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IZMIT BAY BRIDGE
Geotechnical challenges and innovative solutions

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ABSTRACT: The Izmit Bay Suspension Bridge, app. 50 km east of Istanbul, crosses the Sea of Samara with a main span of 1550 m. The foundation of the bridge poses interesting challenges in that the bridge site is in a highly seismic region. Moreover the ground profile ranges from Dolomitic Limestone to a thick sandwich of silty sand and clay layers overlying the bedrock.

KEY WORDS: Earthquake; Foundations; Pile inclusions; Suspension bridge.

1 INTRODUCTION

The Izmit Bay Bridge will feature the world's 4th longest suspension bridge, main span of 1550 m, by the slated time of inauguration in 2016 (see *Figure 1*). It is part of a major BOT infrastructure project in Turkey, the 420 km long Gebze-Izmir highway, with an approximate construction cost of \$1.2 billion out of the total cost of \$11 billion.

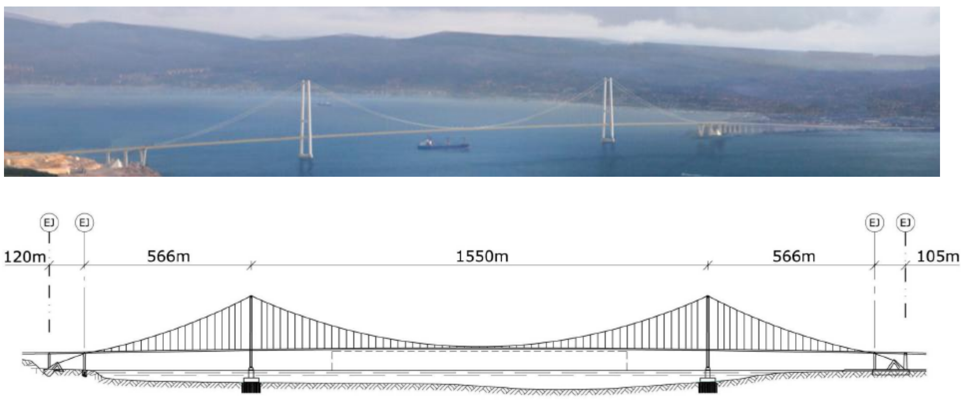

Figure 1. General layout of Izmit Bay Bridge (North to south = left to right)

The Owner is NÖMAYG / Nurol-Özaltin-Makyol-Astaldi-Yüksel-Göçay and the Contractor is IHI Infrastructure Systems CO., Ltd with COWI as bridge designer.

The bridge site is located app. 50 km east of Istanbul and crosses the Sea of Samara. This is a highly seismic region in close proximity to the North Anatolian fault which caused the 1999 earthquake. Moreover, the ground profile ranges from Dolomitic Limestone at ground level on the northern side to kilometre thick sandwich of silty sand and clay layers overlying the bedrock. The top of the soil deposits are very loose/soft and susceptible to liquefaction. The foundation structures are therefore different when moving from North to South.

The North anchor block is a gravity type foundation taking advantage of the Limestone rock but at the same time addressing the effects from inherent bedding planes and joints. The tower foundations are precast concrete caissons. As the subsoil exhibits inferior strength and deformation properties the subsoil is improved by 35 m long Ø2 m driven steel pile inclusions. To allow energy dissipation during seismic events a gravel fuse is separating the concrete bottom of the caisson and the piles. The South anchor block is placed within an artificial island where dense sand at relatively shallow depth allows for a direct foundation. The South anchor block is also verified for a potential secondary fault passing through the anchor site.

The paper focusses on the geotechnical and soil-structure interaction aspects of the design and construction. It describes the design process allowing the foundation challenges to be met by innovative solutions.

The project has been under preparation from the early 1990'ties but after a new tender submission in Sep. 2010 IHI was awarded the contract with COWI as designer. After contract negotiations the detailed design started in September 2011 with preparatory site works in September 2012 and permanent site works in January 2013.

2 GEOLOGICAL SETTING & GROUND INVESTIGATIONS

The Izmit Bay is the eastern continuation of the Sea of Marmara and is primarily shaped by the tectonic movement along the North Anatolian Fault. This is some 1600 km long and extends from Karliova in eastern Turkey to the Aegean. A major right lateral strike slip fault forms the tectonic boundary between the Eurasian plate and the Anatolian block of the African plate. The northern strand of the Anatolian Fault zone occupies the Izmit Bay and projects across the project alignment, presenting the greatest seismogenic hazard source within the area. However, no active faults were recognized in the bridge alignment.

