Dublin Port Tunnel -Excavation of an 11.8m diameter urban motorway tunnel

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ABSTRACT

Every year more than 2,000,000 heavy vehicles pass through Dublin Port. These vehicles contribute to serious traffic congestion in the city centre of Dublin. The Dublin Port Tunnel has been planned to connect Dublin Port to the M1 Motorway and thereby remove 9,000 heavy vehicles a day from the city centre. The twin tube road tunnel is approximately 4.5 km long with each tube having 2 lanes of carriageway and a walkway for emergency exit and maintenance. Approximately 2.6 km of the total tunnel length of 4.5 km is being constructed by bored tunnelling whilst the remainder is being built by cut & cover methods. The NMI Consortium was awarded the design and build contract in December 2000. The consortium consists of Nishimatsu, Mowlem (UK) and Irishenco (Ir.). Nishimatsu are responsible for the design and construction of all the civil works related to the bored tunnels and decided to use two different 11.8 m diameter TBMs mainly due to the differences in ground conditions. One TBM for hard rock will excavate about 86% of the bored tunnel section in hard limestone and below densely populated housing areas. The second machine is an Open-Face TBM equipped with 2 intermediate decks and 3 excavator arms for its 660 m drive in boulder clay. Both tunnels are lined with a structural concrete segmental primary lining and an insitu concrete secondary lining that provides fire protection and a smooth finish..

1. INTRODUCTION

Dublin Port Tunnel links the M1 motorway to Dublin Port, (see Figure 1.1) and comprises 4.5 km of tunnel structure with new surface road connections at each end. The tunnel can be separated into three major construction areas:
1) North Cut and Cover section and portal
2) Bored Tunnel Section
3) South Cut and Cover section and the portal

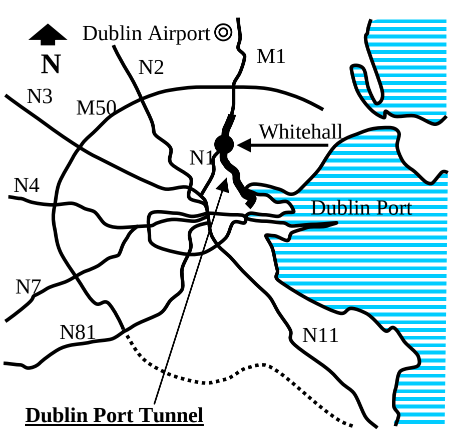


Figure 1.1 Route of Dublin Port Tunnel

Construction of the bored tunnel was originally planned before the Tender Stage to be excavated mainly by drill and blast with a sprayed concrete lining. It is generally considered as a more economical method in hard rock.
Construction by Tunnel Boring Machine (TBM) with a segmental tunnel lining was introduced by Nishimatsu at tender because the drill and blast method was not considered feasible from Nishimatsu's assessments due to the extremely stringent contract requirements placed on maximum peak particle velocity of ground borne vibration.
And so the challenge of excavating unique bored tunnels, of 11.8 m diameter, by hard rock TBM beneath 400 residential properties in a city area then began.

2. GROUND CONDITIONS

The strata predicted to be encountered along the bored tunnel section can be summarised by the following sequence: made ground, glacial silts, sands and gravels, boulder clay and carboniferous limestone. The boulder clay can be subdivided into the upper and lower black boulder clays and sandy brown boulder clay.

The made ground and glacial deposits are close to the surface and were not encountered in bored tunnel operations.

2.1 Boulder Clay

The upper black boulder clay is typically stiff to very stiff dark brown grey sandy silty clay with much gravel and occasional cobbles.

