ir. G. Meinhardt & ir. R.M.W.G. Heijmans ARCADIS Infrastructure

The off-shore immersed tunnel in the Busan-Geoje Fixed Link project in South Korea

Abstract

At the present moment, the large infrastructure project Busan-Geoje Fixed Link is being implemented in the south of Korea. Apart from two cable-stayed bridges, an off-shore immersed tube tunnel will be constructed. During the design of the immersed tunnel it was found that the loads on the tunnel due to 9 metre high ocean waves, in combination with its foundation on a layer of weak marine clay, are determinative factors for the design. Furthermore, analyses were carried out of the integrity of the tunnel during an earthquake.

The project

The Busan-Geoje Fixed Link, with a length of 8.2 kilometres, forms a link between Busan Newport (the port expansion project of Busan) and Geoje Island *(see figure 1)*. The link is necessary to improve the accessibility of the archipelago in the south of Korea. The islands of the archipelago accommodate many shipyards and industries; the population, however, lives in the area of influence of Busan.

The main parts of the link are an immersed tunnel of 3,240 metres long, a 300 metre long rock tunnel, an artificial island and two cable-stayed bridges, one having a main span of 475 metres and one having two spans of 230 metres each. The entire link has two lanes in each direction for car traffic.

The immersed tunnel will be constructed with 18 concrete elements of 180 metres long and 26 metres wide. About two thirds of the immersed tunnel (16 elements) lies in an approximately 15 metre deep trench below the seabed *(see figure 2)*. The maximum depth of the seabed is about 40 metres below still water level. At the end on Daejuk island the tunnel lies above the seabed and the tunnel elements will be located in a dam there. Thus the tunnel features a deep and a shallow part.

The project is being developed by the 'Special Purpose Company' GK Fixed Link. Participants in this company are several banks and the contractor, Daewoo Engineering & Construction, along with seven local contractors. The design bureau is the combination Cowi Daewoo Engineering. Commissioning authority is Busan Metropolitan City together with the Province of Gyeongnam, with which a 40-year concession contract has

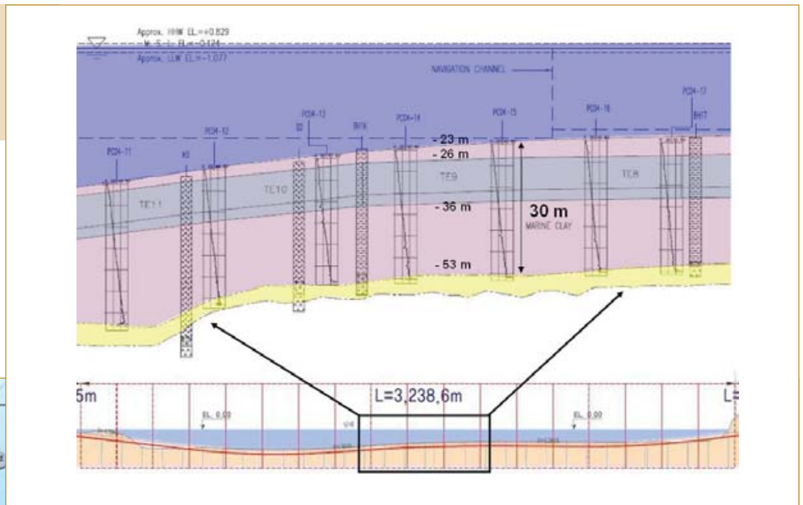

Figure 2 Position of the tunnel in the sea bottom and bottom profile.

