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Busan Geoje Fixed Link

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ABSTRACT

The Busan Geoje Fixed Link project is one of the landmark projects in South Korea, providing a road connection between Gaduk and Geoje Island near to the city of Busan at the Southern tip of the Korean peninsula. The project consists of two Cable Stayed Bridges, an Immersed Tunnel, rock tunnels and it has a total length is over 8 km.

This paper will focus on the immersed tunnel part of the link and the particular challenges that the project faced in designing and constructing the tunnel, these challenges required a significant evaluation in immersed tunnel technology. The tunnel consists of 18 tunnel elements and has a total length of 3.6 km. The tunnel location was characterised by the following conditions:

- a large depth, maximum depth of the tunnel is 48 m below sea level.
- the tunnel is located in a seismic sensitive area.
- the tunnel is founded largely on weak marine clay, but parts also on bedrock.
- part of the tunnel was constructed on a sub sea embankment which is also founded on a weak marine clay.
- the tunnel has been constructed in a location that is facing to the Sea of japan and is exposed to a severe wind and wave environment with large long period swell waves present for part of the year.

Because for only a few months a year the wave- and weather circumstances were suitable for immersion, extensive numerical and physical model tests were carried out to predict the behaviour of the immersion system for the expected conditions. These model tests were used to determine the wind and wave climate that the immersion system could cope and a go, no-go range of conditions.

Additionally, an accurate, sophisticated short and long term wave and weather forecast system was developed. This system enabled forecasts to be made within a 10 cm wave height accuracy, for periods leading up to the element transport and immersion operations which allowed the immersion operations to be possible within acceptable risks.

The tunnel elements (TE) were built in batches in a pre-cast yard (PC). After inundation of the dock the elements were transported to a mooring location close to the PC-yard for fitting out and to wait for favourable immersion weather.

The elements were immersed using two catamaran pontoons and placed in a previous dredged trench on the sea bed. A taut mooring configuration was used in order to reduce the wave influenced motions to a minimum. Anchor points were created by pre-installed plate anchors.

Due to the total length and installation depth of the tunnel the traditional survey systems using towers and total stations were not suitable. Therefore new survey methods were developed, these included a tautwire system and an Ultra Short Base Line (USBL) acoustic system were used for the positioning of the elements during the immersion operation. Fine positioning of the TE's under swell wave influence was done using a special designed External Positioning System (EPS) and placed on the pre-laid screeded gravel bed.

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1. Introduction

The Busan Geoje Fixed Link project is one of a number of large, fixed links built in the last twenty years and it represents one of the landmarks in the development of immersed tunnel technology both in terms of design and in the immersion technology applied. It is located in South Korea and connects Gaduk Island in Busan Metropolitan City to Geoje Island in Gyeongsangnam-Do Province, the location is shown in Fig. 1 and 3. The project consists of two long span Cable Stayed Bridges between Geoje and Jungjuk Island, a rock tunnel of 300 m in length cutting through Jungjuk island and a 3240 m long Immersed Tunnel between Jungjuk and Gaduk Island.

The tunnel consists of twin two lane road galleries with the sixteen standard tunnel elements (E1–E16) have exterior dimensions of 26.46 m width and 9.97 m height. The widths of element 17–18 was increased to 28.46 m to incorporate a climbing lane at the Western Exit of the tunnel. The typical cross section is shown below in Fig. 2.

This paper describes the design and construction of the immersed tunnel, focusing on issues where immersed tunnel technology has been challenged and advanced to cope with the particular project conditions. The paper provides an overview of the main challenges and solutions developed for the project, it makes reference to a selection of the most relevant published papers describing the main aspects of the project, where further details of the design and construction are presented.

The overall design and construction of the Busan Geoje Fixed Link has been described by Jeong et al. (2012b), Janssen et al. (2006), Jeong et al. (2012) and Yeoward et al. (2010). Key aspects of the design regarding the accidental loading of the tunnel and protection of the tunnel for ship impact and sunken ship loads is presented by Muller et al. (2008).

The project was procured as a Public Private Partnership (PPP) concession with a Special Purpose Company the GK Corporation being responsible for financing, design, construction and operation of the link for 40 years. The construction was led by Daewoo Engineering and Construction Co. and as international design expertise was required by the lending institutions, a Joint Venture of Halcrow and TEC was engaged by the Contractor to provide technical advice. Tenders were arranged for an international designer and a Joint venture of COWI A/S and Daewoo Engineering Co. were awarded the design contract for the immersed tunnel in 2003. A contract was also awarded for an Independent Design Checker (IDC) to Ingerop, Acardis and Seo Yong and a Construction Supervisor contract was awarded to Yooshin Engineering Corporation together with Mott MacDonald. International contractors were involved with the construction, with the dredging works being carried out by Van Oord BV and the specialist immersion works being carried out by Strukton Afzinktechnieken 'Mergor' now known as Strukton Immersion Projects. The design of the immersed tunnel commenced in 2003, construction commenced in 2004 and the Fixed Link opened to traffic in 2010.

The layout of the link was constrained by the main shipping channel in the Eastern part of the link between Jungjuk and Gaduk Island which important Navy traffic uses to access the Navy base at Jinahe. The shipping channel (Eastern part of the link) could not be crossed by bridges due to security concerns and this part of the link had therefore to be constructed as a tunnel. Studies were carried out of a number of options for the tunnel considering both bored and immersed tunnels, with an immersed tunnel concept developed by VINCI Grands Projets and GTM in an early PPP process before the GK Corporation were awarded the concession. This choice of an immersed tunnel for the Eastern part of the alignment due to the security constraints preventing a bridge solution was particularly challenging as the foundation conditions in this section of the alignment consisted of a deep layer of soft clay. In comparison, the Western part of the alignment where the bridges were built had much better ground conditions with stiff alluvial soils over most of the alignment. This would have been a much easier location to construct an immersed tunnel, however the security considerations necessitated an immersed tunnel for the main shipping channel in the Eastern part of the alignment. The tunnel alignment is shown below in Fig. 4 together with the ground conditions and the final foundations solutions used for the tunnel construction.

The design concept for the tunnel was an European style segmental concrete tunnel, this used as its basis the Øresund tunnel linking Denmark and Sweden that had been completed in 2000 which represented the "state of the art" in this type of immersed tunnels at that time. Several of the organisations involved had been involved in the Øresund project and thus could bring in the relevant experience. The design concepts adopted from Øresund included the concrete segmental concept with full section casting of the segments, no use of an external

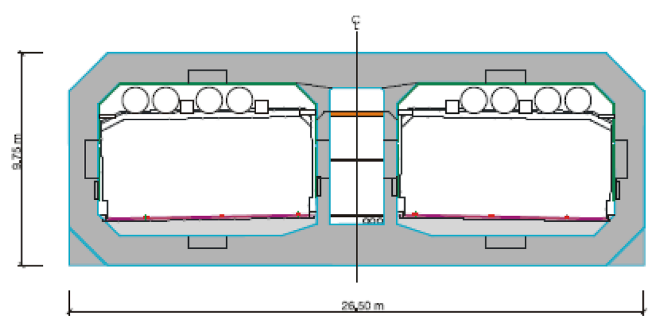

Fig. 2. Typical Tunnel Cross Section, Standard Elements.

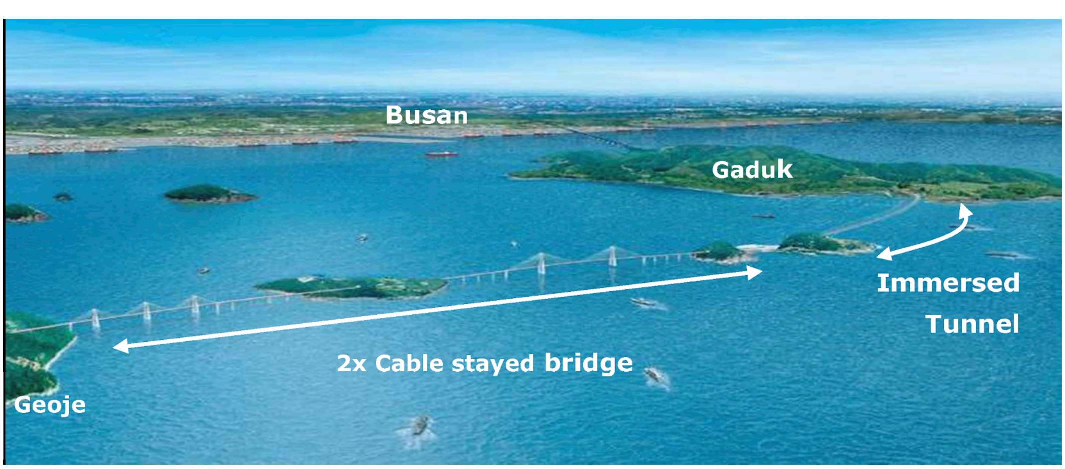

Fig. 1. Site Location.

