were constructed in an in-line casting basin on the north side of the river, then floated out through a temporary retained excavation and sunk in a dredged trench in the river.

Design and construction were carried out by Direct Route (Construction) Ltd, a joint venture of John Sisk and Son, Lagan, Roadbridge and Strabag.

John Sisk and Son were also responsible for the design of the enabling works for the northern float-out and southern cut and cover sections, assisted by their consultants Webber Associates (subsequently Coffey Geotechnics) and MLM Consulting.

## 2 GROUND CONDITIONS AND SOIL PROPERTIES

A schematic section of the ground along the line of the tunnel is presented on Figure 2. Ground level is approximately at datum. On the south side is Bunlicky Lake, an artificial lake created following excavation by the nearby cement factory. Ground conditions comprise up to 20 m of soft alluvial clay overlying a thin and intermittent layer of glacial deposits then Carboniferous (Visean) Limestone.

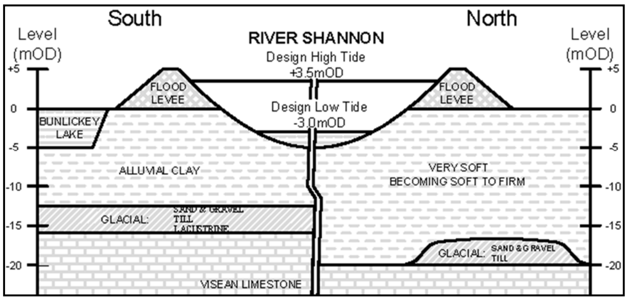


Figure 2. Schematic ground section along line of tunnel

The properties of the alluvial clay dominated the design of the tunnel. They were investigated primarily using static cone penetration tests (CPTs). Figure 3 shows a typical CPT profile. There is a thin crust, below which the strength decreases, then gradually increases again with depth. Between 11.5 and 13.5 m the end resistance increases significantly, and this layer is believed to be a paleo-surface which at some time has dried out. Below it, the cone resistance decreases substantially. Over much of the depth of the clay, the sleeve friction is zero. This is believed to result from the very sensitive nature of the clay, which liquefies as it is penetrated by the cone, so the friction sleeve measures no strength. The measured pore pressures were correspondingly very high.

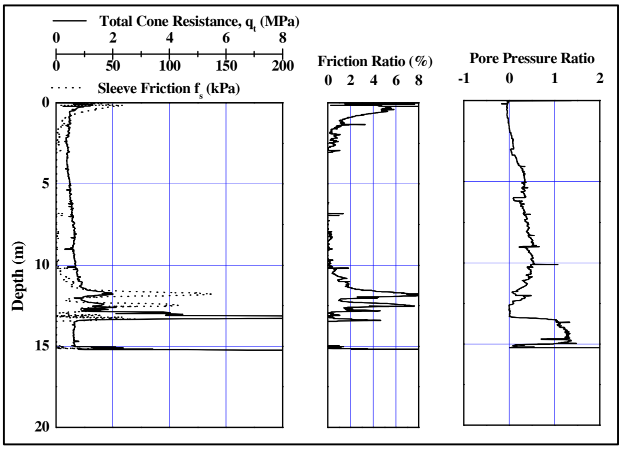


Figure 3. Typical static cone test profile

Figure 4 shows a composite plot of cone resistance, plotted as interpreted undrained shear strength, together with the design profile adopted and results from undrained triaxial tests on U100 samples and from vane tests carried out in boreholes. Both the triaxial tests and the vane tests gave much lower values than the cone tests. It was believed that the very sensitive nature of the clay led both to sample disturbance and to disturbance below the base of the borehole. In order to verify the design profile, high-quality "Mostap" push samples were taken using the cone equipment, and subjected to unconsolidated undrained and direct simple shear tests. These gave values straddling the design profile from the cone tests, and hence gave additional confidence in its use. Buggy and Peters (2007) derived a very similar shear strength profile for the approach roads to the tunnel.