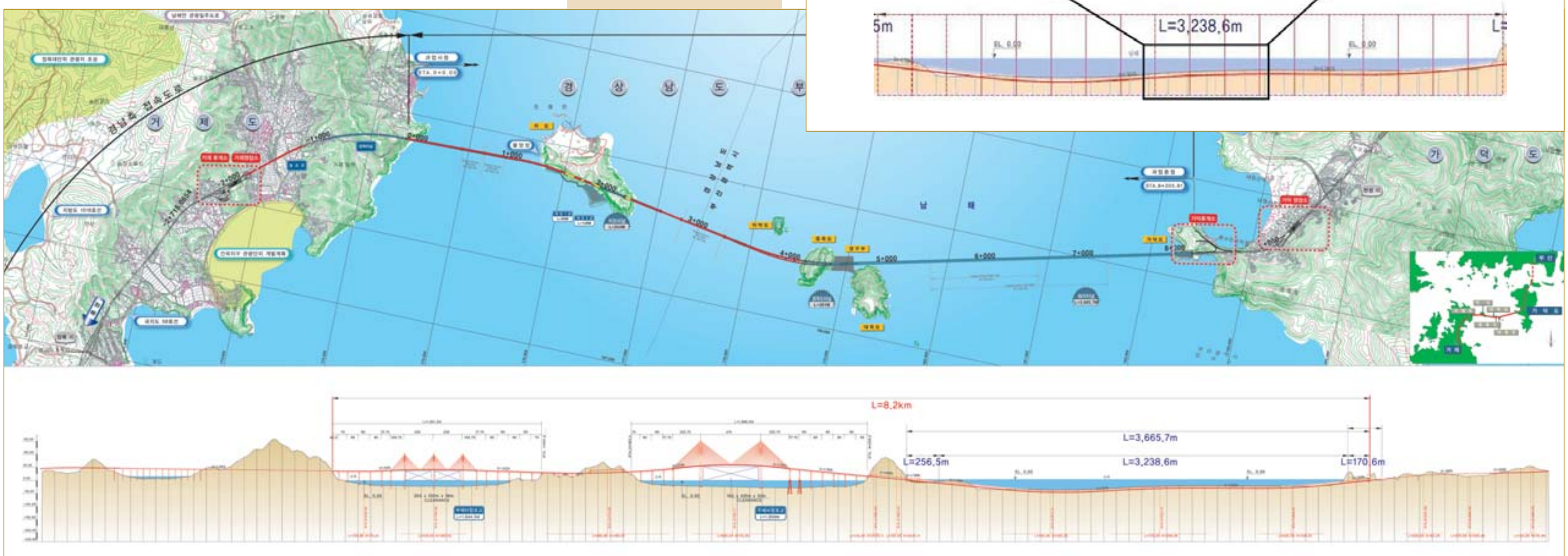

Figure 1 Overview of the Busan-Geoje Fixed Link.

been concluded for the construction and operation of the link.

Independent Design Checker

ARCADIS, together with the French company Ingérop and Korean SeoYeong, acts as Independent Design Checker (IDC) in the project, charged with making an independent analysis of the design and supplying so-called 'design certificates'. The IDC should prevent design errors and guarantee the required technical quality to the commissioning authority.
The IDC has an operational relationship with the designing parties and a contractual relationship with the permitting authorities or the concession holder.

For the Busan Geoje Fixed Link project, all designs of the immersed tunnel and the two bridges, but also the designs of the construction dock *(see photo on the left)*, the temporary harbour and the auxiliary constructions, have been checked independently by the IDC. For this purpose only the design drawings and the raw data – such as geotechnical and hydraulic data, functional requirements and design criteria – were made available to the design checker by the commissioning authority. On the basis of the data and the drawings, independent assessments and check calculations were made of all relevant components of the design.

Wave loads and their effects on the tunnel
The immersed tunnel in Busan will be built up from 18 concrete elements, which are provided with temporary steel bulkheads at the ends. The elements are prefabricated in a dry construction dock, towed to the desired location by tugs during a suitable period of the year, and immersed into an approx. 15 m deep trench in the seabed.
The tunnel is located in an area that is prone to typhoons and steep waves due to the proximity of the coast. The significant wave height at the tunnel location may be as high as 9.2 m. The largest significant wave height documented so far for an off-shore immersed tunnel is 5.3 m in the Naha Immersed Tunnel project [1].

The following two critical scenarios have been identified for the stability of the immersed tunnel in Busan:

- Flotation of the tunnel: Flotation of the construction is checked in this project for the lower limit of the tunnel weight including ballast concrete, with a safety factor of 1.060 to 1.075. In general, water level variations are not significant for the check on flotation of an immersed tunnel. Model tests and calculations have proven, however, that the tunnel is sensitive to flotation due to differences in permeability of the backfill material, for instance, due to silting up of the backfill on one side of the tunnel and water level differences across the tunnel. During the passage of a wave trough, insufficient weight may be available to guarantee a minimum pressure on the foundation plane.

- Horizontal loads on the tunnel tube: a high water pressure gradient across the tunnel due to steep waves, combined with the possible difference in permeability between the inflow and outflow areas, will result in a horizontal force on the tunnel *(see figure 3)*. The horizontal force on the tunnel proved to be linked to a corresponding upward vertical force.

Heavy backfill material beside and on top of the tunnel is required to guarantee sufficient stability of the tunnel *(see figure 4)*. The relevant analyses will be explained below.

The hydraulic boundary conditions were calculated with the numerical model MIKE 21. Using this program the significant wave height for the normative cross-section was calculated at 9.2 m with a frequency of exceedance of once in 10,000 years. The maximum wave height is limited by the water depth and was calculated at approx. 15 m. The shape of the waves at the water surface depends on the water depth in relation to the wave height. In deep water, the shape of the wave can be described by a simple sine function. Furthermore, the pressure on the sea bottom can also be determined with a simple function. Relatively shallow water makes the wave steeper to the point where it wants to break. Description of the wave shape on the basis of a sine function is no longer possible.
In that case a description based on the 'stream function theory' can be used [2]. At the seabed, the waves result in pressure variations and an orbital movement. This is an elliptical or circular movement of the water particles during the passage of a wave. For the specific project situation, the effect of orbital movement on the pressure variations was negligible. The pressure variations of the waves on the sea bottom result in a groundwater flow and different water pressures around the tunnel and in the backfill material.