The sandy brown boulder clay consists of stiff, becoming very stiff and hard brown sandy silty clay/sandy clayey silt with some gravel, cobbles and occasional boulders. In comparison to the upper and lower boulder clay, the sand content increases while the gravel content decreases. Sand and sandy-silt lenses are present and have been seen to extend laterally for 20 m to 30 m and 1 m to 2 m vertically. The maximum thickness of the sandy brown clay varies between 6 m and 13 m and it becomes thinner towards the south of the boulder clay tunnel drive. The brown boulder clay is generally accepted to be weathered lower black boulder clay. The sand lenses contain charged water with a head roughly at the ground surface, the lenses however have a limited water volume and low recharge.

The lower black boulder clay underlies the sandy brown boulder clay. It can generally can be described as very stiff to hard brown grey slightly sandy clay with much fine to coarse, subrounded to subangular, limestone gravel and occasional cobbles and boulders.

The open shield section of the tunnel drive within the boulder clay rises from the construction shaft to the northern portal and has moved through the three boulder clay types. The sandy boulder clay has been the predominate layer observed within the face. Occasional ground water was encountered but this has been adequately controlled by drainage and dewatering.

2.2 Carboniferous Limestone and Shales

The Dublin Formation comprises carboniferous limestones and shale of the Fingal group and was previously known as 'Calp' limestone. Where observed in tunnel, the limestone can be categorised as predominantly strong to very strong, very thin to medium bedded, dark grey limestone. Within the limestone there are occasional interbedded, moderately strong, very thin to thin, dark grey to black, unweathered shaley mudstones and strong to very strong fine grained argillaceous limestone. Through extensive in tunnel face logging, faulting and folding of the limestones and shales has been observed. The dip of bedding generally varies between 5° and 30° with vertical bedding observed within fault zones and highly fractured areas. The limestone within the project area lies wholly beneath the water table and piezometer observations indicate artesian pressure to be near ground level. Water inflows during tunnelling into the hard rock TBM cutterhead were typically around 250 litres/min over the full 11.8 m diameter face. Zones of moderate water ingress have been encountered during construction but forward grouting has not been required.

Shale bands within the limestone formation have given rise to some problems during tunnelling, by way of crushing to a clay/silt consistency. This together with a high dip of >50° led to some plucking of boulder size slabs of the surrounding limestone. Some overbreaking occurred along areas of intersection joints and bedding planes but this rarely extended to more than 300 mm and had little effect on surface settlements.

3. TBM AND METHOD OF CONSTRUCTION

After giving due consideration to the expected geological conditions and possible location of the construction shafts, Nishimatsu decided to use two Herrenknecht TBMs (shown in Figures 3.1 and 3.2) for the bored tunnels. Both machines had been previously used and were fully refurbished and modified prior to delivery.

Figure 3.1 Hard Rock TBM

Figure 3.2 Open Face TBM

3.1 Hard Rock TBM

The hard rock TBM has a unirotational cutter head with seventy two 17” disk cutters. This machine excavates approximately 2,250 m of tunnel from the 56.6 m diameter construction shaft, located towards the north of the scheme, to the cut and cover reception shaft located in a public park area in the south. The TBM is turned around in this shaft for its return drive to the original launch shaft. Almost all of this drive is within the limestone formation with only a short 150m length by the main construction shaft within boulder clay.

In turning around the TBM a special method was used to lift and turn the 1,300 tonne main shield and cutterhead without time consuming disassembly and reassembly. On emergence from a sprayed concrete reception adit, the TBM was moved onto a specially fabricated steel cradle. This cradle was then jacked up using special hydraulic jacks each equipped with a chamber at its base. Pressurised nitrogen gas was pumped into the chamber to greatly reduce friction between the jacks and the specially screeded base concrete so that the TBM on its cradle could be pulled slowly around (from reception adit to launch adit) by winch and wire ropes.

This enormous TBM, 11.8m in diameter, 11 m long main shield, has a power of 3.2 MW and a maximum thrust force of 6,600 tonnes from 20 pairs of hydraulic jacks. Cutterhead rotation speed is 4 revolutions per minute with a maximum torque of 9,000 kN-m. The actual shove speed is typically between 25 mm to 30 mm per minute, although this is governed by the cutter torque pressure (limited to less than 200 bar). At the location of the occasional fault zones, blocks of limestone can jam in the cutterhead resulting in high torque and slower excavation.