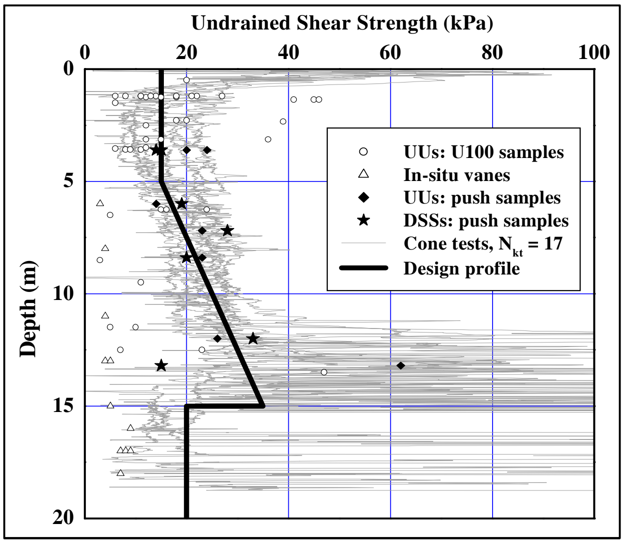


Figure 4. Undrained shear strength results for alluvial clay

It may be noted that the design shear strength profile corresponds to a value of the ratio $s_u/\sigma_v'$ of between 0.30 and 0.35. This is comparatively high for a normally consolidated clay. Two cone tests were carried out from the top of the flood levees, where the value of $\sigma_v'$ would be expected to be 80 to 100 kPa greater, but to have been applied for a comparatively short period. These gave higher shear strengths, but a lower value of $s_u/\sigma_v'$ of 0.24, consistent with what would be expected for normally consolidated clay. It is therefore believed that the high strength ratio of the clay results from some form of aging.

## 3 DESCRIPTION OF ENABLING WORKS

As briefly described above, the tunnel units were constructed in an in-line casting basin on the north side of the river. They were floated out through an open-ended retained excavation 130 m by 30 m (Figure 5). A bulkhead 30 m from the casting basin end retained the river, and was removed when the casting basin was flooded. On immersion, the tunnel units were placed in a trench dredged across the river. The first unit was connected to a cut and cover section at the southern side of the river. This was constructed in a second retained excavation, also 130 m by 30 m, which was essentially a closed-ended box.

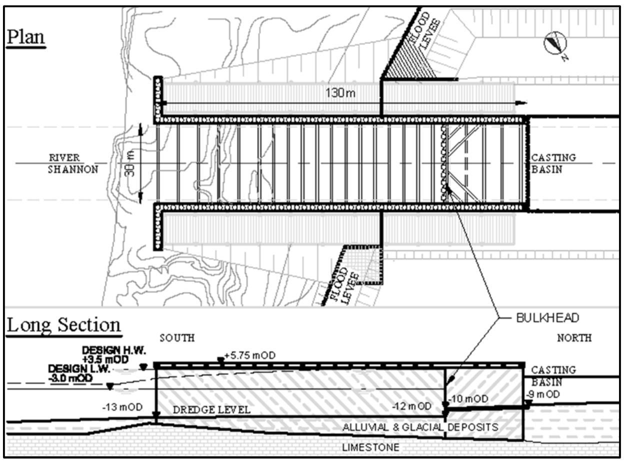


Figure 5. Northern float-out section

Retention for both structures was provided by 1420 mm tubes and pairs of sheet piles. On the north side of the river, the glacial deposits overlying the limestone were not continuous. Dowels were therefore drilled into the limestone to provide toe fixity for the tubes. On the south side, where glacial deposits were continuous, dowels were required only where the walls were directly exposed to the river.

The walls were connected by a concrete capping beam. The capping beams were propped by 914 mm diameter tubes at 5 mOD. The southern cut and cover section was also propped by a base slab. Excavation of the box and casting of the base slab were carried out underwater, before the excavation was pumped dry to enable construction of the tunnel section.