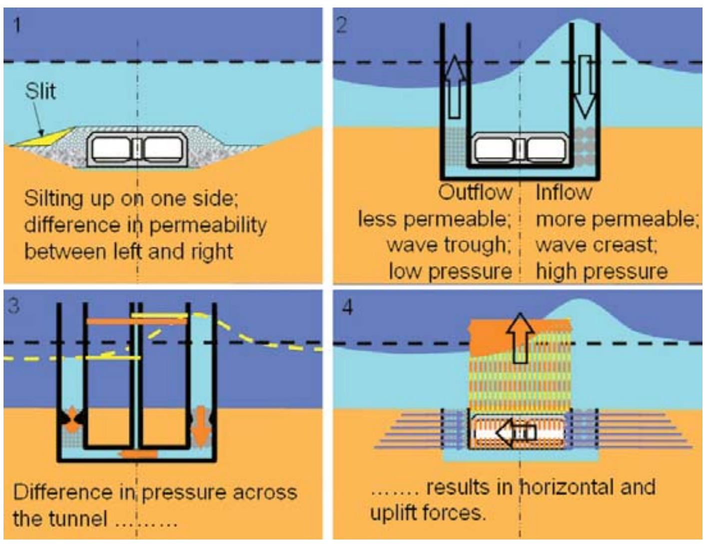

Figure 3 *Effect of the difference in permeability on the water pressure distribution around the tunnel.*

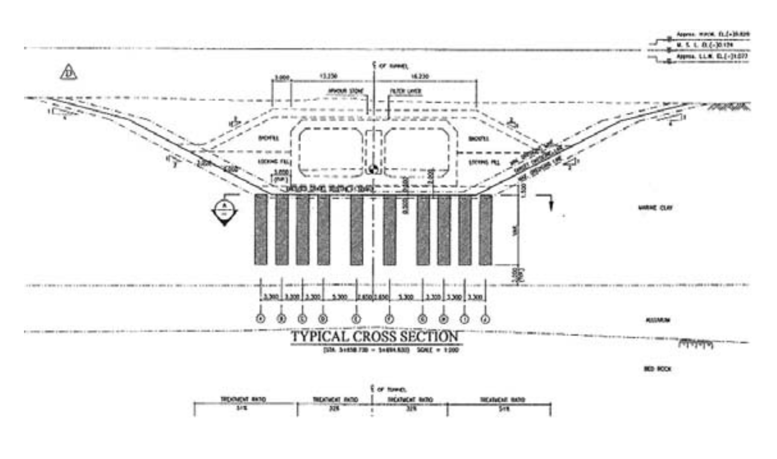

Figure 4 *Cross-section of immersed tunnel, deep position.*

To determine the effect on the tunnel of the pressure variations on the seabed and around the tunnel, extensive PlaxFlow and Plaxis calculations have been carried out. The advantage of combining Plaxis with PlaxFlow is that it allows the correct geotechnical and geohydraulic boundary conditions to be implemented in a model. Firstly, PlaxFlow was used to determine the water pressures in the backfill material and the bottom, and the pressures around the tunnel. The PlaxFlow calculations were based on the pressure distribution of the waves on the seabed *(see figure 5)*. In the flow calculation a steady state groundwater flow was assumed. This is a realistic scenario, because the pressure waves run at a much higher velocity than the waves themselves. Inertia effects, in particular of the tunnel, have been ignored, because of the fact that the waves are present above the tunnel long enough to accelerate the entire mass system of soil, tunnel and water.

The pressures around the tunnel were calculated with PlaxFlow while varying the location of the waves above the tunnel. The effects of different permeabilities on the calculated pressure distribution, as well as the resulting loads on the tunnel, were also studied in this way.
The permeability was varied within the possible limits. The result for the 10,000 year wave and for a difference in permeability of a factor 10 (permeability of outflow area is 10 times lower than that of the inflow area) is presented in figure 6. The calculated maximum vertical force upward per running metre is approx. 380 kN. The maximum horizontal force is approx. 450 kN. This makes it clear that the horizontal and vertical reaction forces are not in phase. The largest upward force occurs when a wave trough moves over the tunnel. The largest horizontal force occurs when the largest pressure difference across the tunnel is present. On the basis of this, the worst case situation was calculated, in which the tunnel is subjected to the largest horizontal deformation. Furthermore, the minimum cover was determined to safeguard a minimum support pressure of 5 kPa.