Two investigation campaigns were carried out. The Phase I campaign in 2009/2010 aimed at identifying regional faults, stratigraphy and ground conditions along the proposed bridge alignment and at site specific locations for the bridge foundation. The investigations comprised geophysical surveys offshore and onshore, CPTU tests (100) and geotechnical boreholes (30 vertical and 3 inclined, with depths from 36 to 200 m below ground level) with sampling and in situ testing (ranging from SPT to down-hole CPTU and suspension logging). Samples of soil and rock were tested to provide classification, strength, deformation and hydraulic parameters. The tests ranged from normal classification testing to advanced laboratory testing comprising anisotropically consolidated triaxial compression and extension test, CRS and incremental loading oedometer tests, static direct shear tests, resonant column/torsional shear tests, strain and stress-controlled cyclic direct simple shear tests.

The Hazard Zonation map forming the basis for the tender showed that the South anchor block location was well within a “Non-fault” area. However, during the detailed design the zone with possible secondary (inactive) faults were extended to the South anchor block location. Furthermore, it was concluded that the location for the North anchor block on the “beach area” in front of the rock outcrop (see Figure 2) was an area of relatively homogeneous Paleozoic limestone without major faults, shear zones or adverse geologic structures.

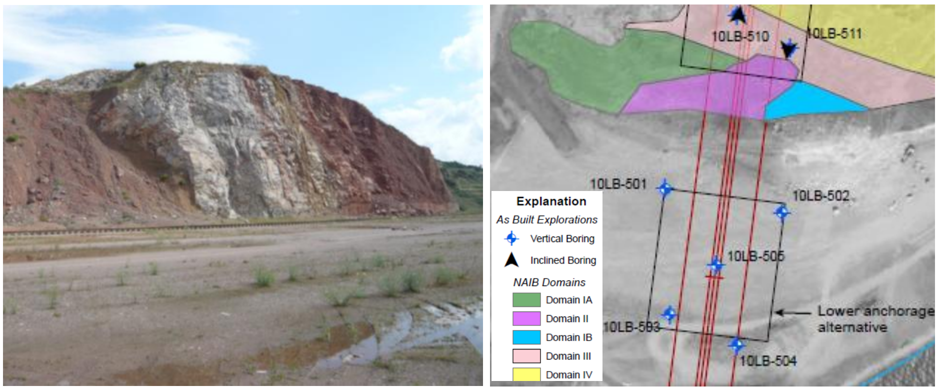

Figure 2. (a) ”Beach” in front of rock outcrop at North anchor block site 2010-06-10; (b) initial interpretation of rock domains (area in grey is the “beach”)

After award supplementary investigations (Phase II) were carried out in 2011/2012. These aimed specifically to identify areas of no-faulting for the South anchor block location area and to gain insight into possible multi-rock-domains rather than one single competent limestone domain at the North anchor block area. This is discussed in more detail in Sec. 4.2.

Furthermore, supplementary geotechnical boreholes and CPTU probing were carried out in the final tower foundation positions as the re-positioning of the South anchor block 160 m to the north necessitated re-location of both of the towers by 80 m to the north.

Based on the ground investigations and with due respect to interpolation between investigation points at foundation locations far apart it was possible to derive idealized geological models in the Geotechnical Interpretative Report prepared by COWI. The design profiles for the different foundation locations are seen in Figure 3 and Figure 4 for anchor blocks and towers, respectively.