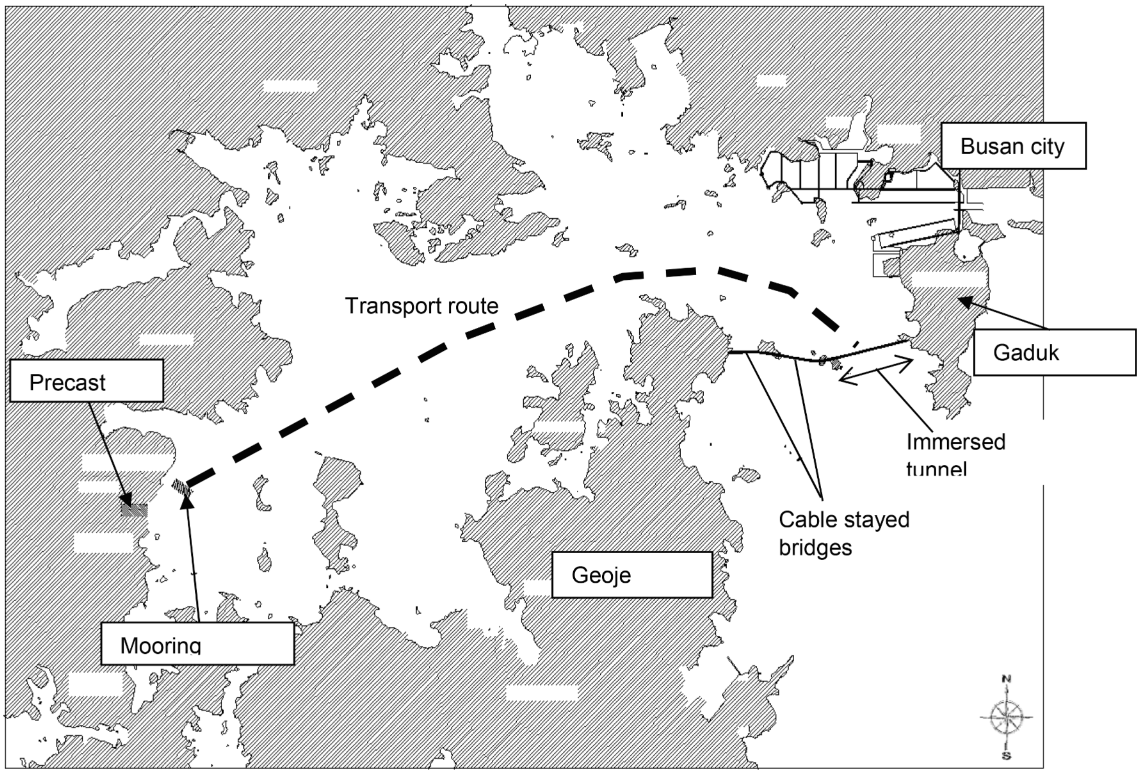

Fig. 3. Site Location overview.

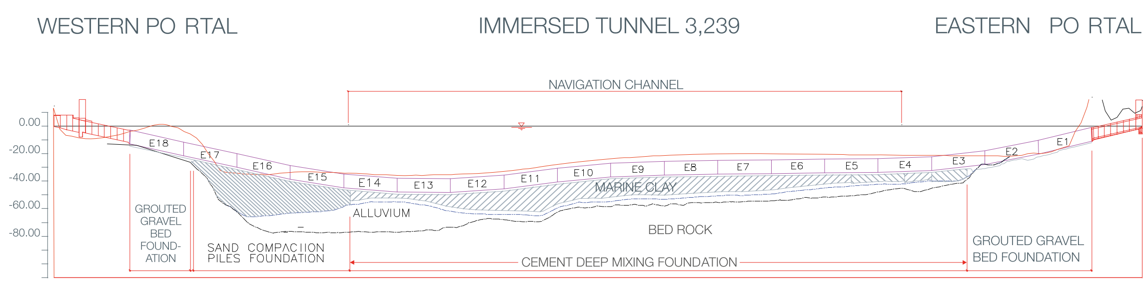

Fig. 4. Immersed Tunnel Alignment.

membrane and reliance on the concrete for watertightness together with the screeded gravel bed foundation. However, Busan was a step further in terms of water pressure from Øresund, with the deepest part of the Busan tunnel at 49 m below sea level, this required further development of the seals for both the immersion and the segment joints. A similar system to that used on Øresund was used for the temporary post tensioning of the elements during transport and immersion was adopted along with many details of the end frames and joints.

The main challenges that the Busan design and construction faced were:

- For the majority of the tunnel alignment it was founded on a soft almost normally consolidated clay. The issues associated with this were not fully appreciated until the main geotechnical investigations were available late in the Basic Design phase. This had major repercussions and required a major redesign at the start of the Detailed Design which included the introduction of ground improvement for the majority of the alignment in order to reduce settlements to manageable values
- A significant seismic loading, as Busan is located in a moderate hazard area with a design rock acceleration of 0,15 g. Both ends of the tunnel were located on the rock, thus fixing the ends whilst the main section of the alignment would experience significant seismic displacement due to amplification through the thick soft clay layer. The seismic design cases introduced significant joint openings which had to be taken into account in the joint design.

The seismic loading had a significant impact on the design of the tunnel at its Western end where it was located on a subsea embankment located on top of the thick layer of soft clay. The seismic loading had to be accommodated in the ground treatment design particularly to deal with the inertia from the embankment.

- Large accidental loading from a sunken ship, from ship impacts in the shallow parts of the tunnel and the design case of a flooded tunnel. These design cases gave rises to large settlements and these accidental loadings had a large impact on the foundation design and the need for a major soil improvement scheme along the majority of the alignment
- The tunnel is in an offshore location facing the Sea of Japan exposed to a combination of wind waves and long period swell waves with significant wave heights up to 9 m. Only in the period May to October were there “wave windows” when the wave and weather conditions were suitable for transport and immersion, particularly governing

were the periods when the swell wave conditions were suitable for immersion. This required an extensive programme of immersion simulations, using both physical and computer models to predict the behaviour of the tunnel elements during transport and immersion. A sophisticated weather and wave prediction system was developed in order to allow weather and wave windows to be defined for the immersion operations together with a "go", "no-go" matrix to be developed based on the predicted behaviour of the immersion system and the predicted weather conditions.
- The large wave loading required very heavy protection works at both ends of the tunnel, with innovative solutions needed to keep the weight of these protection works to a minimum. The resulted in the use of a single layer protection system was needed; however this pushed the design to the limit for such single layer block systems
- The use of a screeded gravel bed foundation layer i.e. similar to Øresund. During Basic Design this was to be a grouted gravel bed, however this solution would have left the tunnel elements exposed to changes in the wave conditions sitting on temporary supports for a significant period until the grouting was carried out. The change to a screeded gravel bed had however many implications for the design and construction. One of the particular difficulties was constructing such a gravel bed a large distance offshore on a soft soil within acceptable tolerances for the element placement and for the structural design to accommodate.
- The immersion works required significant innovations due to the exposed location and water depth which prevented the use of conventional towers on the elements. These innovations included having all the internal systems in the elements remotely controlled and amongst these innovations were the Element Positioning System (EPS), and a sophisticated measuring system to determine the elements position.

As it can be seen from the above, the Busan tunnel had a number of challenges that required a significant step forward in immersed tunnel technology from what had been done before on the Øresund project. Not all these challenges were understood at the start of the design and Busan was a challenging journey for the organisations involved. This paper is intended to describe the key and challenging aspects of the project and references other papers where further details can be found for each of the issues.

2. Wave environment

One of the aspects that makes the Busan Geoje tunnel unique is its offshore location, exposed to the sea of Japan with a combination of wind waves and long period swell waves with significant wave heights up to 9 m, the wave climate is described by Kasper et al. (2008). Extensive hindcast modelling was carried out during the early design stages to transpose the offshore wave conditions to the alignment as the basis for the permanent works design and for carrying out a preliminary design of the tunnel element immersion process. Once these early studies had been carried out it was obvious that the wave climate would have a large impact on the tunnel design both for the permanent works design and for the immersion operations. As well as the impact on the immersion and transport operations as discussed in Section 6, the wave climate had a significant impact on the permanent works design with heavy protection works being required particularly in the shallow water areas at the ends of the tunnel. A heavy single layer protection systems was used, comprising of large Accropods and Coreloc blocks. An extensive effort had to be made in verifying the tunnel stability from exposure to the long period swell waves which induced pressure differences across the tunnel, this was a most unusual condition for an immersed tunnel and is described by Heijmans et al. (2007) and Kasper and Jackson (2008). The consequences of this wave climate for the towing and immersion operations as described in Section 7, mean that for only a few months of the year the wave and weather conditions were suitable for transport and immersion and even within these periods, only certain weather "windows" provided suitable conditions for immersion. This required an extensive programme of immersion simulations, using both physical and computer models to establish the wave conditions that immersion could take place within the limitations of the immersion equipment and the design of the tunnel elements. This proved to be one of the significant challenges of the project, several phases of physical and numerical modelling were carried out with the final phases of both physical and numerical modelling being carried out at Marin in the Netherlands, described by Cozijn and Heo (2009a, b) which was used to derive the allowable range of wave and current conditions and establish a "go, no-go" matrix based on three thousand numerical simulations.

A sophisticated weather and wave prediction system was developed that enabled a practical system to be implemented for decisions to be taken during the immersion planning and immersion operations, this is described below in Section 6 and by Cho and Heo (2012).

Table 1
Summary Marine Clay Classification tests.

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

Table 2
Strength and Deformation properies of the Marine Clay.

Property	Value
Preconsolidation stress $\Delta\sigma$	10–30 kPa
Effective angle of friction	25°
Cohesion c′	3 kPa
Hydraulic Permeability	10^{-9} ms^{-1}

3. Ground conditions

The ground conditions along the alignment are illustrated in Fig. 4, with a soft slightly over consolidated marine clay predominating, this overlies a stiff alluvium which in term overlies granitic bedrocks. The main challenge was the marine clay which required special attention, extensive ground investigations and laboratory testing programmes have been carried out as described by Steenfelt et al. (2008). The field tests comprised 50 boreholes with soil sampling, field vane and SPT tests and 60 CPTu soundings. Classification tests have shown that the clay exhibits high to extremely high plasticity and the classification results are summarised in Table 1. The strength properties of the clay were assessed from the results of CPT and triaxial tests, based on the SHANSEP approach defined by Ladd and Foott (1974). The derived preconsolidation stress $\Delta\sigma$ together with the derived strength and deformation properties are summarised in Table 2.

The clay was thus a highly compressible material with a very low shear strength, how to construct the tunnel foundation on this material would represent one of the large challenges for the project.