The northern float-out section was propped by a base slab on the northern side of the bulkhead. The bulkhead marked the northern extent of the immersed tunnel units: after they were installed, the bulkhead was resealed around the tunnel, the casting basin pumped dry, and the northern cut and cover section of tunnel constructed on the base slab.

On the river side of the bulkhead, the float-out unit was excavated to between -12 and -13 mOD. There was no base slab on the river side, so the wall had no structural support between the struts at 5 mOD and the toe dowels at about -20 mOD.

Design calculations showed that, where the river flood levees abutted the walls of the excavations, they would impose excessive bending moments in the tubes. The bunds were therefore removed and replaced by sheet piles to a distance of 15 m from the walls, and ground level was reduced to datum.

## 4 DESIGN OF NORTHERN FLOAT-OUT SECTION

As described above, the southern cut and cover excavation was a closed-ended rectangular box. The construction sequence was therefore reasonably straightforward. The northern float-out section was much more complex, and is therefore the main subject of this paper.

It is well known that the stresses and displacements induced in propped excavations are very dependent upon the construction sequence. In the case of the float-out section, this dependency was compounded by the effects of the river tides and by the fact that different sections of wall effectively had different construction sequences, which resulted in complex interactions between the walls.

With this in mind, a set of isometric drawings was produced to illustrate the entire construction sequence stage by stage. Figure 6 shows the drawing for a single stage, which may be used to illustrate the complexities of the design. Although the isometric drawings were produced as an aid to the designers, they proved invaluable during construction, helping site staff to manage progress, and being used as a basis for reassessment when considering changes to the construction sequence.

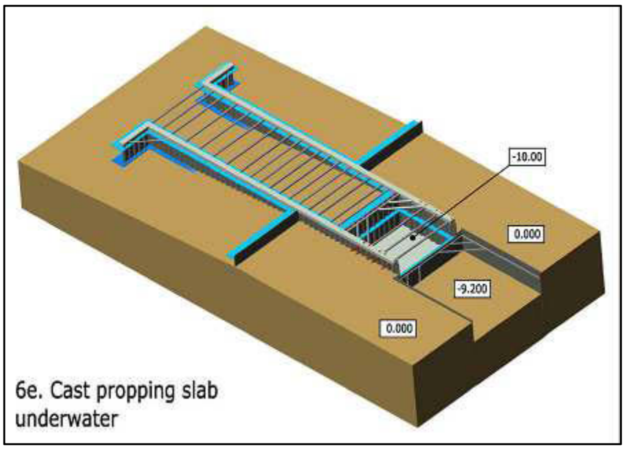


Figure 6. Isometric drawing of construction stage

The full construction sequence was analysed using the embedded retaining wall software FREW. The calculated structural element forces in different wall sections were then passed to the structural engineers in both tables and graphs. Thus, for example, Figure 7 shows the forces in the dowels, struts and base slab restraining the bulkhead. The large effect of the tides, particularly on the base slab, is very evident. This added further complexity to the analysis, since it was necessary to consider the effects of construction stages being carried out at different stages of the tide. Thus one of the most difficult and important aspects of the design was, not so much the analyses themselves, as keeping track of the different cases that had been analysed, and checking that all cases had been considered.

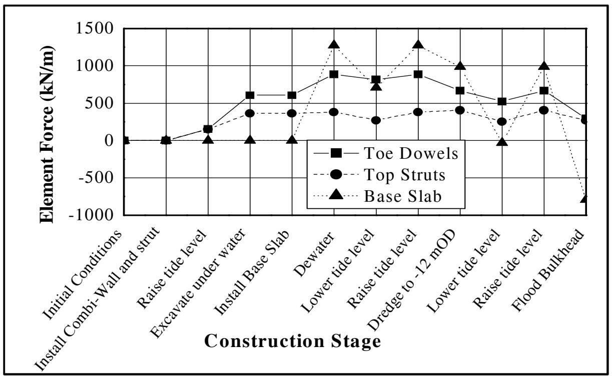


Figure 7. Calculated forces in structural elements

The forces thus calculated were then used to carry out structural design of the remainder of the section. As well as the structural analysis of the walls considered as embedded walls, assessments also had to be made of both lateral and shearing overall stability of all or part of the overall structure. Figure 8 shows the forces acting on the float-out section north of the bulkhead. In order to achieve stability of this section, both the capping beam and the dowels in the limestone needed to provide substantial resistances.