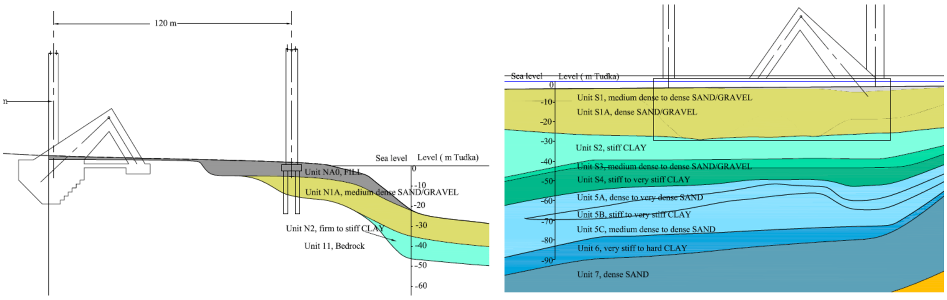

Figure 3. Design profiles; (a) North anchor block; (b) South anchor block

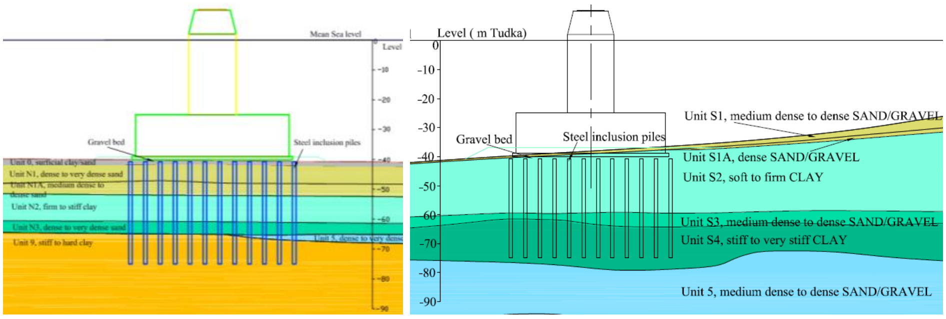

Figure 4. Design profiles; (a) North tower; (b) South tower

Note that the “foundation outline” for the South anchor block corresponds to the original concept with a piled foundation.

It is apparent that the rock (shown non-shaded on Figure 3a) dips rather steeply from the onshore outcrop area behind the North anchor block towards south. Bedrock was met at level -186.7 m at the North tower but was not met in the southern part of the alignment (assumed kilometre depth).

Ten different soil units were distinguished along the alignment although not all of them were found at all locations. In broad terms the soil stratification is a sandwich of sand and clay at the tower locations and the South anchor block.

At the North anchor block location the “beach area” is partly quarried and partly reclaimed with sand/gravel and limestone blocks as seen from vintage Google earth maps in Figure 5 and as indicated in Figure 3.

Figure 5. North anchor block area changes from Google Earth photos; (a) 2004-11-19; (b) 2007-12-31; (c) 2009-04-11; (d) 2013-04-13

3 REQUIREMENTS

The bridge is designed according to Eurocodes with the UK National Annex with the exception that ship impact forces are calculated according to AASHTO. The structural design life requirement is 100 years.

The minimum navigational clearance envelope is 64 m by 1000 m, where the vertical clearance is measured relative to 0.3 m above mean sea level.

For the severe seismic loading three seismic events are considered with rock outcrop PGAs of 0.25, 0.65 and 0.87 g, respectively:

- FEE - Functional Evaluation Earthquake; return period 150 years (equivalent to the 1999 earthquake). Demand: immediate access and no damage
- SEE - Safety evaluation earthquake; return period 1000 years. Demand: Limited access and repairable damage
- NCE - Non collapse earthquake; return period 2475 years. Demand: No collapse and no casualties.

The seismic load response spectra at bedrock level were used for the initial analyses but all foundations were verified by the load output from seven sets of earthquake time histories. As this involved four foundation structures, three return periods and each comprising three components a total of 252 analyses were carried out using COWI’s in-house Integrated Bridge Analysis and Design Software (IBDAS), described in [1]. For information on the IBDAS modelling and the substructure structural design see [2] and [3].

As the bedrock was hundreds of metres below seabed for the major part of the alignment PSHA analyses on competent soils, developing 5% damped target response spectra at depths of 126, 89 and 100 m (with input shear wave velocities of 1000, 490 and 490 m/s), were carried out for North tower, South tower and South anchor block, respectively.

Despite relocation of the South anchor block it was not possible to rule out completely the possible occurrence of a secondary fault within the anchor block foot print (see Figure 6). It was recommended to take this into account by assuming a fault plane at level -100 m with horizontal and vertical slip movements of 0.70 m and 0.25 m for SEE and 1.00 m and 0.50 m for NCE.

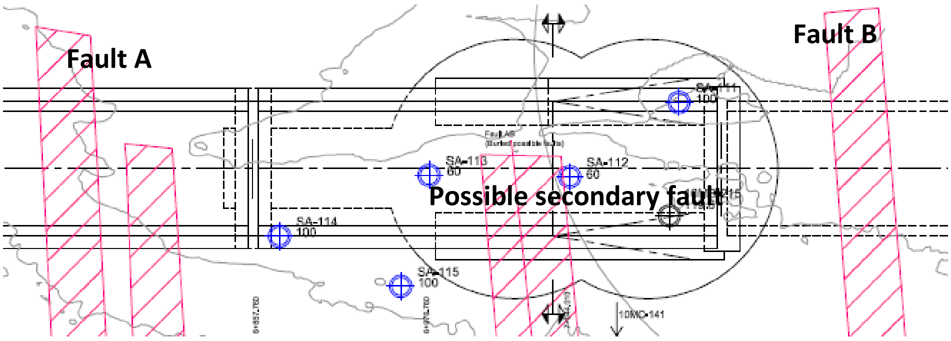

Figure 6. Interpretation of (inactive) faults at the South anchor block location

4 FOUNDATION SOLUTIONS & CHALLENGES

4.1 General

The foundation solutions adopted for the Izmit Bay Bridge are shown schematically in Figure 7 with gravity structures for the anchor blocks and a hybrid gravity/pile inclusion solution for the two towers.