4. Tunnel alignment and foundation

One of the key elements of the tunnel design & construction was the foundation conditions along the tunnel alignment as shown in Fig. 4. Dealing with foundation issues proved to be one of the major project challenges and is described by Jackson and Rotwitt (2012), Kim et al. (2009), He et al. (2016) and Kasper et al. (2009). Each tunnel element consists of eight 22,5 m long segments with a total of 18 tunnels elements.

The joints between the segments are formed by a large waterstop together with concrete shear keys that control vertical displacement and with a secondary Omega seal as a backup seal. Between the tunnel elements there are the traditional Gina gaskets with a secondary Omega seal as a back up seal, both types of joints have limited capacity for longitudinal movement with a significant part of the longitudinal movement capacity being taken up with thermal movement. Therefore in order to ensure the water tightness of the tunnel, joint openings had to be limited. The forces in the segment joint shear keys had also to be limited as there was a maximum load that could be designed for with the tunnel cross section. Both these issues required that the differential longitudinal settlements and thus the total settlement had to be carefully controlled and kept within a relatively limited range of the order of 100 mm.

Without soil improvement of the marine clay below the tunnel, long-term tunnel settlements of up to 350 mm were predicted during the Basic design and after the Basic design was completed the design was essentially put on hold until robust foundation concepts could be found that limited settlements, joint opening and shear key loads to acceptable levels.

Along the alignment three different type of foundations are applied as shown in Fig. 4.

4.1. Foundation on rock, Eastern Portal and Western Portal

At the Eastern end of the alignment the first two elements and part of the third element (E1, E2 & E3) were founded on rock. For this section of the alignment a gravel bed foundation layer was placed on the rock using conventional techniques and the space between the gravel bed and the tunnel grouted through a number of pipes in the base of the tunnel. A similar approach was taken for the final element E18 at the Western Portal.

4.2. Sub-sea embankment, Western Portal

At the Western end of the alignment, the tunnel (E15, E16 and E17) is placed on a sub-sea embankment which lies on approximately 30 m of the soft marine clay. This was quite an unusual foundation for the immersed tunnel particularly when exposed to the large wave loading and to the significant seismic loading present in the area with a 0,15 g horizontal acceleration used in the design. There were the obvious settlement issues associated with the large loading from the embankment and stability issues also given the low undrained strength of the clay.

During the Basic Design the deep layer of soft clay was anticipated to be dredged and replaced by sand fill, this was reconsidered during the detailed design and various options for soil improvement were considered. A solution using sand compaction piles (SCP) was chosen, this is a quite common technique in Korea and Japan where columns of compacted sand are formed for the full depth of the clay layer. This technique was in use in the nearby Busan New Port where it was used to provide the foundations for quay wall caissons. The sand columns are used with relatively large replacement ratios (75%) and together with preloading provide very good settlement control as well as strengthening the soil to provide the required stability for the subsea embankment. SCP was chosen over the alternative deep cement mixing (DCP) which is another common ground improvement technique in the region which was used for the main tunnel alignment. However, cement mixed columns are relatively brittle and are prone to progressive failure from seismic loading. This is a particular problem with the high inertia of the sub-sea embankment placed on the thick layer of soft clay in this area of the alignment and was the reason for the adoption of SCP rather than CDM for this section of the alignment. A major SCP trial was carried out in this area to validate the applicability of SCP method, the trail incorporated trial loading and monitoring of settlement and displacement. Once it was confirmed that SCP was suitable, detailed design of the ground improvement was undertaken. The design was carried out largely with Finite Element Methods using the software Plaxis in order to model the relative complex construction sequence, the preloading of the ground improvement and also deal with the stability of the embankment during construction and in the permanent condition. The design of the SCP foundation provided:

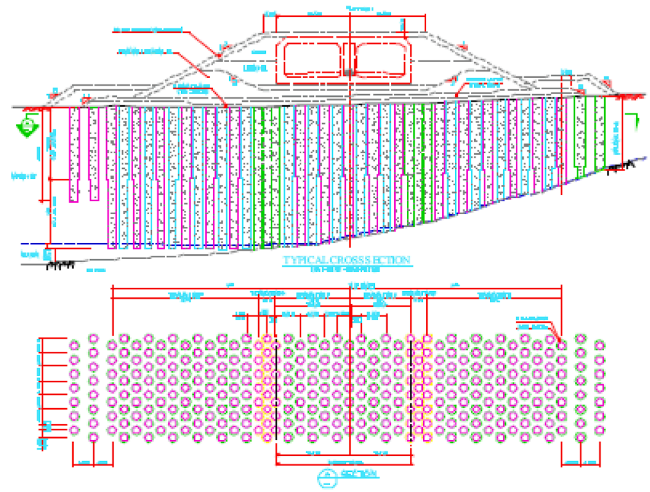

Fig. 5. Typical Sand Compaction Pile Layout for the subsea embankment.

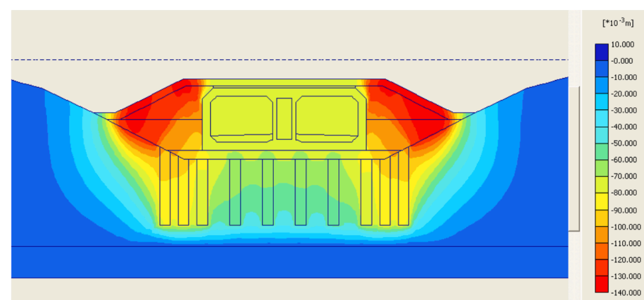

Fig. 6. Typical Settlement Prediction from Plaxis FEM model.

- Sufficient preloading of the SCP by a preloading embankment, to control long term settlements.
- Sufficient safety against failure of the preloading embankment during construction.
- Sufficient safety against failure in the final situation considering the effect of wave and earthquake loading.
- Control of the irrecoverable displacements of the embankment / tunnel due to earthquake loading.

The typical Sand Compaction pile layout is shown below in Fig. 5 which illustrates the distribution of the sand piles and the greater treatment ratio below the more highly loaded areas.

In the subsea embankment area the tunnel was founded on the screeded gravel bed which is discussed in Section 4.4.

4.3. Main tunnel alignment

The typical foundation concept for immersed tunnels is a direct foundation on the underlying strata with a gravel bed, grouted foundation or jetted sand foundation. Typically the foundation loading from the tunnel placed in a dredged trench is low, only a few kPa greater than the in-situ stress prior to dredging of the trench which results in very limited settlements. 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

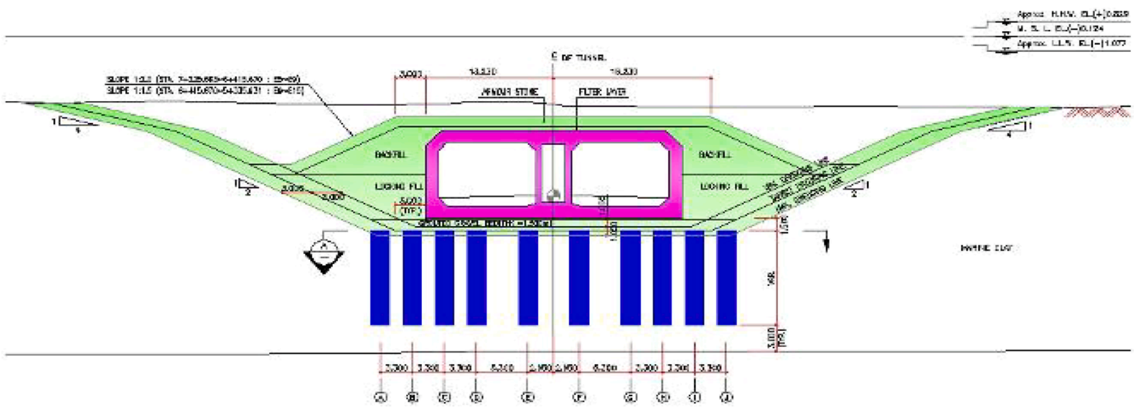

Fig. 7. Typical Layout of tunnel trench with Cement Deep Mixing Foundation.

mm with the theoretical trench profile. Once the tolerances in the trench profile were considered, together with the likely variations in the soil properties, significantly larger settlements were observed from the FEM predictions. The modelling of this behaviour is described by Jackson and Rotwitt (2012) and by Kasper et al. (2009). The stresses beneath the tunnel vary across the trench/tunnel cross section with low stress beneath the centre of the tunnel and much higher stress beneath the heavy backfill. The stresses beneath the backfill far exceed the pre-construction in-situ stresses and consequently lead to high settlements in these zones which essentially drag the tunnel cross section down. The concentration of settlement beneath the backfill is illustrated in Fig. 5 where an extract of a typical Plaxis model is shown, in this model the ground improvement is included but still shows the larger settlements below the backfill where the stresses are high and the loads being taken into the deep mixing columns.

The likely variations of the trench profile were assumed to be ±1 m vertically and ±3 m horizontally, based on extensive discussions with the dredging contractors. This assumption of the variation was confirmed during dredging operations with a Trailing Suction Hopper Dredger (TSHD), the HAM 316 from Van Oord where a large effort was expended on maintaining the trench profile using the best available survey and positioning techniques. This shows the importance of considering the construction technique at an early stage of design. When the variations of the trench profile and particularly the increased amount of backfill were modelled, it became immediatly apparent that the settlements increased markedly and that the backfill volume was the dominating factor in the settlement behaviour. When modelling using the expected trench profile and backfill consolidation settlements of the order of 300–400 mm were predicted. Creep i.e. settlement under constant loading was also expected to induce further long term settlements and is particular issue with the structured marine clay present. Such settlements were considered to be larger than acceptable, as they would have led to unacceptable joint openings between the tunnel segments or between the tunnel elements and unacceptable loads in the shear keys at the segment joints. 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 different types 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 soil improvement with cement deep mixing (CDM) was chosen for the main section of the tunnel in the dredged trench. The rational for its 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 Figs. 5 and 7, 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 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 background for the selection is described by Jackson and Rotwitt (2012).

