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Rion – Antirion Bridge Foundations: A Blend of Design and Construction Innovation

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RION – ANTIRION BRIDGE FOUNDATIONS: A BLEND OF DESIGN AND CONSTRUCTION INNOVATION

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ABSTRACT

This case history discusses the design and construction of the appropriate foundation scheme for the largest cable-stayed bridge in the world. This concession project was financed through a combination of public funds, private equity and bank loans. When completed in 2004, the Rion-Antirion Bridge will connect the Peloponnese, Greece's southernmost peninsula, with the mainland across the Gulf of Corinth.

Alternative foundation concepts that were considered included traditional driven piles, deeply embedded caissons, and soil improvement. The process of how the foundation evolved from schematics to final design, what were the driving forces, how ideas were disseminated among the Design Team, the Design Checker and the Technical Advisors, and the performance of the foundations to date are summarized.

OVERVIEW

The Gulf of Corinth effectively divides Greece by separating the Peloponnese and the Sterea Hellas Prefecture. The Peloponnese is a peninsula forming the southern part of Greece, and its only connection to the mainland is an isthmus at the eastern end of the Gulf of Corinth. This has resulted in a disproportionately higher land use and development in the east, compared with and the west areas of the country. The dream of a permanent link at the western end of the Gulf has been nurtured since ancient times, however, it never materialized due to the formidable geo-physical challenges present in the area. The Rion–Antirion Bridge, now under construction as the longest cable-stayed bridge in the world, will finally provide a fixed link crossing spanning this western end of the Gulf of Corinth and will become an integral part of the Greek Road Master Plan (Fig. 1). This paper describes the history of, the design and construction of the foundations for the Rion Antirion Bridge.

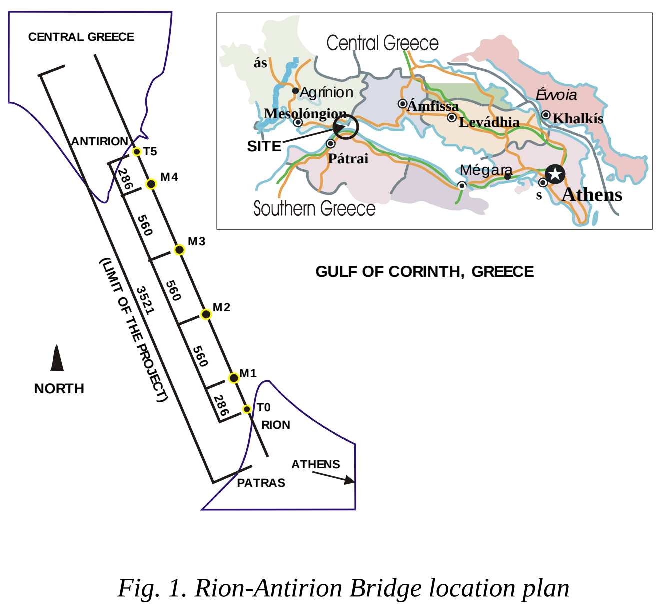

Fig. 1. Rion-Antirion Bridge location plan

History of the Project

Crossing this 2.5-km-wide strait is currently provided by a slow and heavily congested local ferry service made up of small and medium-sized vehicle/passenger ferries. Current traffic demands cannot be accommodated by the ferry service with crossing times often reaching two hours or more during busy traffic times.

In the 1980's, the Greek Ministry of Public Works, issued the initial tender for the project as a design-build. The tender's reference scheme called for a long-span suspension bridge with two abutments supported on pile foundations. This tender did not result in the successful selection of a design-builder. In 1993, after additional technical and environmental studies were completed, the Ministry re-issued tenders as a Concession Project, i.e., design, build, finance, operate and maintain. The tender included limited preliminary subsurface information and detailed performance specifications, with the tenderers being free to determine the specific most appropriate to meet the significant site constraints and challenges. A major step occurred during the tender process, when in 1994, as part of its strategy to promote cohesion between the Union members, the European Union confirmed the Rion-Antirion Link as one of its 14 priority infrastructure projects. As part of the "Trans-European Transportation Network", the Rion-Antirion Bridge, connecting the towns of Rion on the Peloponnese and Antirion on the Sterea Hellas prefecture, would form a vital link for enhancing the European economy through efficient distribution of goods and circulation of people throughout the region by providing the necessary roadway link between the ports of Patras and Igoumenitsa to new points of entry from the ports of Southern Italy.

In January 1996 the Greek Parliament ratified the proposal presented by the successful tenderer. A 42-year Concession Contract was executed with a French-Greek joint venture, Gefyra S.A. (referred to herein as the Concessionaire) for a 2,252-meter-long five-span cable stayed bridge. The contract called for 2 years to complete the final design (Preparatory Period), 5 years for construction of the bridge (Construction Period) and 35 years for Operation and Maintenance. The project sponsor and principal shareholder is the French construction conglomerate VINCI, S.A. Some of the innovative ideas proposed in the concept design included soil reinforcement below each of the four main bridge piers, special drop-in-spans along the deck to accommodate the specified vertical and horizontal tectonic movements between the piers, and huge seismic isolation devices to reduce vibrations and horizontal forces transferred to and from the superstructure and the foundations.