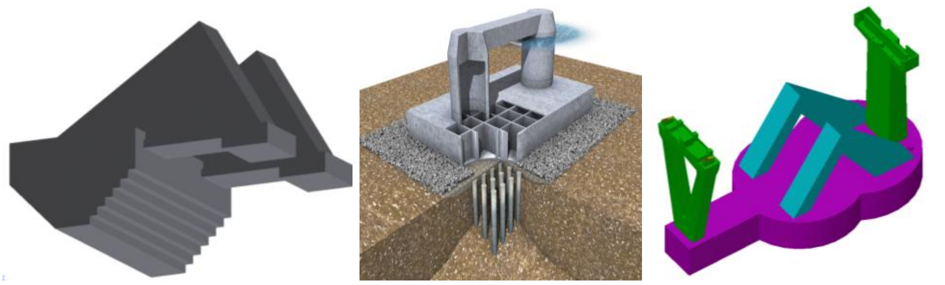

Figure 7. Foundation concepts; (a) North anchor block; (b) Tower foundations; (c) South anchor block

The very high ship impact loads, 246 MN from the 160,000 DWT design ship and particularly the seismic load cases with horizontal forces typically of the order 400 MN, posed very significant challenges for the pylon foundations.
Direct foundation was obviously not an option and traditional piled foundations would be very hard to verify for the seismic NCE event.

Fortunately the ground conditions and the magnitude of loading were similar to the conditions for the Rion-Antirion Bridge in Greece (see f.inst. [5]). The characteristics in both cases are: deep water (40-65 m), deep soil strata, weak alluvial deposits, soil strength increasing with depth and strong seismic activity with large tectonic movement. Despite being a cable stayed bridge the design displacement in between pylons was 2 m for Rion-Antirion where the innovative concept of ground improvement by steel pile inclusions and a gravel bed “fuse” between the piles and the concrete foundation base had been tried out in practice. As that bridge had survived an earthquake and the concept had been successfully modelled by physical modelling (centrifuge testing) and by 2D and 3D FEM calculations it proved possible to transfer this foundation solution to the tower foundations for the Izmit Bay Bridge as shown in Figure 7.

4.2 North anchor block

At the outset it was agreed by the design team that the North anchor block was by far the simplest and most straight forward of the foundation structures. It was considered to use the cavern anchorage concept but due to anticipated problems with a very deep cavern below the ground water table it was decided to use a traditional type gravity structure.

It is constructed as a large concrete gravity massif embedded in dolomitic limestone and two counter reacting front pads. The front pads are rigidly connected to the concrete rear massif by 2m high and 11.7m wide concrete beams. Tension load is transferred to the large concrete massif through the cables running through the splay chamber and the counter reacting compression load is transferred to both front pads via two saddle legs. The entire anchor block works as a monolith and is considered a rigid structure for foundation design and analysis purposes.

The rear concrete massif is 22m deep with basic plan dimensions: 33m in width along bridge alignment by 50m in length perpendicular to bridge alignment. At 10m depth the width of the rear concrete massif is reduced at a 45 degree angle. The south face of the massif is stair stepped by six 2m tall and 2m wide steps. The width at the base of the rear concrete massif is 9m. The main resistance to uplift of the North anchor block is provided by gravity of the large concrete mass and by friction developed along the stair stepped face of the massif.

The front pads are also embedded into dolomitic limestone. With the basic plan dimensions: 12m in width along bridge alignment and 14.96m in length perpendicular to bridge alignment. Each concrete footing is 5m high with minimum 4.5m embedment into Dolomitic Limestone.

Since the North anchor block was assumed to be located in a competent Limestone, referred to as Domain 1, a wedge failure where different joints and bedding link up in front of the anchor block is possible. Such discontinuities that would link up are relatively discontinuous and would require breaking of intact rock bridges between existing discontinuities. The rock mass shear strength along the inferred wedge failure plane(s) should consider the rock mass "Global Strength" derived from the in situ rock mass using the Hoek-Brown failure criterion. Thus, very ample capacity of the anchor block could be easily demonstrated.

However, the North anchor block turned out to produce by far the most comments from the Independent Design Checker and proved to be very challenging in terms of both design and execution.

During the detailed design a specialist review of the ground conditions was carried out. This review revealed a number of uncertainties with possible impact on the design of the permanent structure:

- Clay layers could be present, but have not been identified.
- Marl layers have been described in two of the boreholes within the footprint of the anchorage, but their overall distribution is not well defined. They are likely to be present in other boreholes, but may not have been logged due to reduced effects of weathering or core loss.
- The thickness of possible marl layers is poorly defined.
- Strength of marl layers has not been determined due to the negligible amount recovered in boreholes.
- The fault zone at the southern edge of the anchorage has not been adequately investigated and its nature is currently poorly defined.