A 150 m long back-up gantry follows the TBM and contains the power supply unit and all other equipment to facilitate excavation activities, erection of pre-cast concrete segments, probe drilling and cavity grouting etc.

3.2 Open Face TBM

The other machine is an open face TBM designed to excavate hard clays such as the Dublin Boulder Clay. This machine excavates a relatively short distance of 330 m from the construction shaft to the interface with the north cut and cover section where it is also turned for its return drive back to original shaft.

The machine is equipped with three excavator arms and a screw conveyer for spoil removal from the face to conveyor belt. It has two intermediate support decks and extendable breasting plates for face support.

The machine has excavated under the busy main arterial N1 road between Dublin, the Airport and Belfast. This section of tunnel has little ground cover (7 to 14 m) and a tunnel separation of 1.2 m in places. Surface settlement has been kept to a minimum by using a thixotropical cavity grout and working 24 hours continuously.

4. TUNNEL LININGS

The bored tunnel section of the Dublin Port Tunnel consists of approximately 2600m of twin two lane tunnels. The finished internal diameter to the secondary lining is 10.29 m and this provides for two 3.65 m wide carriageways, a 250 mm margin and a nominal 850 mm wide walkway. The traffic envelope height is 4.9 m above the carriageway level. A typical finished tunnel section is shown on Figure 4.1.

Primary support to the ground is by a precast concrete segmental lining sealed with hydrophilic gaskets and has been designed to carry the full ground and water loads, (see Figure 4.2). An inner insitu concrete secondary lining is cast against the primary lining to provide a smooth finishing surface, fire protection to the primary lining and a further waterproofing barrier above the road carriageway level. The secondary lining is not continuous around the tunnel circumference and has been designed to carry is own self-weight and tunnel fixtures.

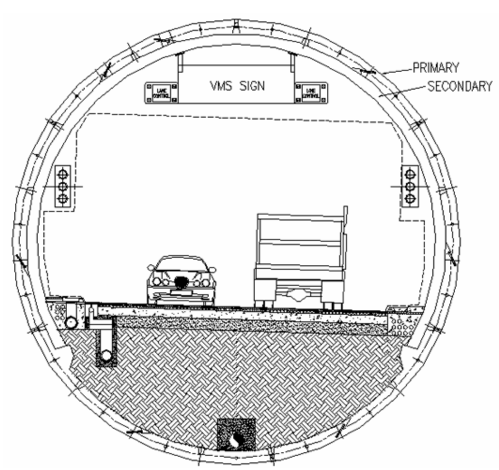


Figure 4.1 Typical Tunnel Section

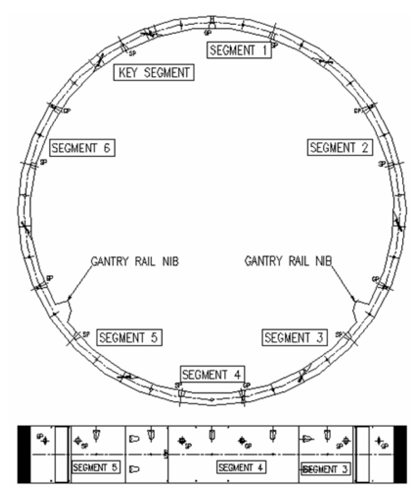


Figure 4.2 Segmental Tunnel Lining

The primary lining has a configuration of 6 segments and a key as shown in Figure 4.2. Each ring is 1.7 m wide and 350 mm thick. The design was developed with Nishimatsu's construction and plant sections in conjunction the TBM manufacturer, Herrenknecht, and Designer, Haswell. The design had to consider both the permanent works requirements and the temporary requirements for the tunnel construction. A review of similar designs used in Switzerland road tunnels with similar machines was also undertaken.