During the period when the design was on hold pending resolution of a suitable foundation concept, solutions were looked at into the use of a light backfill around the tunnel, such as the use of light aggregates or hollow blocks. These were discarded on practicality issues together with not providing the needed resistance to wave and seismic loading on the tunnel.

The CDM foundation was installed after dredging of the tunnel trench, the installation causes some ground heave, and in some cases the tops of the CDM piles were higher than required. Prior to gravel bed placement the heaved material was removed together with the tops of any high CDM piles as they had a low strength that could easily be removed by grab dredging and the trench base restored with a tolerance of ±1 m. The low treatment ration used prevent major ground heave which was another benefit of the chosen method.

The CDM foundation concept proved to be a considerable success, it was undertaken by an experienced Korean contractor with approximately half a million cubic metres of the clay treated in one year. 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 30–50 mm indicating that the foundation was performing as expected.

4.4. Gravel bed design and construction

At the Basic design stage a grouted gravel bed was envisaged with a gravel bed placed by conventional methods and the gap between the gravel bed and tunnel filled with grout from tubes placed in the tunnel

Fig. 8. Gravel bed placement barge KUS-ISLAND from Eunsung O&C.

base slab. This method however has significant disadvantages particularly at the large depth in the deepest parts of the alignment and has particular risks associated with grouting at the highwater water pressures. This approach also left the tunnel exposed to changes in the wave climate whilst it was on temporary supports with only 6% overweight. This was considered to be an unacceptable risk. The foundation was changed to a similar approach to the Øresund tunnel where a screeding (known as scrading on the Øresund tunnel) method was used and the tunnel elements placed directly on the gravel bed and the locking fill quickly placed. This approach has been used on a number of immersed tunnels since the Øresund project such as the second Benelux tunnel, A73 Roertunnel and the Bjövika tunnel in Oslo, it is also now extensively used for wind turbine foundations. The gravel foundation construction is described by Lee (2012).

The screeding method simultaneous places the gravel and levels it by using a fall pipe filled with gravel which is moved as it places the gravel, the level of the base of the fall pipe is controlled within the tolerance required. Berms of gravel are placed by the fall pipe with gaps between the berms. There are however significant differences between Busan and Øresund with the much larger depth (up to 49 m in Busan) to the trench base and the offshore location exposed to a much more severe wave environment than Øresund. On the Øresund project a floating barge was used, its location controlled by spud legs with a sophisticated control system used to control the level of the base of the fall pipe in order to compensate for vertical movement of the barge. For Busan a different approach was taken and a custom designed jackup barge was used that could cope better with the wave conditions. The soil conditions for the Busan tunnel were also fundamentally different from Øresund, with the majority of the Busan tunnel alignment founded on the soft clay, in comparison Øresund was founded on either a hard glacial moraine or limestone. The jackup barge allowed a much more stable platform for the construction of the gravel bed with the jackup legs being able to be adjusted to compensate for settlement of the legs in the clay and for operations to be carried out in a wide range of wave conditions. The barge is shown below in Fig. 8.

One of the challenges with the placement of the gravel bed was to achieve an acceptable placement tolerance for the top of the gravel bed. If for instance high spots were formed in the gravel bed, these would have an undesirable influence on the tunnel structure and induce unacceptable bending moments. Also if the gravel bed induced torsion in the tunnel elements due to systematic placing errors, this could also induce unacceptable stress in the elements. A significant effort was carried out during the detailed design to look at the influence of the gravel bed placing tolerances on the structural performance using 3D FEM and considering various statistical approaches to the distribution of the uncertainties in the gravel bed level. This work built on work that had been done on the Øresund project, however one of the key differences between Øresund and Busan was the foundation stiffness below the gravel bed. As discussed above Øresund, was founded on stiff moraine or limestone and thus the thickness and level of the gravel bed could have a large influence on the stress in the elements, with for instance high spots having a large effect.

In contrast, the Busan tunnel was founded on a soft marine clay which for the tunnel section in a trench used a partial depth cement deep mixing (CDM) foundation. The cement mixed columns penetrated to approximate 75% of the thickness of the clay layer and provided a load transfer solution with the load being transferred to the stiffer deeper part of the clay. The columns did not need to reach the underlying stiff alluvium or rock. The foundation was thus a floating foundation and should high points occur in the gravel bed, then these would be pushed down and the load redistributed. High spots in the gravel bed were therefore not a problem, other tolerances issues were also not such a concern with this type of floating foundation. In addition, the columns were of low strength, typically with an undrained shear strength of 500 kPa, thus if a column was over drilled and reached the hard strata it would fail rather than overstress the tunnel. Variation in the thickness of the gravel was also not a problem as would be the case for a stiff foundation, due to the relative stiffness of the gravel bed and the underlying soft foundation.

The tolerance of the gravel bed was in the range 25–50 mm with a tolerance target of 35 mm, which on a stiff foundation such as the Øresund tunnel would have been at the limit of what was acceptable, however for Busan this could easily be accommodated. The main issue became placing the gravel bed with the correct average level relative to the previous element's foundation. This was able to be carried out satisfactorily, with the gravel bed installed slightly high. The Element

Fig. 9. Dry dock.

Fig. 11. Anchor block.

Positioning System (EPS) could by moving the element longitudinally back and forth displace the higher local spots and move gravel into the spaces between the berms and adjust the level to match the previously placed element. This was one of the key innovations and allowed the correct placing tolerances to be achieved despite the survey challenges such as not having survey towers, the distances offshore and the soft foundation.

4.5. Backfill

Due to the exposed environment and risk of wave conditions causing movement of an element exceeding the resistance caused by the 6% ballast weight after placing it was important that the locking fill was placed within a short space of time after placement of the element. The use of the screeded gravel bed allowed this to happen and was the main driver for the change from a grouted foundation. This was achieved by the use of the gravel bed placing barge that could rapidly place large volumes of backfill. The remaining backfill and tunnel protection was placed later by conventional grab placement equipment.

5. Construction

5.1. Concrete and tunnel casting

The tunnel elements each 180 m in length were constructed in segments of 22,5 m in length cast as continuous full sections with alternate sections match cast sequentially, the design of the tunnel permanent works is described by Tonnesen and Jackson (2012). The full section casting largely reduced early age thermal cracking through the section as there were no construction joints within a segment. Some restraint was applied where the segments were cast against a previously cast segment particularly around the shear keys that crossed the segment joints. This was managed by early age thermal calculations, the choice of a low heat concrete with a high proportion of Ground Granulated Blast Furnace Slag (GGBS) cement replacement and control of temperature within the casting shed.

Any contraction (shrinkage) of the segments was taken up by the temporary post tensioning system prior to the elements being floated up in the casting basin.

The Busan tunnel was designed for a 100 year service life and was one the first immersed tunnels to apply the probabilistic service life design approach to achieve this. The probabilistic approach was first developed in the DuraCrete research project late 1990's which has been subsequently embodied in FIB Bulletin 34, fib 2006, and is in the process of being included in the Eurocodes. The application of this durability approach to the tunnel is described by Edvardsen et al., 2006. This probabilistic approach considered the governing deterioration mechanisms which was chloride induced reinforcement corrosion and specified the durability requirements such as the chloride diffusion coefficient in order to achieve the required service life. The concrete mixes were developed at DAEWOO's Institute of Concrete Construction Technology (DICT) in Seoul and included the advanced durability testing.

Since the Busan tunnel, this durability approach has been applied on many large infrastructure projects and has become the recognised durability approach on such projects.

5.2. Layout dry dock

The construction of the tunnel elements was carried out in a dry dock (Fig. 9) on the western side of the Jinhae Bay, about 40 km from the immersion area. The casting of the tunnel elements has been carried out in two batches of four and two batches of five tunnel elements (Fig. 10). The first 16 tunnel elements were 180 m long, 10 m high, 26 m wide and weighed about 48.000 tons. The final two tunnel elements were equipped with a climbing lane and were 2 m wider. The tunnel elements were equipped with steel, reusable bulkhead panels on both ends.

The ballast water system inside the tunnel elements consisted of six water tanks with pipes and pumps. After finishing each batch, the dry dock was flooded and the dock gate removed.

5.3. Mooring location

The tunnel elements were floated and trimmed one by one. Transport inside the dry dock was carried out by a fixed winching system consisting of seven winches of 20 tons up to 35 tons.

Once the tunnel element was moved through the dock gate, four tug boats were used to transport the tunnel element to the nearby (2 km)

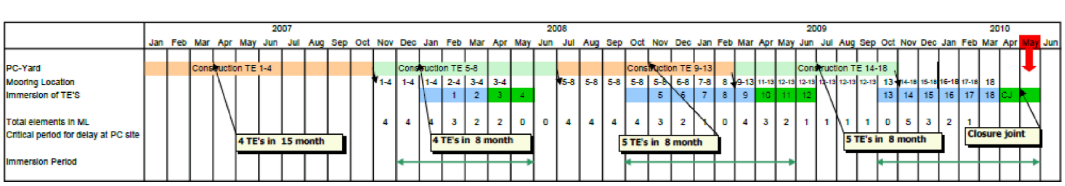

Fig. 10. Construction schedule Tunnel Elements.

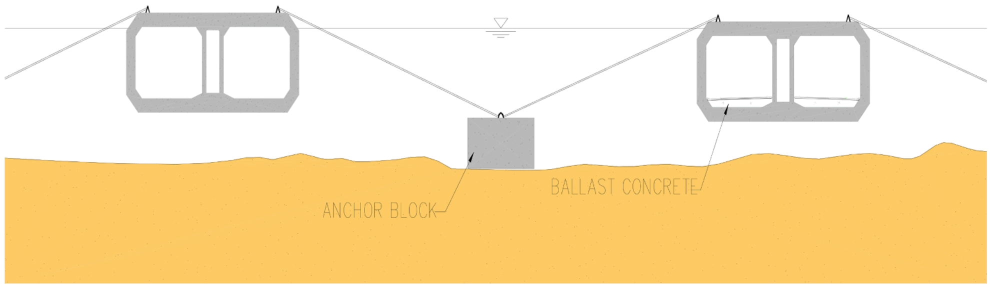

Fig. 12. Anchor block with Tunnel elements.