In the design a bedding plane (strike as observed on site) was included to provide the most onerous sliding plane in a wedge failure as shown in Figure 8a. It was envisaged that any such plane would be "undulating and discontinuous as confirmed in subsequent investigations

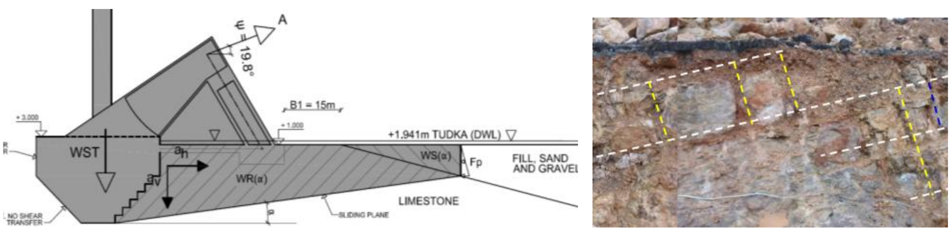

Figure 8. (a) Failure mechanism assumed for North anchor block; (b) Photo of interpreted bedding discontinuity planes in trial excavation wall (white broken lines)

However, to manage the perceived residual risks a programme of additional investigations was initiated. These included one vertical and four inclined boreholes and two major trial trench excavations for identifying the location and nature of the geologic domains and the shear zone(s) boundary, material and strike and to provide samples from the zone (see Figure 9a, b).

Furthermore, samples from the existing cores with and without apparent fissures and beddings were tested in direct shear at a specialized laboratory.

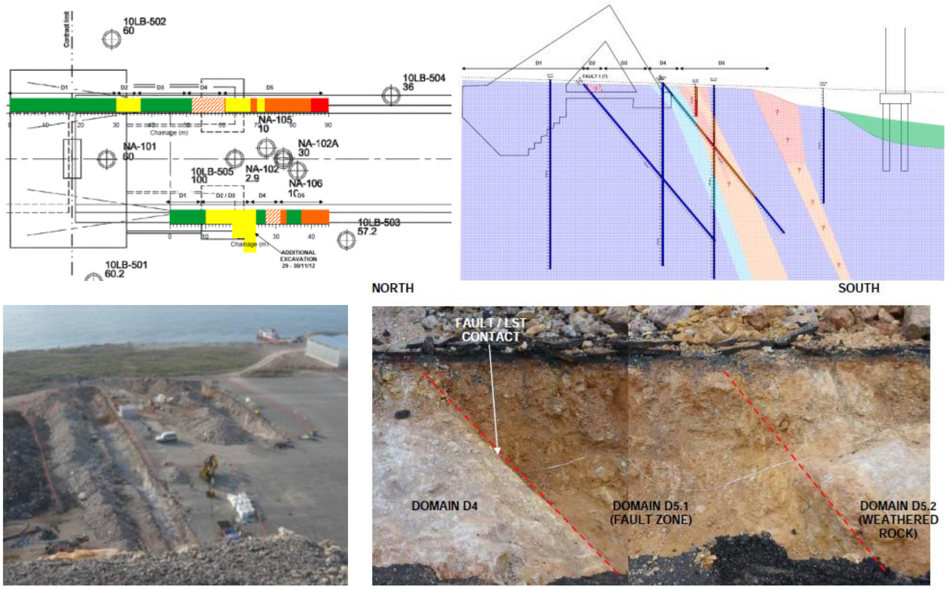

Figure 9. (a) Plan of trail excavations; (b) Cross section showing boreholes and updated geological model; (c) Initial excavation with backhoe; (d) Fault boundary

A site visit where the existing cores and the rock outcrop behind the anchor block were inspected together with the initial trenching brought some comfort as it was concluded that

- There are no clay layers in relation to bedding surfaces
- Marl was not identified in the boreholes
- The presence of a fault zone indicated in one of the boreholes was confirmed but no evidence of larger scale karsts was found
- To err on the safe side a wedge type failure defined by thinly laminated zones at an angle of 15º to the horizontal should be considered
- The design can be progressed with an agreed lower bound angle of friction of 40º for the kinematic failure mechanism

The reporting from the trench excavations and the results from the laboratory testing did (fortunately) confirm that the cautious estimates of strength properties, bedding plane orientation and geologic zonation were justified and the design could be approved.