Although the project was ratified in 1996, negotiation of the final agreement, including securing financing of the project, still had to take place. The total project cost of about 732 million Euros included development costs, capitalized interest and other financial charges. The underlying 585 million Euro fixed price design build contract assumed all risks associated with the design, construction, cost over-runs, delays and currency fluctuations. The lending sources included 69 million Euros in Concessionaire's equity, 305 million Euros in Greek State's contribution and 370 million Euros in European Investment Bank (EIB) loans. The EIB 25-year floating rate loan is drawn in its entirety during the Construction Period, while during the Preparatory Period, the project was funded by the Concessionaire's equity and the Greek State's financial contribution. A Letter of Credit Facility of 407 million Euros was provided by commercial banks to guarantee the EIB loan during the Construction Period. In order to expedite the design process, and in accordance with the project specifications, extensive field investigations were performed in the fall of 1996 prior to the Financial Closing through a special agreement with the Greek State. Financial Closing was achieved on 24 December 1997, which marked the official date for the start of the 7 year Design and Construction period. The terms of the contract anticipate that the bridge will open to traffic by 24 December 2004: but this may be advanced, as at this time, construction is about 7 weeks ahead of schedule.

TEAM MEMBERS

A concession project of this magnitude and technical difficulty requires numerous participants that contribute in many different and significant ways, while safeguarding the diverse interests of the various stakeholders. A simplified organization chart is shown in Fig. 2. The Contractor is responsible for the design and construction of the bridge. Design checking and construction supervision are provided by two independent engineering firms reporting both to the Concessionaire and the Greek State. The Design Checker is the Canadian firm, Buckland and Taylor, with specialty consultants Professors R.B. Peck, R. Dobry, N. Priestley and F. Seible; they collectively provide an independent confirmation of the design proposed by the Contractor including design reviews, approvals and certifications. The Supervision Engineer is the British firm Faber- Maunsell (owned by the U.S. engineering group AECOM); their primary responsibility is monitoring of the progress of the works, workmanship and conformance with the specifications and construction documents. Quality control during construction is provided by Bureau Veritas. The New York firms of Parsons and Langan Engineering and Environmental Services, P.C. were retained by the Commercial Banks as Technical Advisors to provide independent technical review prior to the Financial Closing and throughout the preparatory and construction periods.

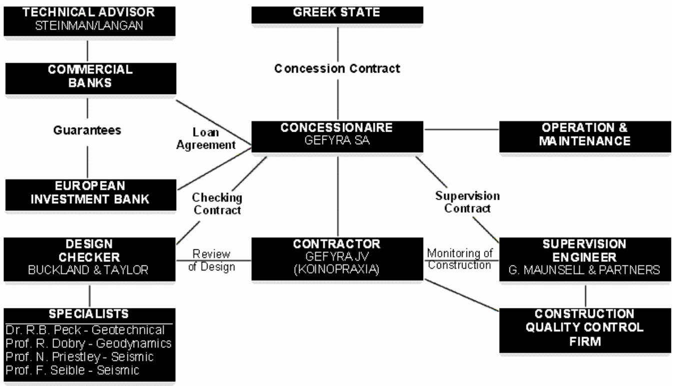

Fig.2. Project Organization Chart

ORIGINAL DESIGN CONCEPTS

The sea depth at the bridge site exceeds 60 m, and is in an area known for its high seismic activity and large tectonic movements. The strong motions are characterized by a design spectrum (KME, 1992) with a peak ground acceleration of 0.48g at seabed level and a maximum spectral acceleration of 1.2g for periods up to 1 second (Fig. 3). Furthermore, the project specifications required that the bridge be able to accommodate permanent tectonic movements of 2 m at seabed level in any direction between any two adjacent pylons. Soil conditions below the seabed range from medium to poor quality, with bedrock believed to be at depths of over 500 m.

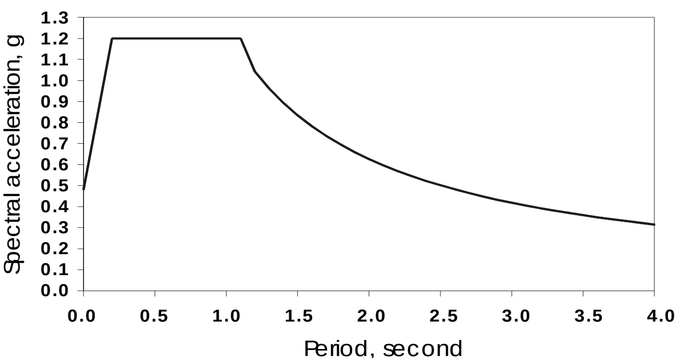

Fig.3. Design Response Spectrum (KME, 1992)