To determine the deformations of the tunnel and to assess the stability of the tunnel as whole, numerical calculations were carried out using Plaxis. The water pressure situation was calculated first with PlaxFlow and implemented in Plaxis as a boundary condition. The bottom soil parameters for Plaxis were determined on the basis of oedemeter tests, which were available in sufficient numbers to perform a statistical analysis for this project. Triaxial tests were available to a lesser extent and were used for verification purposes. In view of the relief and reload situation, the Hardening Soil model was used for the marine clay. The backfill material, the sand and the weathered rock were modelled with Mohr-Coulomb. In view of the short duration of the wave load on the tunnel, the clay was assumed to be undrained and to have increased dynamic stiffness. The calculated maximum horizontal deformation due to wave passage was in the order of magnitude of 40 mm. Finally, insight into possible failure mechanisms was obtained on the basis of the Plaxis model *(see figure 7)*.

Foundation

The foundation of the tunnel is formed by a thick layer of marine clay down to about 30 metres below the seabed *(see figure 2)*, followed by a layer of sand and gravel, underlain by weathered rock.
The properties of the marine clay require special attention. Normally, clay deposits exhibit some degree of overconsolidation due to ageing.
On the basis of the available research data, such an effect could not be found for this marine clay. A remarkable feature was that the clay exhibited fairly stiff behaviour at low stress levels, whereas the deformations increased sharply at higher stresses *(see table 1)*. Furthermore, the specific gravity was low in relation to the stiffness upon reloading.