The resulting geological model is seen on Figure 9b. During the excavation for the anchor block site inspection and mapping confirmed the updated geological model. Minor karstic features were seen during excavation but without impact on stability as all of them were above the global failure mechanism. However, some of them led to water ingress into the excavation possibly assisted by the major blasting operations with dynamite for every 3-5 m depth of excavation of the rear pad (away from the stepped front of the rear pad) necessary to break the sound limestone. Before blasting the excavation was partly filled with water to reduce impact on the surroundings and after drawdown of the water the debris could be broken down by chisel and taken away (ref. Figure 10a, b). Some major inflows were observed before perimeter grouting for the whole anchor block foot print was effectively completed as seen in Figure 10c.

Figure 10. Excavation for North anchor block, (a) Blasting from a safe distance; (b) Debris after blasting; (c) Major water ingress in north wall after blasting

The progress of the excavation is seen in Figure 11. In order to provide safety during construction the walls were secured by rock bolts and shotcrete as deemed necessary and water inflows from fissures were stopped by polyurethane injection or steel plates for the main water inflow locations on the north wall. For the front of the rear pad, the stepped face and the front pads concreting against intact rock was ensured.

Figure 11. North anchor block excavation and casting of front pads and base

The kinematic approach necessitated for the SEE and NCE load cases, with FOS < 1, led to maximum displacements of 50 and 110 mm, respectively.

4.3 North and South towers

The tower foundation is a hybrid solution with steel pile inclusions for soil improvement and a gravel bed acting as a horizontal load fuse between the caisson base and the free-standing piles. This also has the advantage to allow the same foundation level for the prefabricated caissons at level -40 m for both towers. The caissons are 67 m by 54 m rectangular cell structures (to level -25 m) with steel/concrete composite shafts with top at level +10.15 m for easy and fast erection of the low weight prefabricated steel tower units (total height of 241.85 m). The caissons were built in a purpose-built dry dock at the peninsula on the south side close to the South anchor block and subsequently floated out to a wet dock to allow sufficient depth for completion.

Figure *12* shows a section through the caisson and tower and the float out of the first caisson.

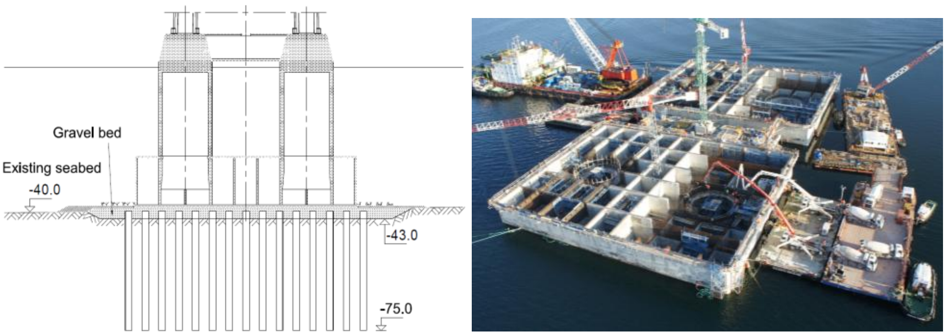

Figure 12. (a) Section through tower foundation; (b) float out from dry to wet dock of caisson

The soil is improved by 195 Ø2.0 m tubular steel piles with wall thickness of 20 mm placed in a 13 by 15 grid with centre to centre distance of 5 m with the top of piles 0.75 m below the top of the 3 m thick gravel bed. The steel piles are provided with external shear keys in the gravel bed to provide transfer of the requisite vertical and horizontal forces from the gravel bed. The hybrid foundation system allows for verification of horizontal and vertical bearing capacity and acceptable settlements.

After initial dredging at the tower positions the inclusion piles were driven and the gravel bed placed by tremie pipe from floating equipment. A purpose-built sub-sea levelling unit (Figure 13a) was used to achieve a level and uniform top of the gravel bed to allow positioning of the precast concrete caisson. To verify the stiffness of the produced gravel bed, gravel was tremied under site conditions (to 3 m thickness) into a large container, where plate loading tests were subsequently performed on land (see Figure 13b).

This allowed for an update of the initial settlements in the gravel bed. This was important as short and long term settlements needed to be compensated for before tower erection in order to meet the clearance requirements.

Figure 13. (a) Demonstration of sub-sea levelling unit; (b) Plate loading test on "retrieved" gravel

The most challenging part of the tower foundation design was the verification for the seismic events. This is described in details in [4] applying the base isolation concept which was previously used for the Rion-Antirion Bridge (see e.g. [5], [6] and [7]).