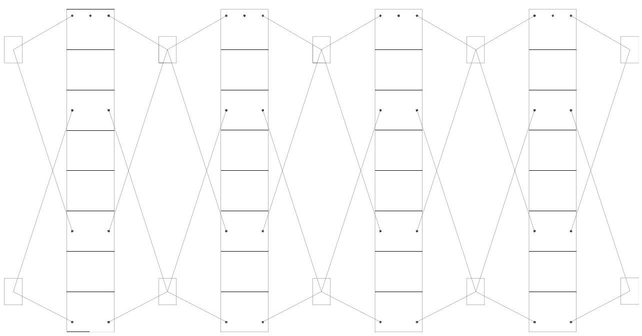

Fig. 13. Mooring configuration.

mooring location (Fig. 14). A maximum of six tunnel elements at a time moored on pre-installed anchor block (Fig. 11) were stored at the mooring location (Figs. 12 and 13). Due to the sheltered area of the mooring location, the tunnel elements were not affected by swell waves and could be stored there throughout the year.

With the sequence of the construction and immersion it was for seen that a minimum of two Tunnel Elements would be Immersed during the construction period of the following batch. With that a storage of six Tunnel Element at the mooring location was sufficient (Fig. 10).

6. Immersion operations design

The Basic Design included both the permanent works and a preliminary design of the immersion operations in order to include the relevant design assumptions and provisions into the tunnel design. During this Basic design phase, no immersion contractor had been appointed and thus the designer had to make a number of assumptions regarding how the immersion contractor would carry out the immersion operations and during this period the first phase of physical modelling was carried out to determine the loads on the tunnel elements during towing and immersion. Also, during the Basic design phase, numerical modelling was carried out to determine the expected wave environment for towing and immersion operations.

The immersion operations design proved to be a challenging task, as the project was going beyond what had been done before and without the immersion specialist contractor being onboard made the process more difficult. The Detailed Design was commenced using the assumptions that had been derived from the Basic Design immersion planning and it was not until part way through the Detailed Design that Strukton Immersion Projects was engaged as the immersion contractor. A significant amount of engineering had to be done by the immersion contractor to take forward the Basic Design immersion operations design and prepare the detailed design of the immersion operations and equipment. This turned out to be a very challenging process, time was also very limited. The final phase of physical and numerical modelling was carried out during this period to determine the loads on the elements and movements for the final range of wave and current conditions, this had to be conducted in parallel with the immersion equipment design.

Conservative design assumptions were made during the Basic Design phase for the design of the element lifting lugs, bollard loads and for the temporary post tensioning of the elements during towing and immersion. Essentially these were designed for the maximum practical loads based on the Basic Design modelling and experience from other projects and fortunately these assumptions provided sufficient capacity except for some minor redesigns.

The design of the immersion operations and the immersion works are described by Vlaanderen-Oldenzeel et al., 2010, Louis, 2008, Jille and De Groot, 2009 and Bruins Slot et al., 2008.

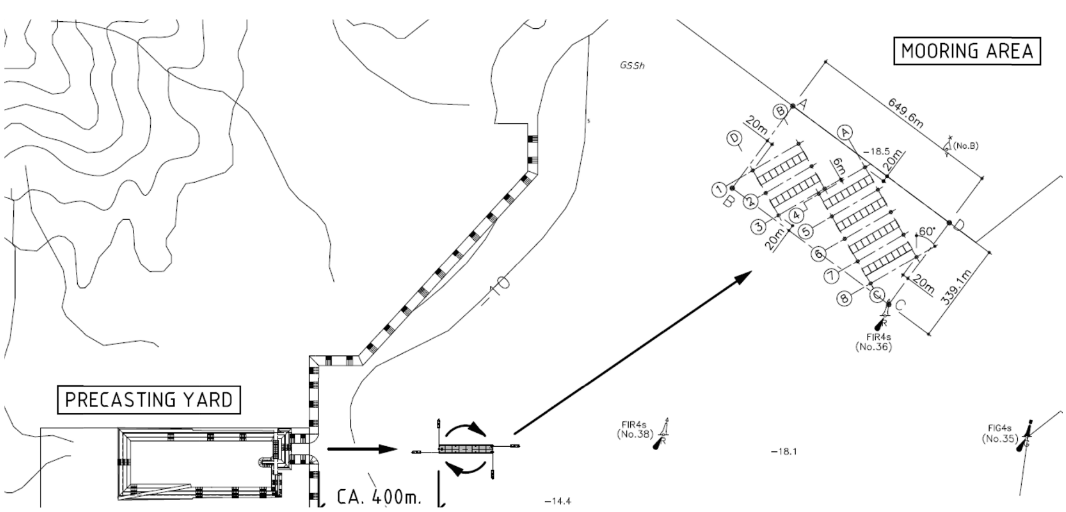

Fig. 14. Overview PC Yard and mooring location.

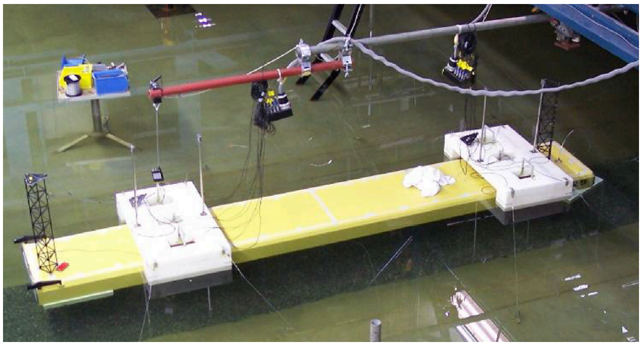

Fig. 15. Scale model for hydraulic tests.

Tunnel Element at 0.5 m above the Gravel Bed
Water Depth 23.0 m
Current - Vc = 0.0 m/s from South
Swell - Hs = 0.4 m, Tp = 8.0 s from South
Wind Seas from South

0,80						
0,70					red	
0,60				orange		
0,50						
0,40		green				
0,30						
0,20						
Hs / Tp	3,0	4,0	5,0	6,0	7,0	8,0

Fig. 16. Workability Criteria table.

6.1. Weather window analysis

The capacity of the immersion spread had been determined to resist swell wave heights of up to approximately 0,5 m. This means that immersion only could take place in the period from October to May. Outside this period, extreme waves of up to 8 m could occurred because of typhoons. Due to the chosen capacity of the immersion spread, the operations were restricted by criteria linked to wave and current effects. Restrictions were present on bending moments of the tunnel element, line forces and motions of the tunnel element just before joining the previous tunnel element, differences between motions of the immersion pontoon and the tunnel element, and movements of the tunnel element directly after immersion.

6.2. Model testing

As the tunnel element was very sensitive to wind and swell waves, the response of the system to waves was tested and a wave forecast system was set up. On the basis of this forecast system, Go / No-Go decisions were made at several moments before and during the immersion operation. The basis of this decision system is explained below.

An extensive hydraulic model test programme was carried out to determine the effect of waves on the tunnel element during several stages of the immersion procedure. In a maritime laboratory, a scale model (Fig. 15) of the immersion spread was tested with several combinations of wind and swell waves. For these model tests, the immersion trench was modelled together with the tunnel element, the immersion pontoons and the winch wires. The mooring and contraction wires were modelled with a stiff string and a calibrated spring to obtain the specified stiffness of the wires in the taut situation. During the model tests, the motions of the tunnel element and immersion pontoons in all six directions of movement and the forces in the winch wires were registered.

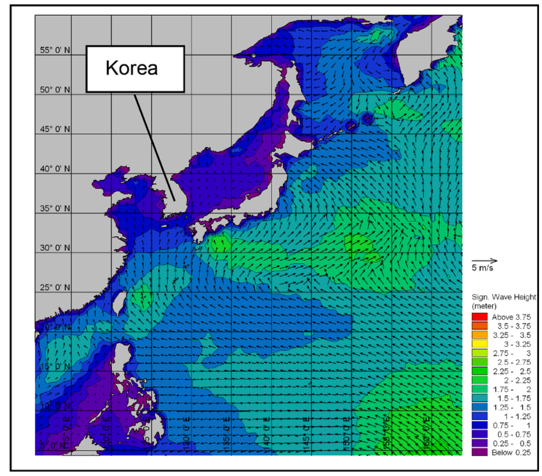

Fig. 17. *Swell wave prediction (large* area).

Subsequently a numerical model was constructed in a multi-body time-domain simulation tool. This numerical model was calibrated so that it reproduces the results from the scale model tests as accurately as possible. This calibrated model has been used for over 5000 simulations considering a large number of environmental conditions divided over three sets, with variation of wind wave height, wave period, wave direction and swell conditions. Current forces were also added to this numerical model. Statistical analysis results (mean value, standard deviation, minimum and maximum) were generated for each case. The results per combination of conditions were compared with the criteria value and visualised in numerous of workability criteria tables (Fig. 16). If none of the limiting values are exceeded, a green colour is shown. Exceeding of availability and failure constraints are visualised with an orange and red colour.

6.3. Wave forecast system

For the project location, a detailed wave forecast system was developed based on the world wide MIKE 21 SW wave model. This new generation spectral wind and wave model is based on unstructured meshes. The model simulates the growth, decay and transformation of wind generated waves and swell in offshore and coastal areas.

The wind and wave model covered a large area around Korea (Fig. 17), with a fine mesh in the area close to the immersion location. The forecast system for Busan-Geoje Fixed Link provided twice daily hourly forecasts for a five-day horizon of the following parameters:

- Wind speed and direction
- Significant wave height, period and direction
- Swell wave height, period and direction

Since 2004, wave measurements were taken by a wave rider station close to the tunnel location. These data sets were used for calibration and validation of the forecast model. After the first immersion season (January 2008 – May 2008), the data was compared with the given forecasts. On basis of the differences, a "Model Response Function" was derived to give better predictions of the waves. This value was dependent on wave direction and wave height. The accuracy of forecasts demonstrably increased in time. The forecast data was made visual in a wave prediction graph (Fig. 18) and was available on the internet.