The Concessionaire's proposal consisted of a cable stayed bridge with a total length of approximately 2.9 km including a five span main bridge structure 2,252 m long and approach viaducts on each end. The bridge is to carry four 3.5-meter-wide lanes of Class 60/30 traffic, with two 1.9-m-wide shoulders for emergency stopping of vehicles. It has a total width of 27.2 m. The roadway deck elevation of the main bridge is situated approximately 50 meters above sea level, to allow for maritime traffic. The Rion approach viaduct consists of ten spans with a total length of 392 m. The Antirion approach viaduct consists of six spans with a total length of 239 m leading to an approximately 580 m long embankment, which accommodates the toll plaza. The original design concept, submitted at the time of the tender, contemplated five drop-in-spans, each 50 m long, to be placed between each of the three main 560 m long cable stayed sections and at each approach viaduct end; the purpose of these drop-in-spans was to accommodate vertical and horizontal translations and rotations resulting from the every day performance of the bridge, as well as from the design seismic event and the specified tectonic movements at seabed level (Fig. 4).

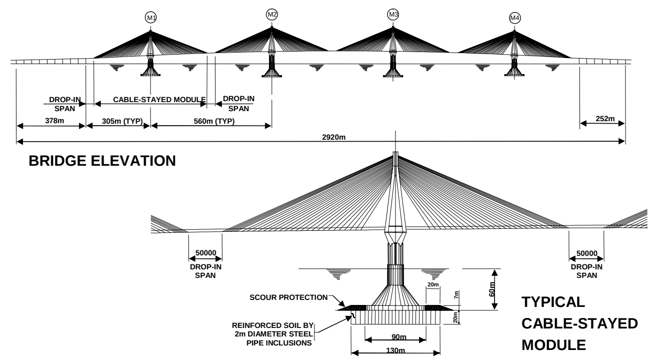

Fig.4. Conceptual Stage Bridge Profile

Traditional pile foundations were ruled out due to the depth to rock or other satisfactory bearing material. Instead, the seabed was to be reinforced with 2-m-diameter steel inclusions driven open-ended and projecting 0.5 m above the seabed. The base piers and the pylon structure were to rest on top of the inclusions and the 0.5 m void filled with cement grout contained at the perimeter by steel skirts. The purpose of the inclusions was to improve the shear strength of the upper soils and protect against seismic liquefaction. The grouted void had to provide a structural connection between pier base and soil foundation to transfer lateral loads and prevent sliding of the pier. The original design also utilized a unique, highly specialized base isolation system of shock absorbers, bearings, cables and stoppers at each bridge pier head immediately below the bridge deck, to reduce or "isolate" the horizontal seismic forces transferred from the foundation piers to the superstructure pylons and vice versa.

Several key issues with the original design concept were identified during the pre-Financial Closing period including:

- The behavior of the soil reinforcement concept and the validity of the relevant theoretical model.
- The soil reinforcement - pier foundation connection, including the evaluation of the effects of foundation uplift during seismic excitation.
- The behavior of the intricate base isolation system at each of the main bridge pylons; the proposed novel application constituted a leap forward in this type of utilization.
- The design details of the specialized expansion joints

and drop-in-spans to accommodate very large vertical and horizontal tectonic and seismic movements and the resulting maintenance/ replacement issues.

- The shape of the below water portion of the piers which was believed to be difficult to construct and would possibly induce higher than desired hydrodynamic forces at foundation level.

In an effort to resolve the challenges resulting from the exceptional combination of deep water, weak alluvium, strong seismic activity and tectonic movements, all of the above constituted first-time innovative applications that went significantly beyond the current state-of-the-art. As such, they presented significant risk to the financial partners (loan guarantors) and were the focus of extensive and sophisticated evaluations during final design. In the end, as will be seen in the forward sections of this paper, all of these issues were resolved by adopting alternate design methods, intertwined with each other as the solution of each issue had a direct and distinct effect on the other.

SITE INVESTIGATION AND SOIL PROFILE

The Greek State implemented two sets of subsurface investigations (in 1988 and in 1992) in the general area of the bridge alignment prior to issuing the tender documents for bid. These investigations were intended to provide an understanding of the subsurface conditions and soil engineering properties sufficient for preliminary conceptual design by the bidders. In 1997, the Contractor supplemented the available investigations with the collection of direct SPT and undisturbed samples for laboratory testing, cone penetration tests with piezocone pore water pressure measurements, and seismic cone measurements directly below each pier location (Geodynamique, 1997). The depths of the various offshore exploratory methods ranged from 40 m to 100 m. Similar targeted subsurface geotechnical investigations were performed in the areas of the two approach viaducts.

Geotechnical laboratory testing included soil classification tests to determine the index properties of the soils, consolidation tests to determine the compressibility of the soils, drained and undrained triaxial compression and extension tests, simple shear tests to determine the angle of internal friction and shear strength of the soils, and cyclic undrained triaxial tests to determine the strength degradation potential of the soils.