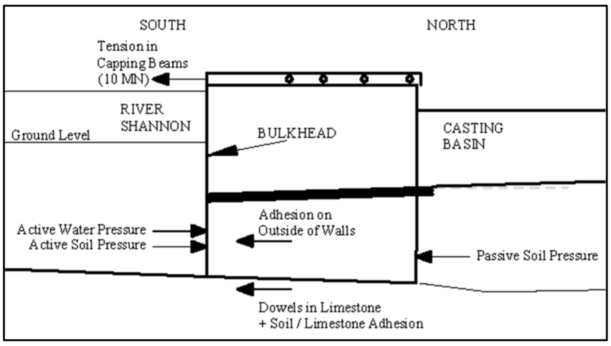


Figure 8 Forces acting on float-out section north of bulkhead

Shearing stability was a less obvious problem, for at first sight the entire structure might be considered to be a rigid box. However, it was realized that the sheet piles between the pipes could deflect out of plane. They thus provided no resistance to rotation of the piles about their toe, the dowels having very small moment capacity. A small model was constructed to demonstrate this effect (Figure 9). In order to provide stability, three anchors inclined at 45°, each of working capacity 1 MN, were installed from each capping beam into the limestone.

During construction, survey markers were installed on top of the capping beam. After excavation on the river side of the bulkhead, it was found that the structure was oscillating about 5 mm between high and low tide. Forces were measured in selected struts using strain gauges and also by jacking them away from the capping beam. Measured forces were found to be lower than calculated values.

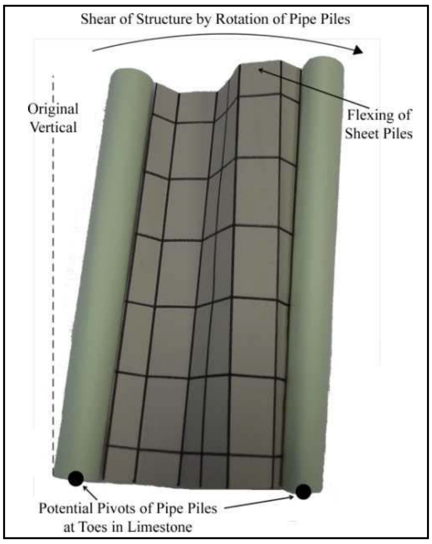


Figure 9 Model to illustrate potential "shear" mechanism

## 5 CHANGES TO CONSTRUCTION OF FLOAT-OUT SECTION

Design calculations had indicated that excavating north of the bulkhead in the dry would lead to excessive moments in the piles. It was therefore intended to install temporary sheet piles between this area and the casting basin, and to excavate and cast the base slab under water. During construction of the southern cut and cover section, considerable difficulties were experienced maintaining a suitable surface for underwater casting of the base slab, due to large volumes of suspended silts slowly settling there. In addition, excavation underwater, although practicable, was much slower than excavating in the dry. It was therefore decided to develop a strategy for excavating north of the bulkhead in the dry.

By this time considerable experience had been gained in construction of the casting basin. Although not as deep as the two structural sections, it was a substantial excavation in its own right. It was retained by sheet piles with anchors at high level, inclined at 45°. Despite the free length of up to 30 m through soft clay, installation into the limestone to achieve a working anchor load of 1 MN had become reasonably routine. The sheet piles were also retained by struts below the casting basin floor. Before the permanent struts were installed, the walls were supported by temporary hydraulic struts.