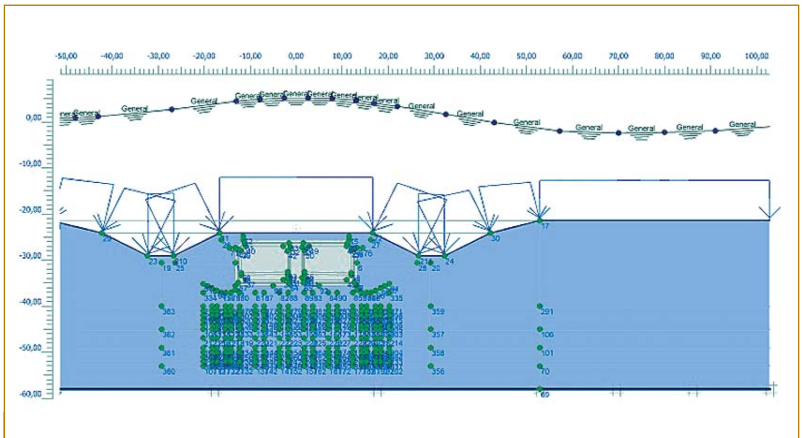

Figure 5 *Distribution of water pressure on sea bottom in PlaxFlow.*

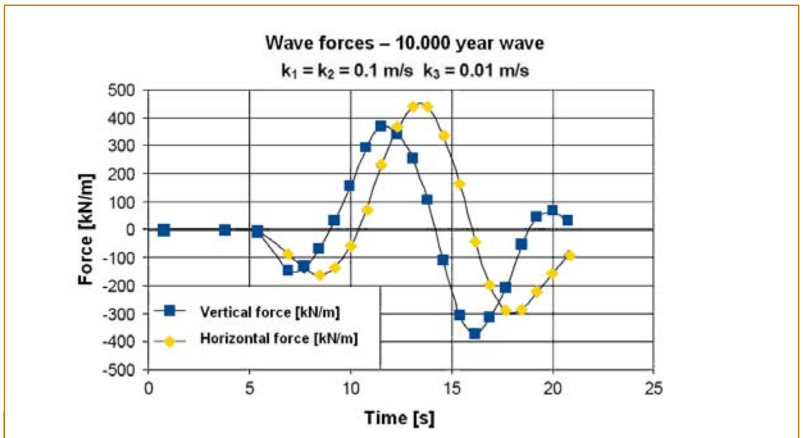

Figure 6 *Resulting forces on the tunnel due to water pressures.*

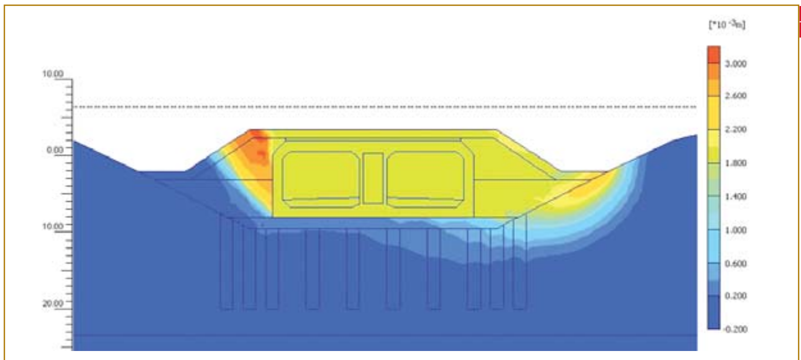

Figure 7 *Possible failure mechanisms around the tunnel.*

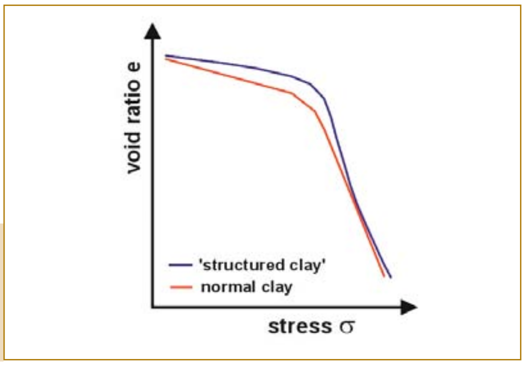

Figure 8 *Stress and deformation behaviour of clay and structured cla.*

Soil type		Weight γ_{nat} kN/m³	e_o	Compressibilty parameters C_c	C_{rc}	C_α	Strenght parameters Ø' °	c' kPa	c_u kPa
Marine clay 'stuctured clay'	Average	14,7	2,44	1,25	0,091	0,044	25	3	Variable*
	Band width	n.d.	1,99-3,24	0,83-1,82	0,041-0,133	0,029-0,064	n.d.	n.d.	n.d.

n.d. = not determined ** depends on OCR and vertical effective stress, indication: C_u = 20 à 50 kPa*

Table 1 *Soil parameters of marine clay.*

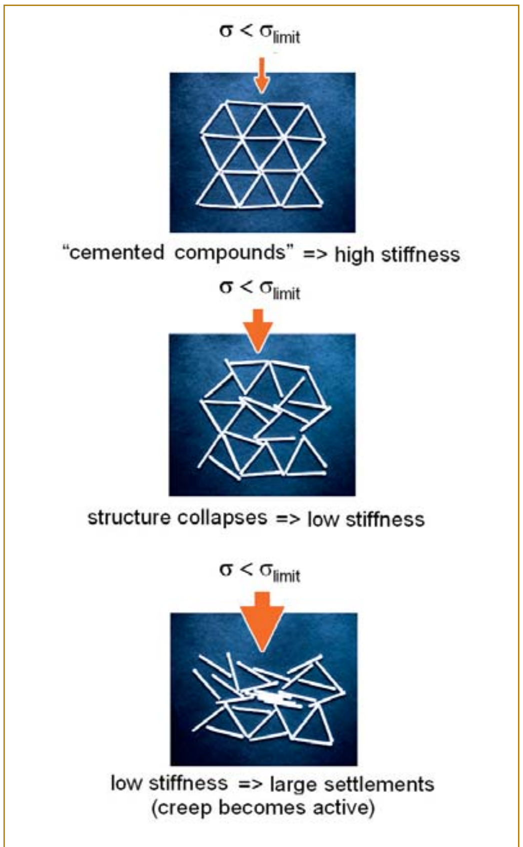

Figure 9 *Failure mechanism of structured clay.*

Eventually the clay was classified as 'structured clay'. This means that the clay contains calcium compounds, which cause a relatively high stiffness at relatively low stresses [3], [4]. In *figure 8* this is shown schematically for 'normal' clay and for 'structured' clay. If the calcium compounds are broken up by an increase of the stress level beyond the so-called 'limit stress', then they collapse like a house of cards and the clay - behaves like a very weak material *(see figure 9)*. Furthermore, creep effects may be reactivated.

Usually the weight of the tunnel under water and the backfill beside the tunnel is lower than the weight of the excavated soil. Then, by definition, the stress level under the tunnel cannot become higher than the stress originally present.
With this tunnel, however, due to the off-shore conditions, as explained above, a heavy backfill on and beside the tunnel is necessary to offer sufficient resistance to wave pressures. Consequently, the soil stress below the tunnel increases beyond the limit stress, thus significantly increasing the settlements of the tunnel. This can be expected to be associated with large settlement differences. These in turn may lead to opening of joints, possibly beyond the capacity of the waterstops.

Soil improvement with SCP process at the western end of the tunnel.

To remove the uncertainties described above with regard to the settlements, different soil improvement techniques have been used. At the ends of the tunnel, the clay layer under the tunnel is relatively thin. Here the clay is replaced with rock fill. Along the major part of the tunnel route the tunnel is founded on a grid of columns formed in the soil, using the Cement Deep Mixing (CDM) process.