In order to generate stresses acting on the caisson bottom directly the modelling applied distributed springs at the 13 by 15 grid of the inclusion piles which provided sufficient resolution and accuracy. The springs were linear in the vertical direction but non-linear in the horizontal direction and included hysteretic behaviour. A very comprehensive calibration exercise was carried out in order to provide a match between the global IBDAS model and 2D plane strain Plaxis finite element modelling (vertical load) and ABAQUS 3D finite element modelling to determine the load-displacement behaviour of the gravel bed springs. The latter was achieved by applying different vertical loads to the caisson bottom slab and "pushing" it in the horizontal direction.

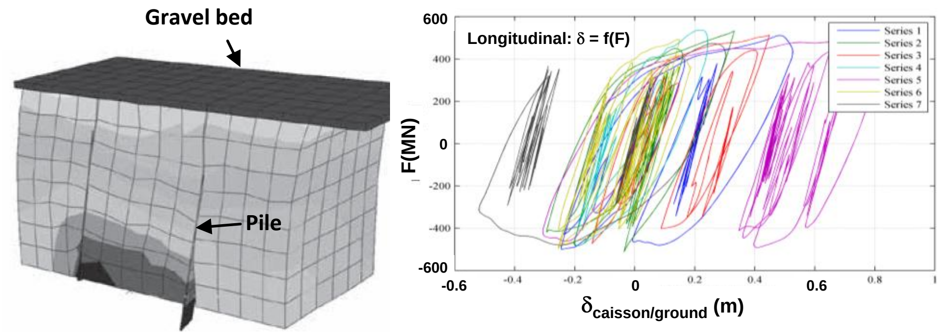

Figure 14. (a) Abaqus model for determining gravel bed springs; (b) Relative displacement at gravel bed/caisson base for NCE time histories for the North tower in the longitudinal direction

The relative displacement of the North tower in the longitudinal direction between gravel bed and caisson slab for the NCE time histories are seen in Figure 14 together with the Abaqus model for determining the gravel bed springs.

The application of an advanced non-linear model with distributed springs allowed implementation in a practical manner of a displacement based verification for high magnitude earthquakes with dissipation of seismic energy by rocking as well as a controlled and limited sliding.

The serviceability limit state verification involved calculations by an axisymmetric Plaxis finite element model to represent a pile inclusion in an infinitely large pile group. From the pile toe level and downwards the calculated settlements are manually modified to consider a load spread of 1:2 together with calculation of the contribution from creep in the clay layers. The resulting estimated settlements are seen for the North tower in Figure 15a.

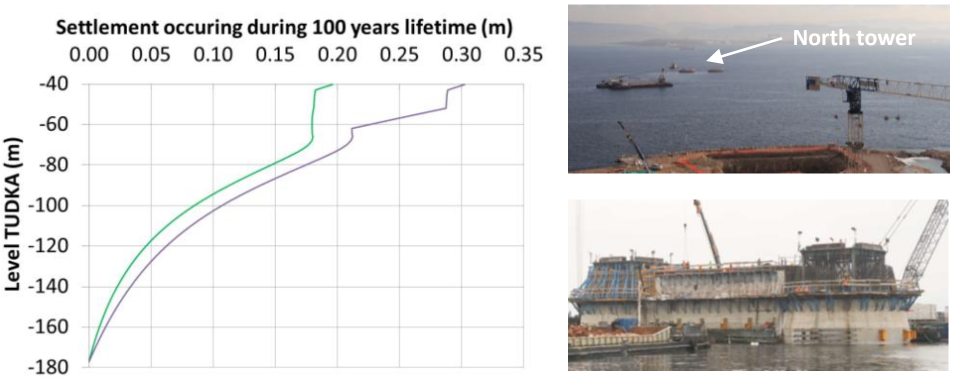

Figure 15. (a) Settlements estimated for the North tower during the 100 years lifetime; (b) Caisson on gravel bed (view from North side span pier); (c) Work for transition to steel tower erection

Intensive settlement monitoring is in place for the tower foundations. The initial settlements, recorded when the caisson was transferred from the wet dock and lowered on the gravel bed and the following weeks, indicated that the short term settlement prediction was accurate enough to allow erection of the steel tower without further compensations. The first steel segment was mounted in early July 2014.

4.4 South anchor block

In the Tender Design the layout of the South anchor block was in line with the North anchor block using two front pads and a rear pad. However, as the shallow ground conditions were relatively soft alluvial deposits the pads needed to be founded on piles to provide sufficient bearing capacity and acceptable settlements. The modelling of this layout and the choice of piles (bored piles versus barrettes in both cases inside an anchored diaphragm wall) to resist the earthquake were causes for concern. The particular problem of providing non-collapsible transition between the piles and the anchor block pads called for very intricate plastic hinge design. Any consideration of deep direct foundation was futile, as the ratio of horizontal to vertical load, H/V, was well above unity. The construction and the schematic layout of the South anchor block from the tender stage are shown in Figure 16a.