Both the pre-tender and post-tender investigations indicated a heterogeneous subsurface profile consisting typically of a 4 to 7-m-thick surface deposit of gravel and sand underlain by interbedded sedimentary deposits of granular and cohesive materials (silty clays and clayey silts) with thin layers and/or lenses of gravel. The soils are more homogeneous beyond a depth of about 30 m and consist mainly of silty clays or clays. Rock is estimated to be over 500 m below the seabed level.

Generalized subsurface profiles are shown in Fig. 5.

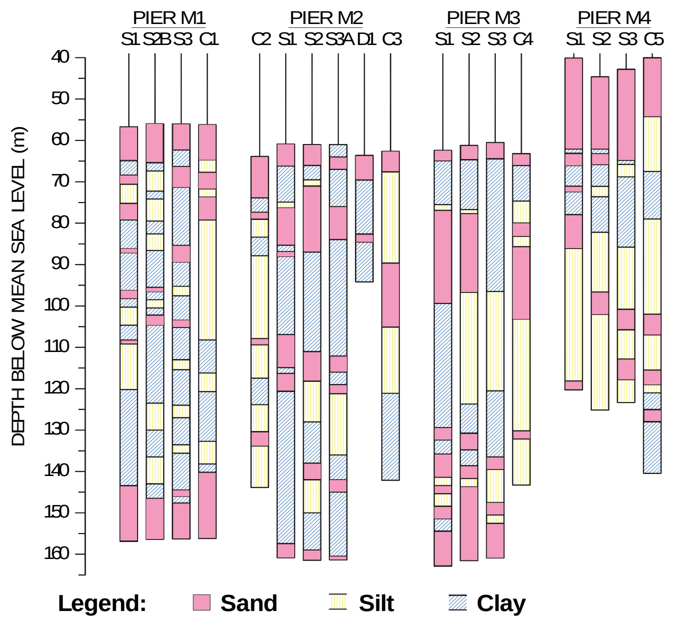

Fig.5. Generalized Soil Profile

The soil strata are mostly normally consolidated. The upper clay material is over consolidated although the over consolidation ratio tends to decrease with depth to about 1.3 at a depth of about 50 m. The undrained shear strength varies form about 30 to 50 kPa at the seabed to about 100 to 150 kpa at a depth of about 50 m. The average shear wave velocity of the soils varies from about 100 m/s at seabed to about 400 m/s at a depth of 100 m. Lower bound shear strengths were estimated to reflect post-liquefaction residual conditions. Upper bound shear strengths were computed corresponding to the apparent drained shear strengths. Fig. 6 gives typical shear strength data under Pier M3.

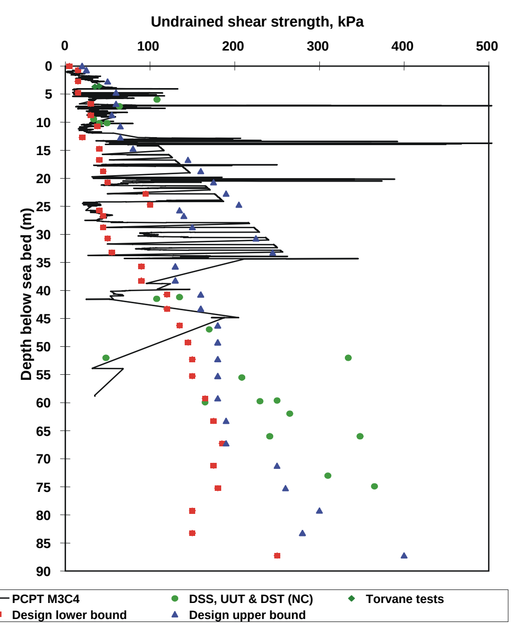

Fig.6. Undrained Shear Strength at Pier M3

FOUNDATION ANALYSES

Soil analyses showed that a shallow foundation was a satisfactory solution as long as the top 20 m to 30 m of soils could be improved to the point where the shear strength would be sufficient to withstand the large seismic forces as well as hydrodynamic water pressures likely to be experienced during the design earthquake. The behavior of the shallow foundation system was evaluated using analytical and numerical methods, including limit analyses based on yield design theory and 2D and 3D non-linear finite element models (see Fig. 7). The finite element analyses modeled explicitly the interactions between pier base, soil mass and steel inclusions. The numerical models predicted different failure mechanisms for the various horizontal loading conditions. The pier base was modeled as a steel frame using beam elements. The inclusions were modeled as beam elements with limiting shear capacities at the top to allow for slippage. The gravel base was modeled with a drained friction angle of 40°. The pier base-gravel interface was modeled with an interface friction angle of 35°. Finally, the in-situ alluvial materials were modeled using the lower bound undrained shear strengths, which allowed sliding and pier uplift to develop. Varying overturning moments as a result of load eccentricities were determined by imposing a horizontal load at increasing distances above the seabed. See Fig. 8.