The lining has several characteristics relating to these requirements including:

- A nib on the segments at haunch level for the TBM backup gantries to run on and to provide a starting point for casting of the secondary lining.
- The maximum bogie load from the TBM main gantry was a significant 200 tonnes.
- Tapered linings were required to provide the fully sealed primary lining following the tunnel alignment.
- An invert segment was placed assisting in a fast and accurate ring erection.
- Small building bosses were placed on the circumferential joint to assist in the ring placement.
- The segment has been designed for a maximum of 3 thrust rams each carrying a working load of 265 tonne each.
- The longitudinal spoil conveyor was hung from the segments.

The structural design of the lining included for:

- Single ring design to cover all locations within the tunnel. No difference was made between the boulder clay and hard rock locations.
- Flat radial joints but with 90 mm relief section on each face to prevent loads on the segment edges.
- A maximum of 30 m of overburden to crown and a minimum of 7.5 m.
- A minimum distance between two tunnels of 1.2 m
- Use of C60 concrete and conventional reinforcement bars with approximately 90 kg/m^3.

The lining arrangement with the nib segments and central invert segment results in cruciform joints for the lower four radial joints. The straight rings key positions above axis are alternated to stagger the joints. The concern of leakage of hydrophilic sealing gaskets was considered in great detail with the supplier, C S Kasai. The final selected Hydrotite strip is 22 mm wide and 5 mm thick placed on all surfaces around the individual segment. The manufacturer developed a cruciform testing apparatus and a series of tests were carried out, see Figure 4.3. The worst-case test was for no pre-compression between linings, a 10 mm step between gaskets, a 3% saline water and 45 m head of water pressure. The tests were successful with the main conclusion being the need for a butt joint at the segment corners.

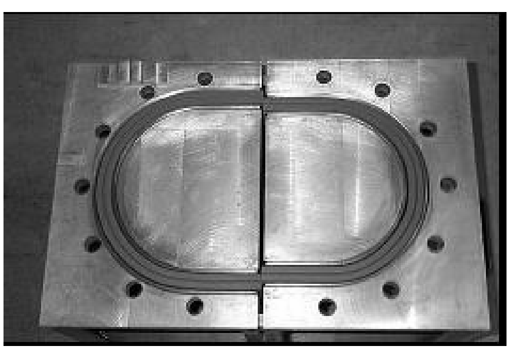


Figure 4.3 Hydrophillic Test Apparatus

Figure 4.4 Fire Test Panel

The secondary lining has several functions including:

- To produce a smooth surface for air flow, painting and cleaning.
- To provide a waterproofing layer to prevent water ingress above carriageway.

- To form a thermal barrier to protect the primary lining from a tunnel fire.

The secondary lining consists of a nominal 275 mm thick unreinforced insitu C40 concrete cast from the primary lining nibs and includes 1 kg of 18 µm monofilament polypropylene fibres per cubic metre of concrete. The minimum structural thickness, after tolerances, is 200 mm. The Employer's Engineers were concerned on the practicalities of pouring a relatively thin lining so a project review was carried out using the Nishimatsu database to show other tunnels with linings of this thickness and reference was made to the examples in Austria.

The Contract required that the tunnel lining provided a resistance to a specified fire curve. The curve was based on a modified hydrocarbon fire with a peak temperature of 1200 ºC for two hours. The design included a number of thermal analyses to demonstrate that the 200 mm lining provided an adequate thermal barrier to protect the primary segmental tunnel ling. In addition to the thermal analysis, a compliance fire test was carried out at TNO in Holland on a representative slab, see Figure 4.2. A number of small cylinder tests were carried out to confirm the effect of varying preload and fibre content. Following the cylinder tests a compliance test on two 2 m square slabs with the selected 1 kg/m^3 was carried out. The fibre volume was selected on the basis of the test and the need to keep the mix workable. The addition of the fibre adequately controlled the spalling and temperature gain through the section and therefore met the requirements to protect the primary lining from damage.