This process involves a pontoon from which a vane on a drilling shaft is turned into the soil, which mixes the clay with cement. The columns each have a diameter of 90 centimetres, and are placed as continuous walls under the tunnel *(see figure 5)*. The stiffness and strength of the clay are sufficiently increased by the CDM columns to avoid undesired settlements.

At the western end, the tunnel will be resting in a dam construction built on the sea floor. The CDM method is considered less suitable here, as it may cause disks to be created, which would form a potential slip surface during an earthquake. The solution chosen here is the use of sand columns according to the Sand Compaction Pile (SCP) process. These sand columns, with diameters of 1.2 to 1.8 metres, are inserted to a record depth of approx. 60 metres *(see photo this page)*. The dam is built onto the column grid with some excess height, so that after placement of the tunnel elements the remaining settlement will be very small.

For the design of the foundation with soil improvement, use has also been made of a large number of Plaxis models for both the deep position of the tunnel in the trench and the high position in the dam. All possible load situations were taken into account, such as a sunken ship on the tunnel. Subsequently the length and location of the columns were optimised.

In the context of the CDM and SCP methods it should be noted that, apart from the design, the independent design checker has also assessed implementation aspects. Technical specifications have been prepared for these two techniques. For the CDM method, for instance, the correct strength is an important aspect. On the one hand the columns must be sufficiently strong and stiff. On the other hand, the columns must not be too strong in order to avoid damage to the columns during cutting off and excavation to the correct depth.

Earthquake analysis of the dam

At the western end, the last two elements of the tunnel run through a dam in an elevated position. In the preliminary design, the dam was provided with 1:1.5 slopes without shoulders. However, the preliminary design was based on the scale model tests carried out for dimensioning of the tetrapods of the dam. No account was taken of the total stability of the overall soil massif below the dam. Moreover, earthquakes and the steepness of the waves above the slopes may have a negative effect on slope stability, which has not been included in the model tests either. An earthquake will lead to an increase in the gradient of the wave above the slope of the dam *(see figure 10)*. Furthermore, excess pore pressure will be generated in the soil below the tunnel during an earthquake.

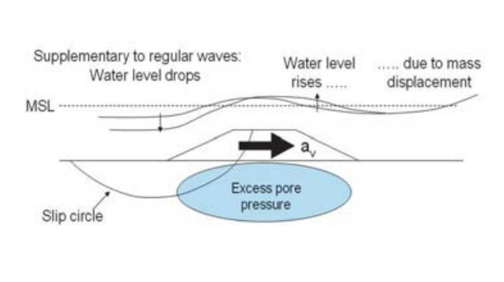

Figure 10 *Schematic representation of hydrodynamic effects on the slip plane of a dam during an earthquake.*

Earthquake	*Year*	*M*	*Dam*	*Acceleration* k_h	*Safety factor* *SF*	*Effect of earthquake*
Santa Barbara	1925	6.3	Sheffield dam	0,10*g	1,2	Complete failure
Santa Fernando	1971	6.6	Santa Fernando dam *downstream*	0,15*g	1,3	Waterside slope collapsed
			upstream	0,15*g	2-2,5	Caving in on land side

Table 2 *Observed failure modes of earth dams during earthquakes (Source: Seed 1979).*