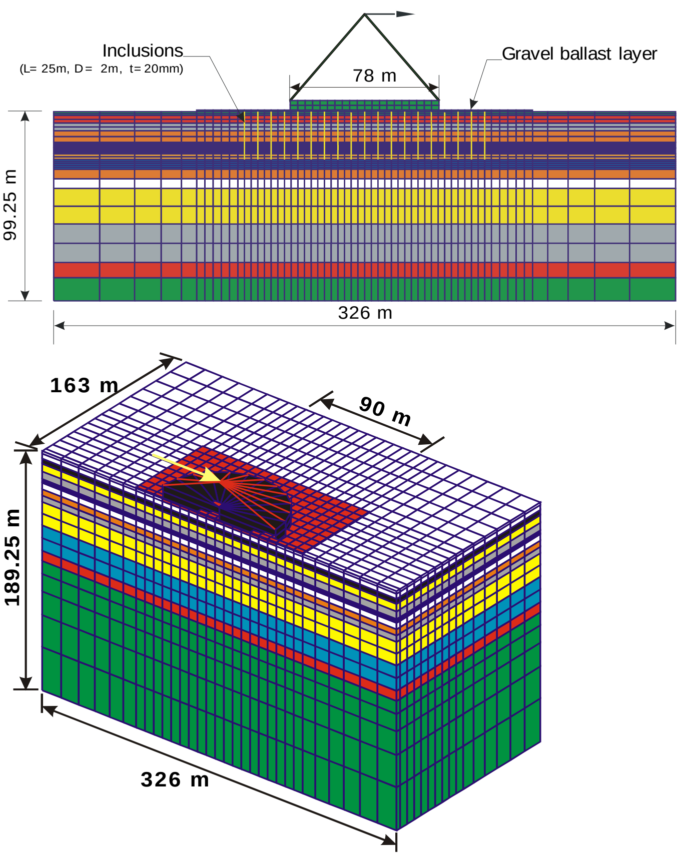

Fig. 7. 2D and 3D Finite Element Models for Pier M3

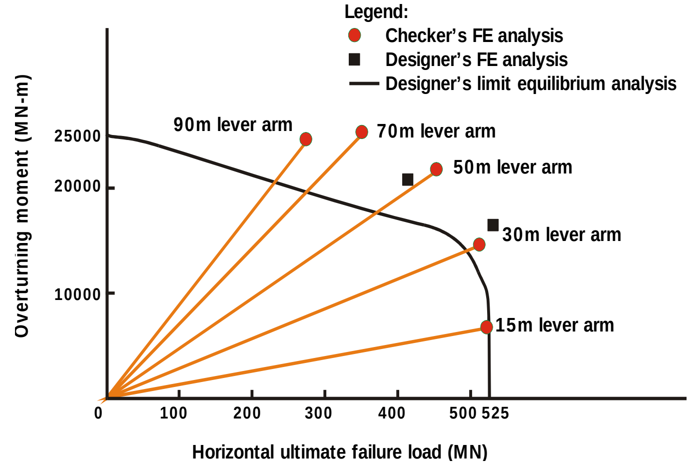

Fig. 8. Comparison of 2D analysis results for Pier M3

A series of centrifuge model tests were performed to validate the model concepts by providing information on the ultimate lateral bearing capacities of the foundation and its failure behavior. The failure modes obtained from the centrifuge tests were compared to the failure modes predicted by the numerical model. Three distinctive failure mechanisms were predicted from the soil-structure interaction modeling: a sliding mode, a combined sliding/rotational mode, and a rotational mode; while the centrifuge test results indicated two distinctive failure features: digging of the front toe into the soils and uplift of the tension side of the footing. The numerical models confirmed that the steel inclusions provided additional shear resistance in the soil and tended to act as load paths to transfer loads into deeper and stronger soil strata near their tips. See Figs. 9, 10 and 11.

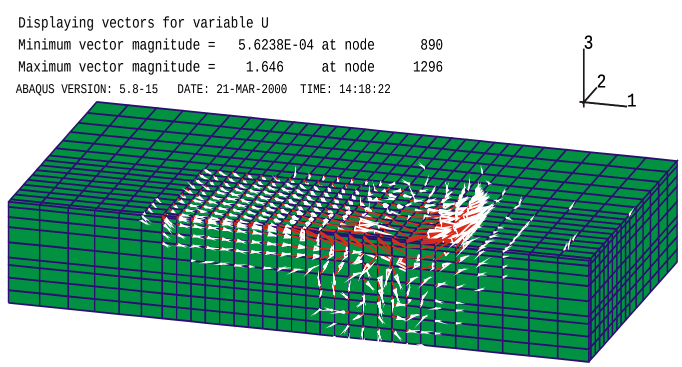

Fig. 9. Displacement vectors in soils (lever arm=30m)