5. SURFACE SETTLEMENT

Surface settlement predictions and monitoring is a critical part of the project due to the large number of residential properties the tunnel route goes under. Comparing the observed surface settlement with predicted values along transverse and longitudinal monitoring stations plays a central part in assessing the ongoing effects of the tunnelling. Transverse plots were produced for surface settlement using the traditional Gaussian distribution curve method. The loss of ground, termed 'volume loss - V_L', is then expressed as a percentage of the cross section of the tunnel. Its magnitude depends not only on geological conditions but also on the tunnelling method used. Contractual limits, termed amber and red trigger levels, were set for the slope of the settlement trough at 1/1000 and 1/500 respectively. Observed slopes have generally been within a range of 1/9000 to 1/3000, far less than the Contract trigger levels.

5.1 Open Face TBM

Data from a typical transverse monitoring array are shown in Figure 5.1. The overall surface settlement of the first drive over the tunnel centre line varied from 35 mm to just over 40 mm.. Localized areas of higher settlement may be due to extensive dry sand lenses encountered along this drive. With k-values (settlement trough width parameter) generally varying between 0.4 and 0.8 the calculated volume loss due to tunnelling lies within a range from 0.5 % to 0.7 %. This is within the predicted volume loss of 0.8 % for the tunnels in the stiff boulder clay.

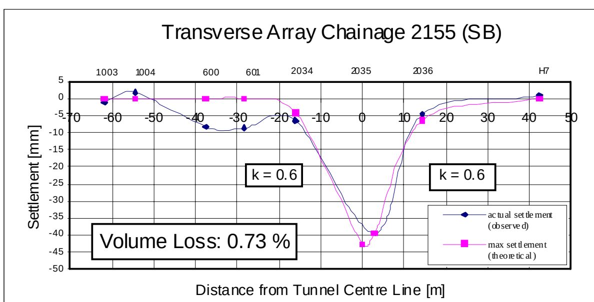


Figure 5.1 Transverse Array Open Face

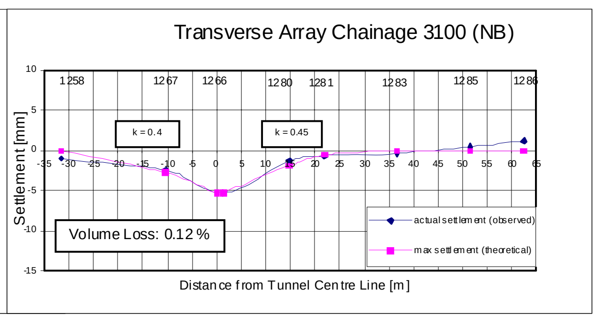


Figure 5.2 Transverse Array Hard Rock

5.2 Hard Rock TBM

Generally surface settlement occurred at a much lesser scale throughout the area of full limestone cover excavated by the hard rock TBM. Settlement and volume loss are limited by the arching action taking place within the limestone. Typical surface settlements being around 5 mm. Data form a typical transverse monitoring array is shown in Figure 5.2. K-values for this area vary but are generally between 0.3 and 0.5 thus producing a volume loss of 0.07 % to 0.12 %. There are a few localised areas of fault zones and other changes of ground conditions where settlements were greater than 5 mm.

6. GROUND BORNE VIBRATION BY TBM

The vibration generated by the large hard rock TBM in the limestone was a major area of concern considering a significant section of the hard rock tunnel is constructed beneath large residential area. The Employer specified very strict vibration limits partly as drill and blast excavation methods were anticipated to be used. The main vibration limits are 15 mm/s PPV above frequencies of 50 Hz and 10 mm/s PPV at frequencies of 50 Hz and below.