6.4. External positioning system (EPS)

From the model test results, it appeared that the motions of the tunnel element were relatively high during immersion, even when the

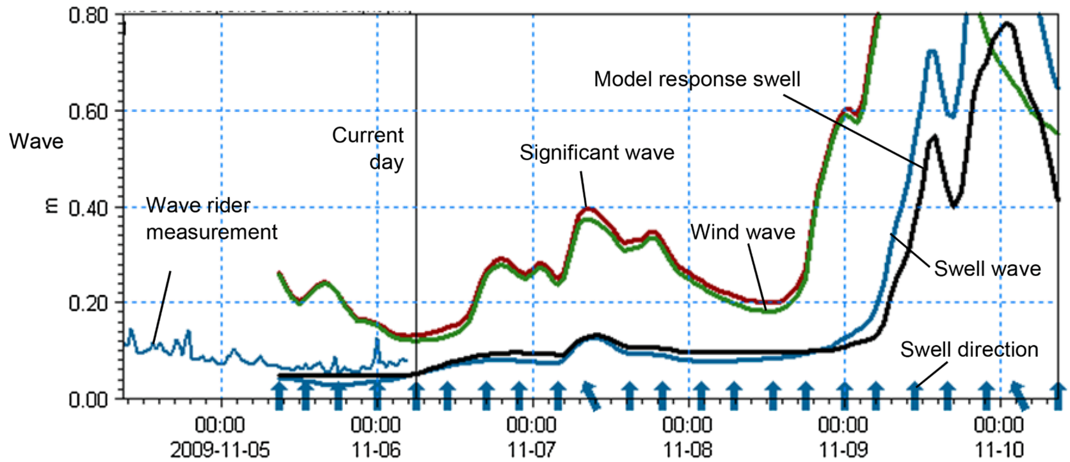

Fig. 18. Wave prediction graph.

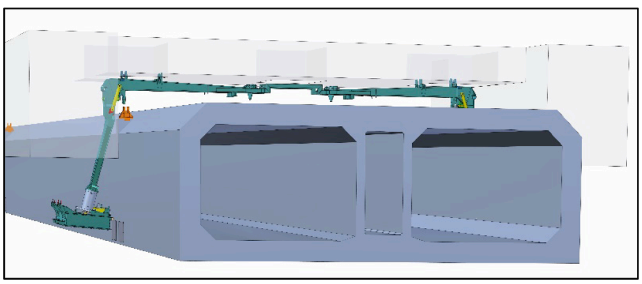

Fig. 19. EPS on element.

Fig. 20. Immersion pontoons at mooring location.

tunnel element was close to the seabed. To prevent the tunnel element from colliding with the previously immersed tunnel element, the external positioning system (EPS, Fig. 19) was developed. As well as its positioning purpose, the EPS was also equipped with a re-alignment system. Re-alignment with hydraulic jacks in the immersion joint was not possible due to the great depth and the length-width relation of the tunnel elements.

7. Immersion operations

7.1. Transport

The tunnel elements were prepared for the immersion operation at the mooring location. Prior to the transport, two immersion pontoons were positioned over the tunnel element (Fig. 20). The pontoons were of the catamaran type and consisted of a main deck (42,5 m × 24 m × 2,5 m) and two floaters (36 m × 6 m × 6 m). The pontoons were each designed for at least 1000 ton pulling force. The Element Positioning System (EPS) was mounted underneath the pontoons. As well as on the pontoons, the EPS was installed on the lifting lugs of the tunnel element.

On the tunnel deck, bollards, lifting lugs, sheaves and a landing tower were installed, as well as guide beams on the primary side and catches on the secondary side of the tunnel element. The landing tower was used for locking the self-propelled diving bell to the tunnel element, in case emergencies occurred during the immersion operation. The guide beams and catches were used to guide the tunnel element sideways during the immersion process. Equipment for the remote-controlled ballast water system and the immersion survey systems were installed inside the tunnel element. After the installation of the pontoons, the deck layout and the systems inside the tunnel element was intensively tested and verified with extensive checklists.

As soon as the predicted weather and wave conditions were within the limits and the final go decision had been made, the total system was transported to the immersion location. Transport was carried out by four tug boats varying from 2400 hp to 3600 hp. During the transport, the immersion pontoons were pretensioned to the tunnel element with 200 tons per suspension point.

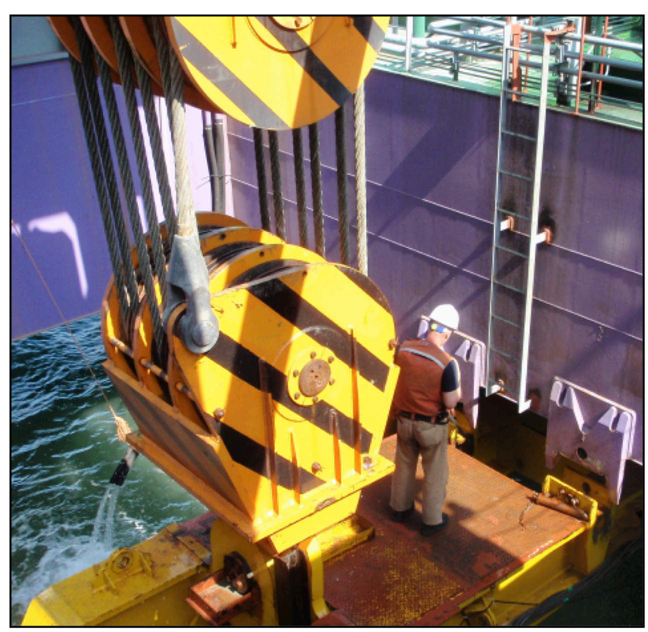

Fig. 21. Travelling block connected to EPS.

Fig. 22. Manta VLA anchor before installation.

7.2. Anchoring system

A spread consisting of 18 winch wires was applied in order to be able to carry out a controlled and secure immersion operation:

- Mooring wires (four per pontoon) were connected to seabed anchors outside the trench. These wires were used for positioning the immersion pontoons above the target position of the tunnel element. The mooring winches had a capacity of 35 tons each.
- Contraction wires (three per pontoon) ran from the pontoon, through a moon pool in the main deck, to the sheaves on the tunnel deck. Two wires ran diagonally to the corners of the tunnel element; one wire ran in longitudinal direction. These wires were also connected to seabed anchors. The contraction wires were used for positioning the tunnel element throughout the immersion process in sideways and longitudinal direction. The contraction winches had a capacity of 60 tons each.
- Suspension wires (two per pontoon) were connected by travelling blocks to the EPS (Fig. 21) on top of the tunnel element. These wires carried the overweight in the tunnel element during immersion. The suspension wires were reeved 13 times. The suspension winches had a capacity of 60 tons each.

The used seabed anchors were plate anchors of the Manta VLA (Vertical Loaded Anchor) type, an anchor in which the traditionally rigid shank has been replaced by a system of wires connected to a plate (Fig. 22). The anchor was designed to resist vertical (or normal) loads and was suitable for a configuration with taut wires. As with a conventional drag embedment anchor, the anchor was installed by an anchor handling vessel with a horizontal load to the mud line in order to obtain the deepest penetration in the seabed of up to 15 m. At this depth by vertical (or normal) loading, the maximum soil resistance was mobilised. This anchor was suitable for soft soils as present at the immersion location. The anchors were recovered after each operation and re-placed at the position of the next tunnel element.

As soon as the tunnel element arrived at the immersion location, the winch wires were connected one by one. The mooring and contraction wires were installed in a taut configuration in order to reduce the motions influenced by waves to a minimum (Fig. 23).

7.3. Immersion process

After taking in the necessary water for the 2% overweight, the tunnel element was lowered to the gravel bed. The immersion pontoons fully carried the immersion loads during the immersion process. The connection with the previous tunnel element was done using a Gina gasket and an immersion joint which was emptied. An Omega Gasket was mounted at a later stage for extra safety with regard to water tightness.

The stability of the tunnel element after immersion was increased by filling the water tanks up to 6% overweight. Subsequently, a gravel fill was placed on both sides to reduce influences induced by waves. Later on, the tunnel has been covered with rocks and concrete blocks to protect the tunnel against high swell waves resulting from typhoons.

As no access shaft was used in this project, all equipment inside the tunnel element was remotely controlled or monitored from one of the immersion pontoons. For example, the valves of the ballast tanks were equipped with actuators and were operated from the control unit on the secondary immersion pontoon. Water levels inside the ballast tanks were monitored by using level transmitters. Dome cameras were installed inside the tunnel element to check the ballast tanks and bulkheads during the immersion process. They were also used to track people inside during the immersion preparations.

All the data from the above systems and the immersion survey system were directed through an umbilical, running from the secondary bulkhead to the umbilical winch on the secondary immersion pontoon. Furthermore, all winches on the immersion pontoons were centrally controlled from two consoles in the command unit. In this command unit, all the relevant data was presented on screens and computers, varying from the immersion survey and ballast water data, forces in the immersion winches, up to the latest wave and weather forecast data. With this information and the control systems, the immersion commander had a good and clear overview of the situation.