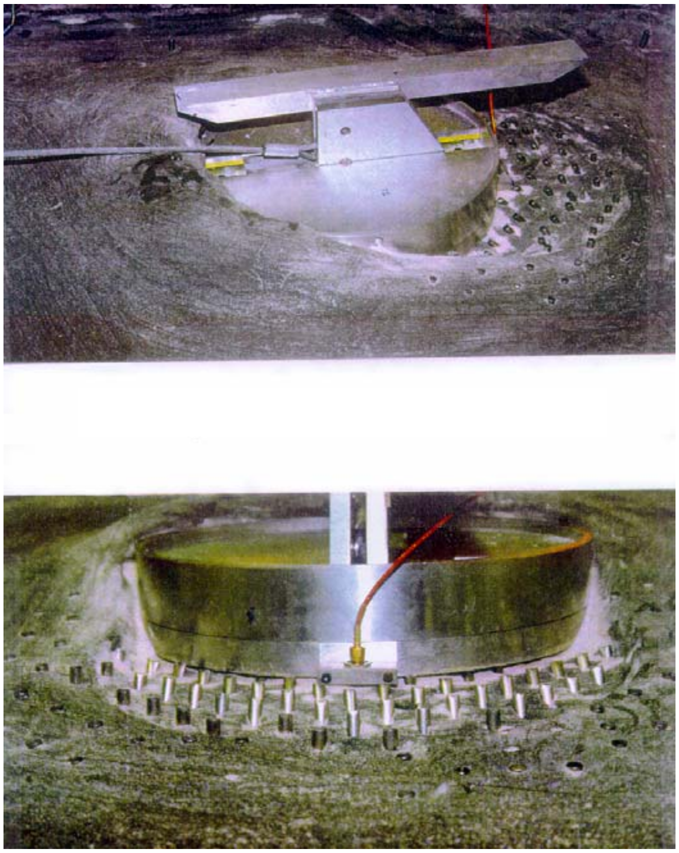

Fig.10. Failure behavior from the centrifuge test (full-scale equivalent lever arm=30m)

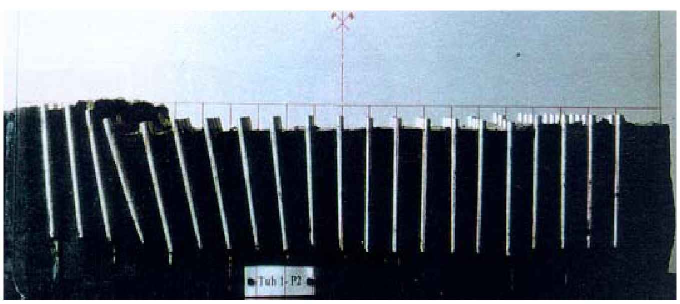

Fig.11. Pile deflections in centrifuge model (full-scale equivalent lever arm=30m)

A combination of traditional methods and refinements in the shapes and connections of the various elements of the bridge was considered as a more prudent and cost effective means of reducing the horizontal loads experienced by the foundations and the superstructure rather than providing base isolation at each pier head. The models confirmed the concept of a sliding failure mechanism of the pier base at the gravel interface. The gravel layer allowed plastic deformation of the pier base and the inclusions prohibited deep seated rotational failure. See Fig.12.

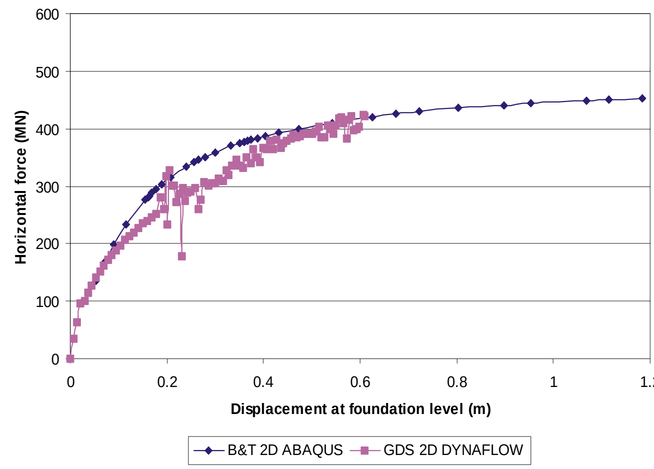

Fig. 12. Comparison of load-displacement curves for Pier M3 (lever arm=50m)

FINAL DESIGN

As a result of the design process, significant changes from the original design concept were adopted into the approved Final Design. These changes included:

The cable supported deck is a continuous composite system suspended from each pylon head in lieu of direct fixation at the pier head. This allowed elimination of the concept design's 50 m drop-in spans at the mid-point of each cable stayed main span (see Fig. 13). Also, the original intricated base isolation system between the bridge roadway and each pier base was substituted by a system of dampers and fuses controlling the movement of the deck where it runs through the pylon legs (see Fig. 14). Transition piers were provided at the two ends of the deck where it meets the most rigid pile supported approach viaducts; they were capable of accommodating three dimensional displacements and rotations (see Fig. 15).