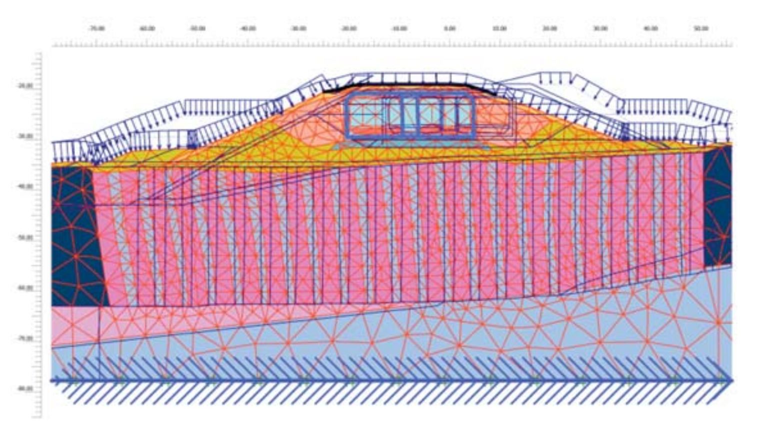

Figure 11 *The Plaxis model for earthquake analysis.*

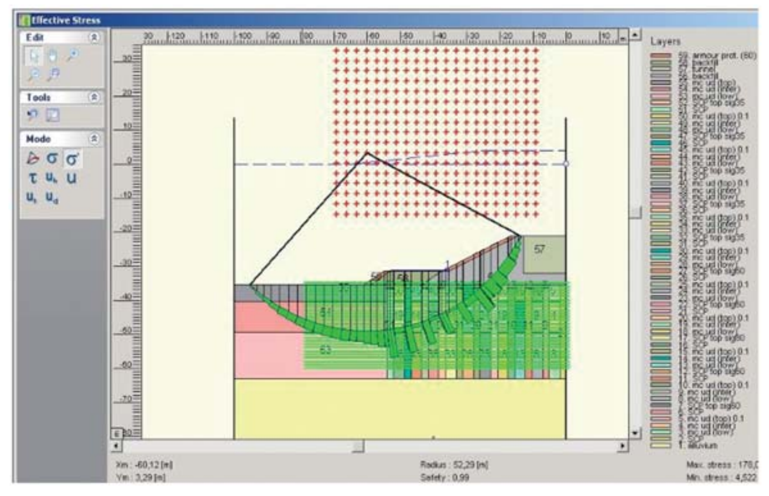

Figure 12 *Schematisation with MStab.*

The fact that these effects can lead to instability of earth dams is demonstrated by Table 2, which has been derived from article **[5]**. In 1925, the 'Sheffield dam', with a calculated safety factor SF = 1.2, collapsed completely due to an earthquake of M = 6.3 on the Richter scale with an effective horizontal earthquake acceleration of 0.1*g (acceleration of gravity). An analysis of the stability of the dam with the embedded tunnel was necessary.

The earthquake analyses for the immersed tunnel were made using accelerograms suitable for the seismic zone. However, these provide data on the accelerations of the deep rock. To obtain representative acceleration levels of the earth dams, dynamic Plaxis calculations were performed. The NERA software **[6]** was used to determine the output signal on the rock, the so-called 'rock outcrop signal'. This has been entered into Plaxis as a boundary condition *(see figure 11)*. Possible accelerations of the dam with the tunnel were calculated on the basis of the methodology described in the Plaxis manual **[7]**. At the location of the tunnel, earthquakes may occur that generate accelerations of up to approx. 0.15g. These earthquakes are in the order of 6 to 6.5 on the Richter scale. The current version of Plaxis does not accurately calculate the generation of excess pore pressures due to an earthquake. Therefore it was decided to perform the stability calculations using a slip circle model. The MStab program of DelftGeoSystems **[8]** was selected for this purpose. To ensure correct modelling of the stress situation with MStab, a number of schematisation steps were necessary. For instance, due to arching effects increased stresses occur on the columns as a result of the load from the dam *(see figure 12)*. Here the stress distribution on the columns was based on the results of the static Plaxis calculation. On the basis of the MStab model, the sensitivity of possible excess pore pressures in the columns during an earthquake and the influence of the strength parameter (internal friction angle) of the columns could be analysed with a larger number of calculations than with more elaborate numerical calculations.

At first, the classic approach to stability calculations was chosen, using a pseudo-static method as described in the MStab manual, in which a horizontal earthquake acceleration is added to the vertical acceleration due to gravity **[8]**. The results showed, however, that even with relatively gentle slope gradients (more than 1:2) and relatively large shoulders of more than 20 m wide, the level of safety was insufficient (SF < 1.0). This led to an approach in which plastic deformations in the dam beside the tunnel are regarded as acceptable. In literature this approach is known as the Newmark method **[9]**. On the basis of this method, Makdisi & Seed presented a graph in 1978 *(see figure 13)*, which allows estimation of the possible plastic deformations of a dam on the basis of the ratio between the maximum acceptable horizontal acceleration to prevent failure (yield acceleration, k_y) and the maximum possible local acceleration (k_{max}) **[9]**.

The yield acceleration (k_y) is calculated with the slip circle model. The expected maximum acceleration (k_{max}) is determined with Plaxis and on the basis of NERA, as described above. On the basis of the basic assumptions with respect to the soil parameters, the hydraulic boundary conditions, a slope of 1:2 and a shoulder of 20 m wide, k_y/k_{max} ratios between 0.2 and 0.3 were calculated for the present project. According to the graph of Makdisi & Seed this will result in plastic deformations of 10 to 20 cm, which is considered acceptable for the soil massif beside the tunnel.

Summary

During the final design phase of the immersed tunnel in the Busan-Geoje Fixed Link, the following three issues were found to be critical from the hydraulic and geotechnical point of view due to the off-shore location of the tunnel. These issues have led to the following adjustments to the reference design:

- Vertical and horizontal pressure differences across the tunnel due to waves have resulted in the necessity of heavy backfill material on and beside the tunnel.