7.4. Survey

The survey of an immersion operation is usually carried out by a Tachometric System consisting of three Total Stations on shore, measuring four prisms mounted on an access shaft and an alignment tower. This system functions appropriate up to a distance of 800 m. For this project, the tunnel elements with a depth higher than 30 m the access shaft and alignment tower could not be used as result of the

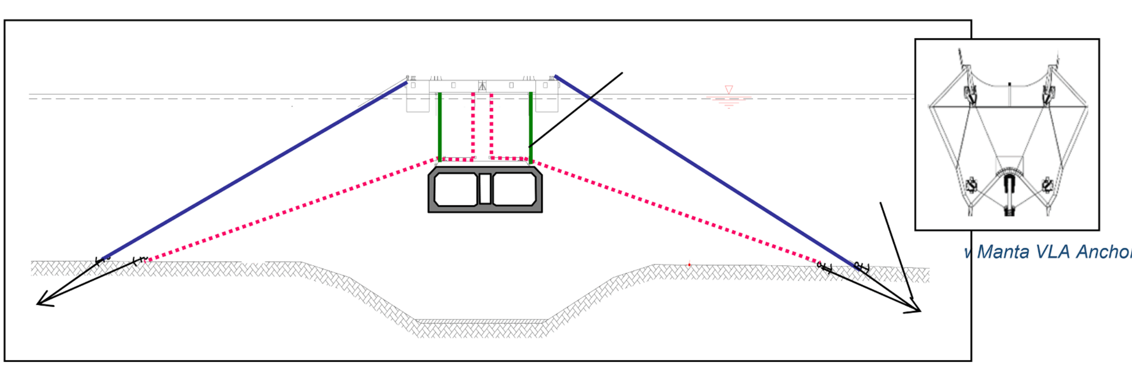

Fig. 23. Immersion spread.

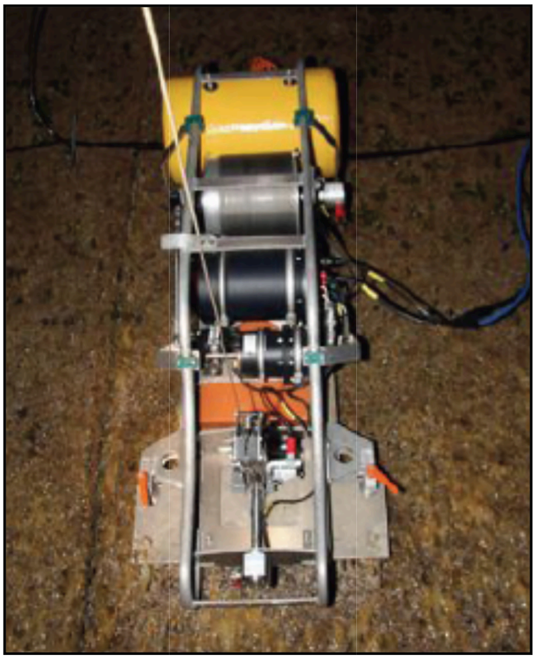

Fig. 24. Taut wire system.

Fig. 25. Distance sensor.

Fig. 26. EPS frames after construction.

current force. Therefore a new underwater survey system was developed.

The new system consisted of a combination of new and existing techniques with an increasing accuracy from the transport phase up to the joining of the tunnel elements.

Transport: A GPS system on the immersion pontoons was used for the transport phase and for the positioning of the tunnel element above the immersion trench. In this phase, all anchor wires were connected and the final immersion preparations were carried out. These included: the removal of Gina protection, the installation of immersion survey equipment on the bulkhead and the testing of the immersion systems. The accuracy of this system was ±1 m.

Approach: To get the guide beams within the tolerances of the catches, a taut wire system (Fig. 24) had been designed. The taut wire is an instrument running a tensioned steel wire up and down on a drum. The taut wire unit was attached to the primary bulkhead of the tunnel element and the wire was connected to the secondary side of the previously immersed tunnel element. The taut wire read the length of the wire, as well as the angles of the arm guiding the wire. Information about the secondary side, related to the primary side, was provided by a gyrocompass. The accuracy of this taut wire system was ±5 cm.

A USBL (Ultra Short Base Line) system was used as a backup of the taut wire. This acoustic survey system consisted of a transducer and several transponders. The transducer, mounted on the primary side of a tunnel element, transmitted an acoustic signal. This signal was received by transponders mounted at known positions on the previously immersed tunnel element. The transponders replied to the transmitted signal with their own acoustic tone which was then received back at the transducer. After correcting the sound velocity differences and other variables, the angles and distances from the received replies were recalculated to the tunnel element's position. The accuracy of this system was ±10 cm.

In the final phase of the immersion process, the Gina gasket was pulled against the steel end frame to obtain the initial water tightness, necessary to empty the immersion joint. For this phase, distance sensors (Fig. 25) had been developed to provide accurate measurements. With their range of approximately 25 cm, the four distance sensors were extended just before the moment the Gina profile touched the previously immersed tunnel element. The millimetre accurate readings of these sensors were used in several ways. The reading of the stroke was a direct indication of the distance. Using the four distance sensors at the corners of the primary bulkhead, the position of the secondary end could be calculated using the differences in readings of the sensors. The accuracy of this system was ±0,2cm.

End survey: As soon as the bulkhead doors were open, an alignment measurement was taken with a total station to confirm the final position of the tunnel element. If required, the tunnel element could be re-aligned sideways at the secondary end with the EPS system.

The taut wire system and distance sensors were removed out of the immersion joint after the immersion operation from inside. The USBL system was removed by divers.

7.5. Element positioning system (EPS)

The EPS consisted of two large portal constructions which were placed over the primary and secondary sides of the tunnel element (Fig. 26). The EPS was connected to the travelling blocks of the pontoons and the lifting lugs on the tunnel element. The 10 m high legs of the frames were equipped with two vertical lifting cylinders of 800 tons each. The base plates were equipped with two horizontal positioning cylinders with 250 ton capacity and were deployed on the gravel bed as soon as the tunnel element was installed on the sea bed. The base plates were connected to each other with steel wires running underneath the tunnel element. The steel wires prevented the base plates from sliding aside when loaded by horizontal forces. When the guide beams were in reach of the catches, the tunnel element was placed on the gravel bed at

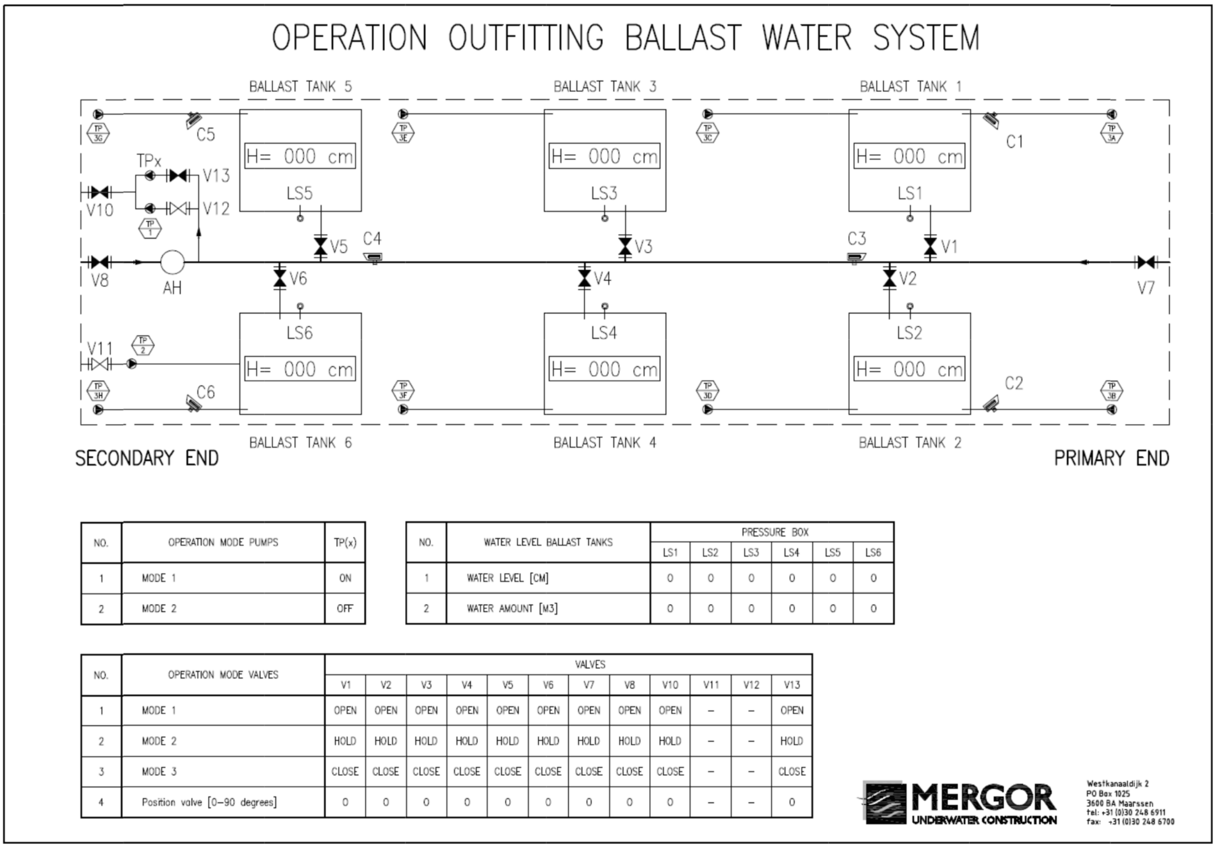

Fig. 27. Operation Outfitting Ballast Water System.

0.5 m from the previous tunnel element. The tunnel element was lifted by the EPS and pulled towards the previous tunnel element by using two 120 ton push-pull jacks which were installed on the tunnel deck. This procedure ensured that the tunnel element could safely be connected without friction of the gravel bed and with no risk of damaging the previous tunnel element by wave induced motions. All the cylinders of the EPS were controlled from the immersion pontoons.

In case the tunnel element would be installed out of tolerance, the EPS could also be used for realigning. To realign the tunnel element, the lifting cylinders lifted the tunnel element from the gravel bed. By using the positioning cylinders on the base plates, the tunnel element was pushed in a sideways direction. When the lifting cylinders were retracted, the tunnel element was lowered on to the gravel bed again. If necessary, the base plates could be repositioned to make another sideways step.