Fig.13. Finalized Bridge Profile

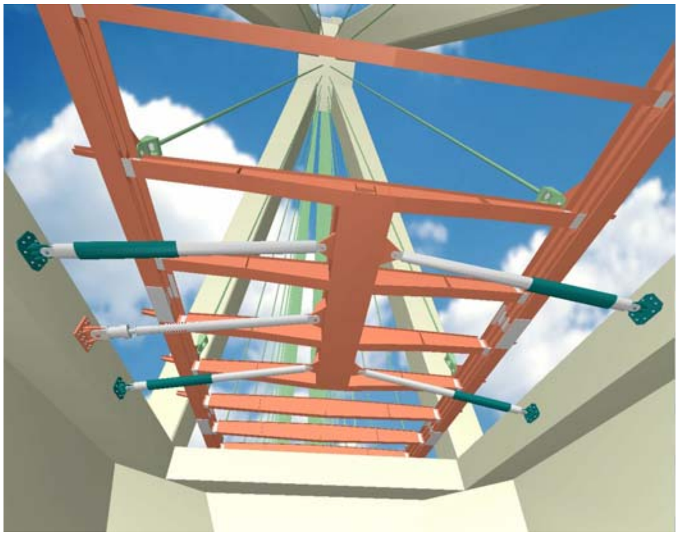

Fig.14. Deck damping system at pierheads

Fig.15. Transition pier details

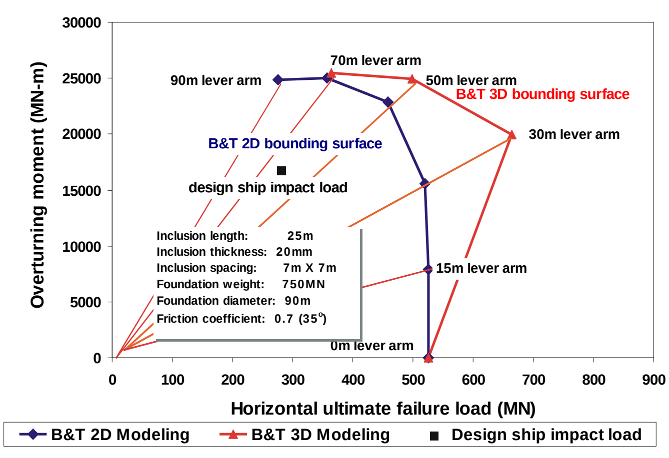

Fig. 16. Comparison of 2D and 3D results for Pier M3

1. Extensive analytical and numerical studies confirmed that the steel inclusions are effective soil reinforcing elements that significantly increase the shear resistance in the soil mass and bridge the weak in-situ soil layers at shallow soil depths. The spacing of the inclusions beneath each pier base was optimized through the use of centrifuge tests, numerical modeling, and parametric studies. Thus the grid spacing of the inclusions was increased to 7 m and 8 m from the initial uniform 5 m, while some peripheral inclusions were eliminated resulting in an overall decrease of nearly half the originally proposed inclusions. The use of 3D finite element modeling and the validation through extensive centrifuge testing were instrumental for the optimization of the inclusion spacing under each pier; see Fig. 16 for comparison of 2D vs. 3D results for the 7m x7m inclusion grid spacing at Pier M3.

2. Three of the four piers (M1, M2, and M3) rest directly on a 3-m-thick filter and gravel ballast layer placed over and around the soil reinforcing inclusions. At Pier M4 near the Antirion shore it proved beneficial to increase the dredging quantities to reach a deeper gravel deposit and eliminate the inclusions. At piers M1, M2 and M3, the inclusions increase the shear strength of the in-situ soil without being connected to the pier base; the top of each inclusion is 0.75 m below the base of pier. The non-connection of the inclusions to the pier base limits the inertial shear forces that can be generated by the superstructure during seismic events. All piers act as gravity base structures free to slide during seismic events providing additional isolation of seismic forces. The internal hysteretic damping of the piers provides a large portion of the total available damping. See schematic representation of the foundation details in Figs. 17 and 18.

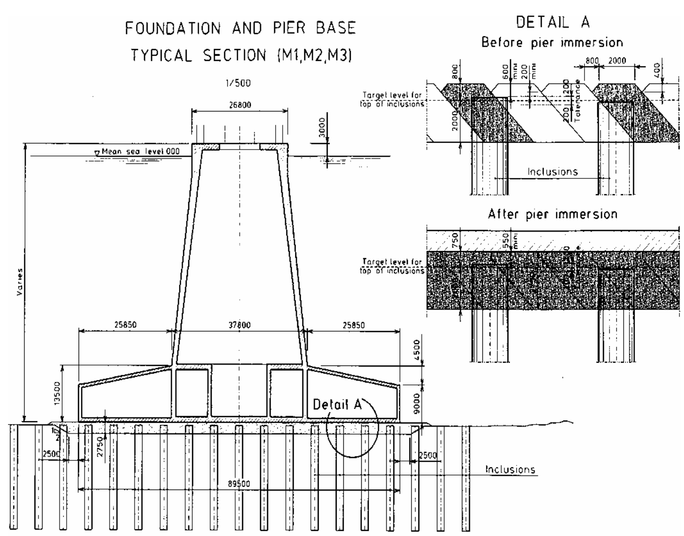

Fig. 17. Typical section of the foundation design

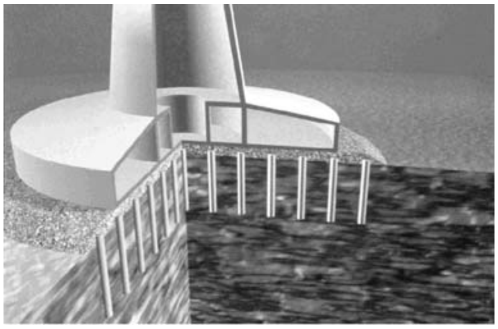

Fig.18. Foundation cross-section of a bridge pier, showing gravel layer, natural soils and steel pipe inclusions