Nishimatsu’s Research and Development Department in Tokyo commenced a research study on groundborne vibration from hard rock TBMs 3 years ago before the start on site of Dublin Port Tunnel. A desk study of available published data was carried out together with vibration monitoring and numerical simulations of three tunnel schemes in Japan, namely the Odori, Suzuka and Katsuragawa tunnels. The research has resulted in the development of a prediction procedure for the vibration that will occur on surface properties from TBM construction. To further verify the predictions, specifically related to Dublin, a vibration monitoring trial was carried out on the initial hard rock drive in a ‘greenfield’ situation i.e. before the TBM reached the residential area.

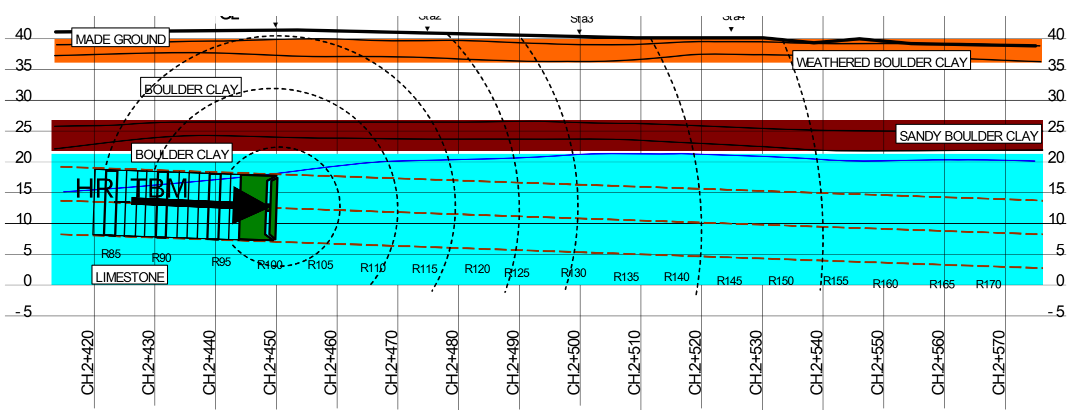


Figure 6.1 Emission of vibration from Hard Rock TBM

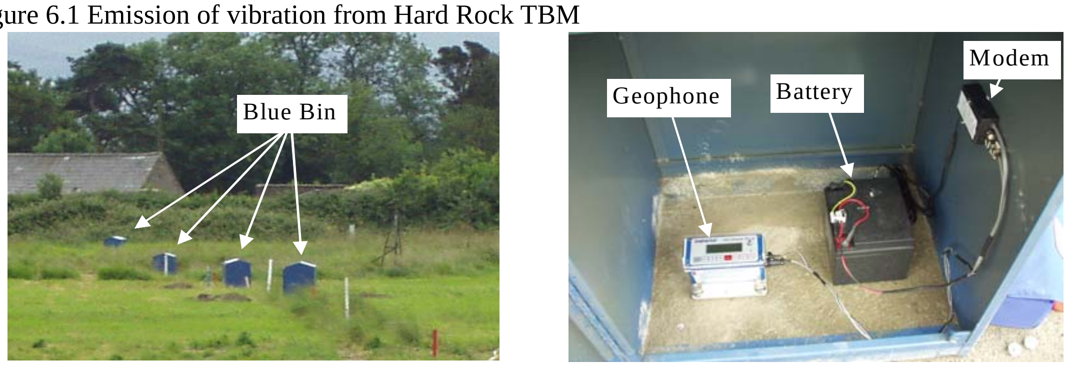
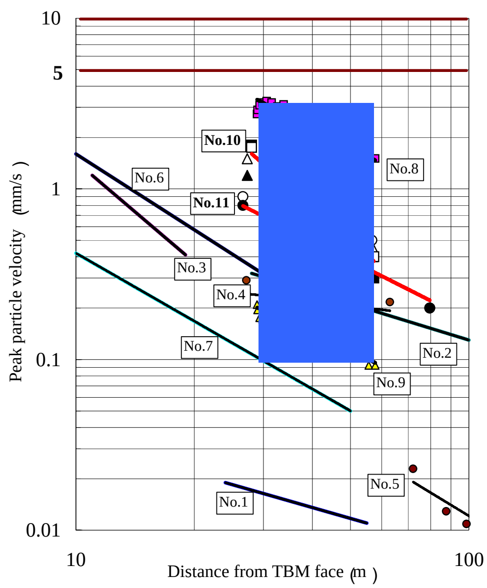