A combination of these techniques was adopted for the float-out section. The key to their success was the reduction of the external soil and water forces on the walls. This was achieved (Figure 10) first by constructing a bund at the far southern end of the section to prevent the river from acting against the bulkhead. Then 2.5 m deep excavations were made outside each wall. Temporary struts were installed between the side walls at -6 mOD, and anchors installed at the same level in the bulkhead.

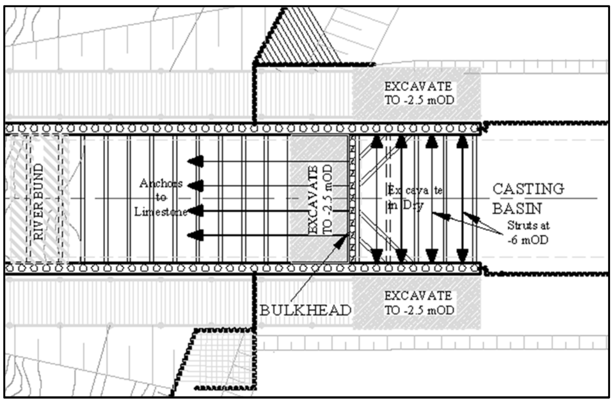


Figure 10. Measures adopted to enable construction in the dry

This operation demonstrated again how much easier excavation was in the dry rather than underwater.This factor, together with a change in the design of the permanent tunnel works, led to a requirement to excavate in the dry on the river side of the bulkhead also. A construction sequence was devised (Figure 11), which again involved the use of temporary struts (this time at two levels), the exclusion of the river by means of bunds outside the walls, and the reduction of ground level outside the walls. Other features were a sheet pile wall across the open end of the excavation, a 1:5 slope down from the sheet pile wall, and a 3 m thick mass concrete slab at -12 mOd as permanent works to support the northernmost tunnel unit.

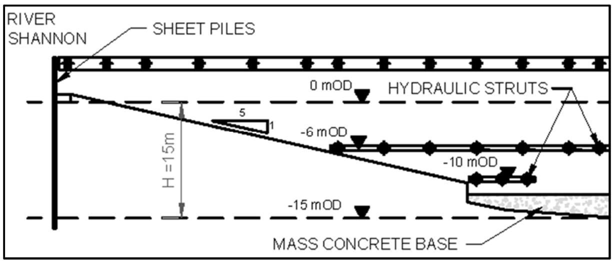


Figure 11. Section through new construction measures

These proposals were set out on a colour-coded plan, to facilitate their checking by the design consultants. The checking led to some minor changes in detail, and to the identification of one potential major problem. This involved the stability of the end slope, which although only 1:5, would be 15 m high after excavation for the mass concrete slab. A simple calculation using Taylor's curves demonstrated that an average shear strength of 33 kPa was required to achieve a safety factor of 1.3 against undrained failure, considerably greater than the design values (Figure 4). The slope was therefore flattened still further, to about 1:8. Its toe was raised to -6 mOD, and further excavation at the toe supported by an anchored sheet pile wall.

These works were successfully implemented, and float-out of the first tunnel unit took place on programme on 7th September 2008.

## 6 CONCLUSIONS

Successful construction of the Limerick Tunnel enabling works was achieved despite the complexity of the structures and the challenging nature of the ground conditions. Technically, one of the most important factors was that the forces on the structures were dominated by water and ground levels, and therefore that significant advantages could be achieved if these levels could be controlled and hence varied.

Most of the analyses were carried out using industry-standard software. Simple calculations for force and moment equilibrium slope stability proved also to be of great value. Perhaps more important than the calculations was the identification of mechanisms that needed to be analysed. Tools to aid understanding of these included isometric diagrams (Figure 6), graphs (Figure 8), a physical model (Figure 9) and colour-coded plans. Such simple but effective tools enabled designers and constructors to achieve a common understanding of the project and hence its successful completion.

## 7 REFERENCE

Buggy, F. And Peters, M. 2007. Site investigation and characterisation of soft alluvium for Limerick Southern Ring Road – Phase II, Ireland. *Proc. IEI Conference on Soft Ground, Port Laoise, paper 1.6.*