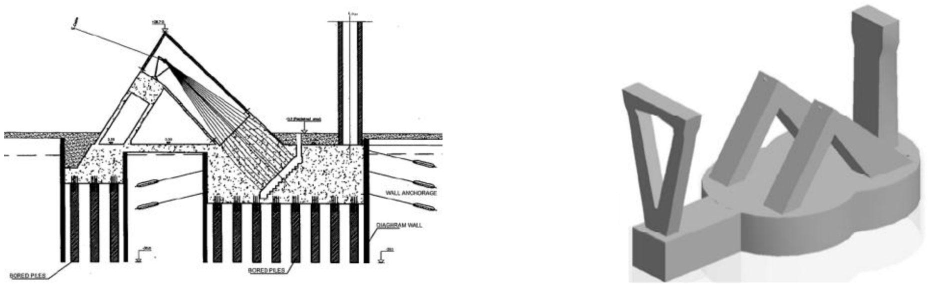

Figure 16. (a) Conceptual design of Tender design for South anchor block; (b) Alternative plan

As the detailed design progressed the loads changed (in part due to the relocation of the anchor block) and in view of the conceived risk of possible secondary faults, it proved beneficial to include the South side span pier in the anchor block structure. Thus, very late in the design process it became clear that due to the composite changes and more accurate modelling of loads in IBDAS that H/V was now considerably lower than unity.

Thus, as competent sand was found at a reasonable depth the concept was “overnight” changed to direct foundation at level -15 m. However, due to the poor ground conditions at shallow depth, the high ground water table and the risk of liquefaction in the top sand layers the challenge was how to establish the excavation for casting the gravity structure.

A new solution was arrived at where the side span pier would be an integral part of the structure and where ground anchors could be avoided. The latter was almost a must due to program problems resulting from multiple anchor levels and space constraints in the excavation. To resist the earth pressures a “banjo-type” solution with two overlapping circular walls (Ø 58 m) of secant piles with a capping beam as internal ring support allow space for the splay chamber as shown schematically in Figure 16b. The length of the anchor block is 124 m.

The rectangular front of the anchor block (37.5 m by 18 m) the secant pile walls are supported by internal strutting. The secant piles served two purposes, to allow water cut off and to retain the earth pressure. The supplementary investigation hence also aimed at finding the depth to the continuous clay layer serving as a cut off and to allow sufficient soil plug weight inside the walls to eliminate the need for dewatering.

Figure 17 shows the South anchor block in its position at the reclaimed area. At the intersection of the two circles an internal temporary support is established by consecutive concrete slabs supported by steel piles.

Figure 17. South anchor block after completion of excavation and initial concreting at the base

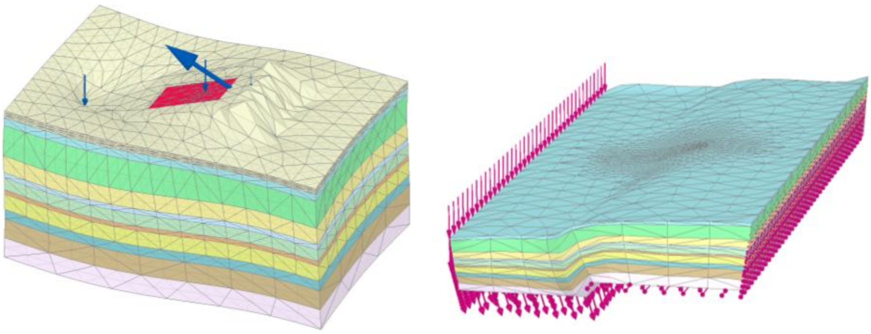

Figure 18. Exaggerated displacements for 1.0 m horizontal and 0.5 m vertical slip at 100 m depth

The very robust design of the anchor block as one big gravity structure had the added advantage that it could be verified even when subjected to severe oblique-slip fault movement, cf. [6]. The verification entailed very sophisticated 3D finite element modelling with an innovative cushion material as horizontal boundary as described in [8].

An example of the graphical output is shown in Figure 18. Furthermore, the geo-technical verification of the South anchor block involving detailed modelling of the geometry in 3D finite element and the impact of liquefaction is presented in [9].

5 CONCLUSIONS

This extremely interesting project posed a multitude of geotechnical challenges. The project re-emphasized that in geotechnics you must expect the unexpected. By dedicated cooperation between geotechnical and structural engineers and very fruitful cooperation with the Contractor IHI it was possible to overcome the challenges using innovative solutions and readiness to adapt to changing conditions brought about by the ground conditions, the construction programme and economy.

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