- The heavy backfill material, in combination with the mechanical properties of the marine clay, a so-called 'structured clay', makes soil improvement for the foundation of the immersed tunnel necessary to limit deformations and guarantee stability. On the basis of earthquake and implementation considerations, it has been decided to use sand columns as soil improvement in areas where the tunnel is embedded in a dam. In stretches where the tunnel runs below the sea bed, use has been made of Cement Deep Mixing columns.

- Earthquake analyses of the dam have resulted in the use of a more gentle slope gradient and a shoulder. Plastic deformations of the dam in accordance with the theory of Makdisi & Seed are considered acceptable during an earthquake.

Conclusion

The stability of an immersed tunnel under off-shore conditions calls for a multidisciplinary approach between hydraulic, geotechnical and structural specialists in a design team. This project is a good example of how the Dutch expertise in the fields of immersed tunnels and off-shore constructions can make a vital contribution to an international project. The following Dutch parties are involved in implementation of the project: the tunnel trench was dredged last year by Van Oord, TEC is the engineering consultant of the concessionaire, Trelleborg-Bakker is supplier of the sealing strips, and MARIN in Wageningen carried out a series of scale model tests of the immersion process. At the moment Strukton is busy to immerse the tunnel elements into their final position. During the last audit of the IDC in Busan, implementation was in full swing, and completion of the Busan-Geoje Fixed Link is expected in 2010. ■

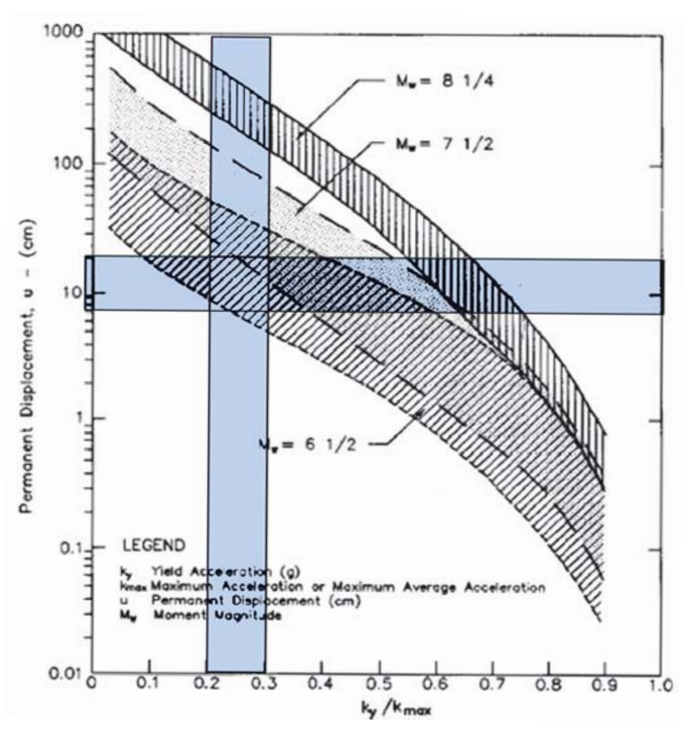

Figure 13 Estimation of plastic deformation of soil, Makdisi & Seed (1978).

Literature

[1] Aono T., Sumida K., Fujiwara R., Ukai A., Yamamura K. and Nakaya Y.; 2003; *Rapid stabilization of the immersed tunnel element*; Proceedings of the Coastal Structures 2003 Conference Portland, Oregon, August 26-30, 2003, 394-404; American Society of Civil Engineers.

[2] Heijmans R., Jackson P., Kasper T., Meinhardt G., Schmitt J., Voortman H.; 2007; *The effect of wave passage on immersed tube tunnels - Busan Geoje Fixed Link in South Korea*; IABSE 2007 Weimar.

[3] D. Masín, S. E. Stallebrass and J. H. Atkinson; 2003; *Laboratory modelling of natural structured clays*; Int. Workshop on Geotechnics of Soft Soils - Theory and Practice.

[4] J.-C. Chai, N. Miura, H.-H. Zhu, and Yudhbir; 2004; *Compression and consolidation characteristics of structured natural clay*; Can. Geotech. J. 41: 1250-1258.

[5] G. Biondi & M. Maugeri; year unknown; *A modified Newmark type-analysis according to EC-8 requirements for seismic stability analysis of natural slopes*; University of Catania, Italy.

[6] University of Southern California, Department of Civil Engineering, NERA, *A computer program for nonlinear earthquake site response analyses of layered soil deposits by J. P. BARDET and T. TOBITA*, April 2001.

[7] PLAXIS, 2D - Version 8, Manual

[8] Geo Delft, MStab User Manual, Release 9.8, February 2004.

[9] Steven L. Kramer, *Geotechnical earthquake engineering*, Prentice Hall, 1996, ISBN 0-13-374943-6.