7.6. Ballast

During the immersion operation of a tunnel element it is important to lower it in a controlled manner. To achieve this, sufficient overweight is needed. The amount of overweight needed is often expressed in percentage of the water displacement of the fully submerged tunnel element and is depended on various factors:

1. Changes in water density and the predictability of these changes.
2. External loads on the immersion spread (tunnel element and immersion pontoon) for waves and current.
3. Mechanical layout of the immersion system (statically (un)determined, ridged or floating supports).
4. Lowering speed (hydraulic shape of the tunnel element and adherend water at an emergency stop situation for example).
5. Accuracy of the winch control and load measurement (fiction losses in sheaves and blocks etc.)

In normal, inshore and sheltered conditions an overweight of 0,5–1% is often sufficient. In this specific situation with wave loads from swell waves working on the tunnel element area of 180 × 26 m and the immersion pontoons working in wind waves up to Hs = 1,5 m the model tests showed that an overweight of 1,7–2% was needed to prevent accidental slackening of one of the lifting cables. With a total weight of approx. 48.000 tons per tunnel element this meant a nominal load per lifting lug (4) of 250 tons. Due to the waves, the variation in salinity and dynamic factors the immersion pontoons where equipped with an immersion system that could take 500 tons per lifting lug as SWL.

The ballast system (Fig. 27) comprises of 6 ballast tanks per tunnel element (the centre ones were needed to reduce the bending moment when ballasting to reduce the freeboard to zero) and a main feeder pipeline with branches to the tanks and with inlets at both bulkheads together with a main pump and various leak water pumps. The ballast water system was completely remote controlled, fitted with level sensors and optical verification by CCTV

Ballasting in 6 steps:

After floatation, trimming the element level (roll and pitch) and to an average freeboard of 30 cm for transport to the mooring location.
After the placing of the immersion pontoons and transport to the immersion location the ballasting to freeboard zero, all 6 tanks.
Ballasting to overweight of 1,7–2% in ballast tanks located directly under the lifting lugs.
Ballasting to keep the minimum load in the lifting lugs during immersion.
After landing on the gravel bed, ballasting to 2,6% when in EPS to avoid movement by swell waves.

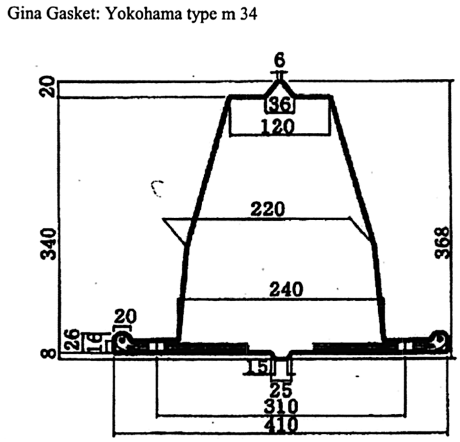

Fig. 28. Gina gasket.

Ballasting to 6% when tunnel element is in final approved position.

On this deep tunnel, most elements had no access shaft therefore no direct connection to the atmospheric pressure. While filing the ballast tanks the air pressure inside the tunnel element rose with the volume ratio of total amount of air inside and the volume of ballast water taken in. Although this is only a few millibar, it was necessary to equalize the pressure inside the tunnel before the bulkhead door could be opened.

After removal of the bulkheads, ballast concrete was placed in between the ballast tanks in exchange of the ballast water which could then be pumped out and the ballast tanks were removed. The final overweight of the tunnel, without backfill, was 10%, i.e. a factor of safety of 1.1.

7.7. Immersion joint

The immersion joint was a standard layout with steel end frames with post-welded in plates and a Gina and Omega solution. Because of the specific circumstances the Gina joint design was somewhat special. The combination of longitudinal seismic movement and temperature led to a decompression of up to 50 mm per joint, combined with high water pressure of 50 m resulted in a Gina of 340 mm height (Fig. 28).

The secondary steel end-frame was fitted with stoppers to reduce the lateral movement of the compressed Gina. Part of the space between the Gina and the Omega was fitted with an air-filled shock absorber (Fig. 29), thus if the space between the GINA and Omega was water filled and in a seismic event when significant movement occurred this shock absorber would compress and prevent rupture of the Omega joint.

During the emptying of the immersion chamber the high and relative narrow Gina had the tendency to deflect inwards at the deeper tunnel elements. At the inspection of the Gina joint, just after the bulkhead door was opened, one could see that at some point the nose of the Gina was pointing inward. Changes in the process of joining the tunnel elements prevented this phenomena later on. These changes involved among other things the monitoring of the pressure and pressure drop in the immersion chamber during the dewatering.

During the warping out operation at the dry dock, one of the Gina's got accidentally damaged by an object at a depth of 10 m. With the use of a purpose build habitat (Fig. 30) it was possible to repair the damaged Gina in dry.

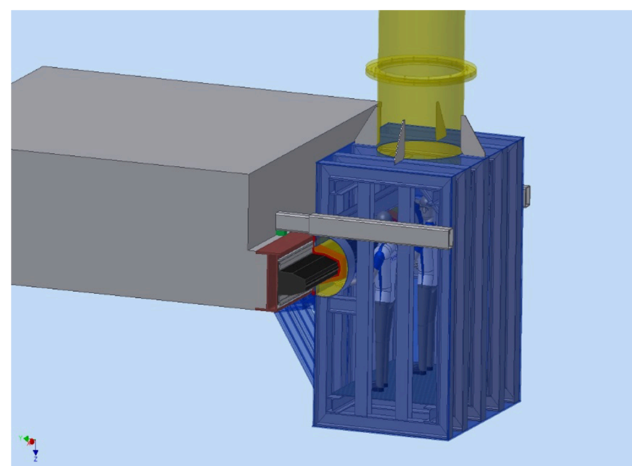

Fig. 30. Habitat for Gina repair.

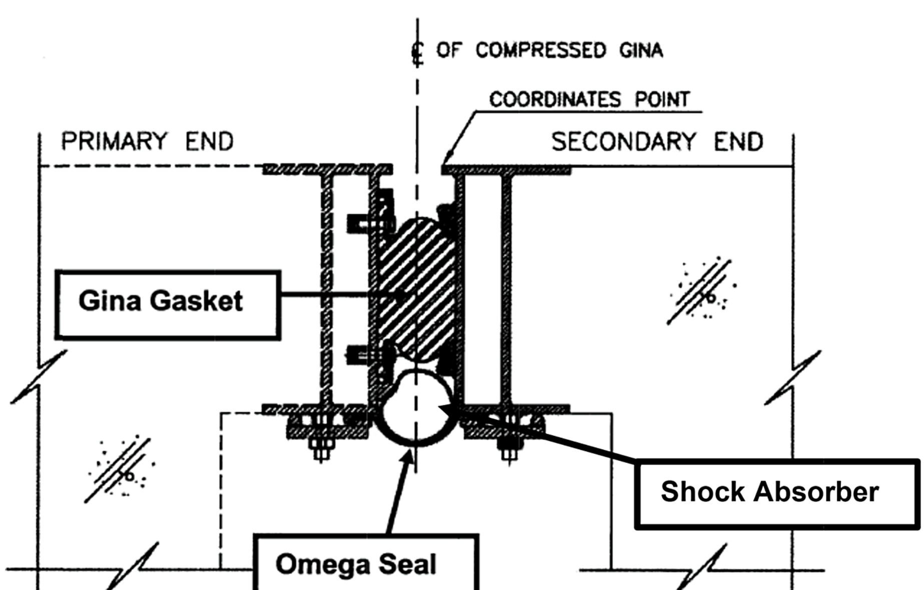

Fig. 29. Crossection immersion Joint.

8. Conclusions

The Busan Geoje tunnel started in concept largely as a copy of the Øresund tunnel, however as described in this paper the project turned out to be very different from Øresund in many respects and was a challenging journey for all the participants. It required significant advances in immersed tunnel technology in a number of areas, these included:

The Design, where a number of innovations had to be developed particularly regarding the longitudinal behaviour of the tunnel and coping with significantly larger joint openings for both the immersion and segment joints than the Øresund project. This challenged particularly the design of the Gina gasket for the immersion joints and also the joint system for the segment joints.

The foundation design, which was one of the major challenges which required ground improvement by either Cement Deep Mixing (CDM) or Sand Compaction piles (SCP) for essentially the whole tunnel alignment. A floating foundation was used with partial depth columns which consequently eliminated many of the problems with the scraded gravel bed such as high spots and twist. This was not part of the Basic Design and entailed major design changes before the commencement of Detailed Design which caused approximately an one year delay to the design.

Whilst a considerable effort was made during the Basic Design to prepare a preliminary immersion design in order to make the correct provisions in the design for the immersion loads. Without the immersion specialist who only came on board late in the Detailed Design, much of this work had to be redone and further numerical and physical model testing carried out for the immersion operations design. Fortunately this was just possible within the time available and fortunately the tunnel structural design had included essentially the largest sensible loading that could be accommodated.

The weather/wave window prediction systems was a development of what was in use for the offshore industry, however this was a large advance in its application to immersed tunnel construction.

The immersion operations, the deep water depth required significant advances in survey and positioning techniques and in the remote control of the ballasting systems as survey towers could not be used.

The development of the Element Positioning Systems (EPS) was probably the largest advance in immersed tunnel technology that was developed for the project, this used together with the scraded gravel bed and the soft "floating" foundation was essentially the key to the success of the Busan project The three systems, the EP, the scraded gravel bed and the soft foundation provided a way of installing the elements to high tolerances in the very challenging conditions where the normal tolerances and techniques could not provide the desired accuracy. These were three separate developments which ultimately came together during construction to provide the answer to many difficult issues.

The project was a considerable success and all involved can take pride in being involved, particular pride however should be should be felt by the contractor Daewoo Engineering and Construction Co. who persevered during the difficult times on the project and without their perseverance it would not have been the success it is.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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