Figure 6.2 Vibration Monitoring Equipment

The trial location is illustrated in Figure 6.1 and equipment used in the field trial is shown in Figure 6.2. The trial also varied several of the TBM excavation parameters to study the sensitivity of generated vibration to the TBM controls. A comparison of predicted results to the vibration data from the field trial along with the desk study data are presented in Figure 6.3. The regression lines denoted as No.8 and No.9 describe the upper and lower bound predictions respectively from the numerical simulation. Regression lines denoted by No.10 and No.11 describe the maximum and minimum field data from the trial. The other lines of No.1 to No.7 are regression from vibration data from other TBM driven tunnels derived from the desk study.

The results of the vibration monitoring data from the trial section were well within the range of predicted vibrations for the hard rock TBM. During tunnelling operations, data from the extensive vibration monitoring within the residential areas have also shown that vibration has been significantly less than contractual limits and predicted values. Nishimatsu Research and Development Department, the engineering staff on site and researchers from Trinity College, Dublin, are continuing the monitoring, assessment and analysis of the groundborne vibration from both of the Dublin TBMs.



Regression lines for FEM results on Dublin TBM compared with those for various tunneling operations

Summary of tunnel vibration data sites, including predicted vibration used for comparison

Symbol	Site (Predictor)	Project	Geology (UCS : Uniaxial Compressive Strength)	TBM type	Tunnel diameter (m)	Tunnel cover (m)	Empirical equation (Regression)
No.1	Durand's Wharf	Rail (JLE C107)	Sands and clays	Herrenknecht Mixshield	5.13	21.2	$y=0.15x^{-0.66}$
No.2	Holywell Coombe	Rail (Channel Tunnel)	Gault Clay /Lower Chalk	Howden full face TBM	8.72	22.6 - 55	$y=3.4x^{-0.71}$
No.3	Cardiff	Cable Tunnel	Mercia Mudstone (moderately strong mudstone)	McAlpine full face TBM	2.44	11	$y=133x^{-1.96}$
No.4	Odori (Nishimatsu)	Emergency Tunnel	Nohi rhyolites (UCS : 176MPa)	Komatsu Open Shield	4.5	100	$y=0.60x^{-0.27}$
No.5	Suzuka (Nishimatsu)	Road (Pilot Tunnel)	Sandstone (UCS : 18MPa)	Komatsu Open Shield	5.0	70	$y=7.6x^{-1.40}$
No.6	(Nelson et al)	no information					$y=48x^{-1.47}$
No.7	(Fomaro et al)	no information					$y=8.8x^{-1.32}$
No.8	Dublin (Nishimatsu)	Road (Dublin Port Tunnel)	Limestone (UCS : 140MPa)	Herrenknecht Open Shield	11.8	23	
No.9							
No.10	Dublin (Monitoring)					21	$y = 1362x^{-2.02}$
No.11							$y = 36x^{-1.16}$

Figure 6.3 Comparison of Predicted result and Monitoring data

7. SUMMARY

The Dublin Port Tunnel is a major project that will improve Dublin's road infrastructure. It is the first large diameter bored tunnel undertaken in Ireland and is a major project for any part of the world. Constructing over 5.2 km of 11.8 m diameter road tunnel with all the associated layby enlargements, crosspassages and niches is a significant undertaking in terms of technical and logistical challenges.

A large amount of technical work on design innovations, settlement prediction, settlement monitoring vibration studies, vibration assessment and monitoring has been undertaken and is ongoing.

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