3. The shape of the immersed portion of the piers was simplified. The original design entailed a combination of a cone with steep wall slopes in the lower portion of the pier and a cylinder in the upper portion, while the final design is a single cone for the entire depth which can be constructed using standard climbing formwork similar to that used in the construction of cooling towers. This shape change also had the beneficial effect of reducing the design hydrodynamic forces.

4. Preloading of the piers was incorporated into the construction procedure. Water ballast pumped onto the hollow chambers of the pier base was used to apply the full design load. This method allowed a full scale test to be performed and verify the behavioral characteristics of the underlying soil, prior to construction of the pylon legs and bridge deck.

All of the above design improvements were done while evaluating and refining the construction methods which are quite unique and involve techniques typically used in the offshore industry. Unlike traditional offshore structures, however, the specifications and tolerances imposed or afforded on a civil structure are much more stringent and tight. Consequently, all construction methods contemplated for this project directly affected the design in a way never experienced before.

CONSTRUCTION METHODS

A staging area of about 120,000 square meters was established on the Antirion side, east of the bridge alignment. This area is being used as fabrication and storage for reinforcing steel, concrete prefabrication, deck segments, fabrication of steel pipe inclusions, along with the batching plant and its storage needs for aggregates, cement, and water for the main bridge and the Antirion Approach Viaduct. A horse-shoe shaped dry dock (about 250 m by 100 m) was created for the construction of the pier bases. In a departure from the tender phase concept, the original single dry-dock was modified to accommodate the construction of two pier bases at a time. A costly floating front gate was replaced with a sheet pile supported dike, which allowed the dry dock to be sealed off and de-watered for the construction of the first two piers. When the first inland pier reached a proper height, the dike was removed and the pier was floated out to the wet dock; it was then replaced by the second pier which essentially acted as a “dam” (with additional sealing walls on each side) allowing for the dry dock to be de-watered again. This cycle was repeated to complete the remaining pier bases.

A second staging area, the wet dock, was used for construction of the submerged portion of the main piers prior to sinking them at their final locations. This second staging area was connected to the Antirion shore. The floating pier base was held in position by three steel mooring chains; one chain anchored on land, while the others anchored to single 2-m diameter steel piles driven into the seabed. A third staging area of approximately 15,000 square meters was established on the Rion side to accommodate storage and prefabrication of reinforcing bars, offices, a warehouse, and a service area for embarking on small boats. The purpose of this staging area is to support the construction of the Rion Approach.

Construction of the foundation piers was performed by employing methods and equipment typically used for offshore gravity oil platforms. The pier base was constructed in the dry dock on the Antirion staging area using tower cranes one of which was later fixed to the base and followed the pier shaft throughout its construction. When the dry dock concrete works were completed (approximately 18,000 cubic meters of concrete for each pier), the dry dock was flooded, the dike removed and the pier base towed to the wet dock using tugs. At the wet dock, the piers were moored in 60 m water depth. Construction of the pier shaft continued in lifts using sea water as ballast to control trim, freeboard and stability. Work at the wet dock proceeded to the height necessary for the pier shaft to be stable and to extend above the water level after each pier was placed in its final position.

Prior to the tow out of each pier from the wet dock to its final position, the seabed had to be prepared to receive the pier foundations at each pylon location. This included relocating existing high-voltage electric cables that were resting on the seabed, excavating the upper soils and leveling the seabed at each pier location using a remotely operated dredging vehicle, placing the gravel ballast bed to very tight tolerances, and installing the soil reinforcing pipe inclusions. The gravel ballast and the steel pipe inclusions were installed from a specially designed barge kept in position by four 700 ton counterweights and the principle of tension leg platforms. The barge was equipped with six guiding cages that allowed the driving of the inclusions using an underwater hydraulic hammer. Upon completion of the seabed leveling and installation of soil reinforcement, the piers were towed to their final position and “sunk” into place by ballasting its hollow chambers with sea water. Positioning was controlled by GPS and was accomplished to within 5 cm to 35 cm of each pier’s theoretical location. Piers M1, M2 and M4 were placed within the specified tolerance of 10 cm, while M3 (the first pier to be floated to its final location) was placed 35 cm off and required minor design adjustments of the bridge alignment details. At the present time, all four pier bases have been placed at their final position and the pylons have been constructed above; installation of the deck segments and cable stays is in progress.

PIER PERFORMANCE

Settlements were estimated for each pier base. The vertical stress distribution was analyzed using 3D Finite Element analyses to model the specific characteristics of each pier, imposed load, pier diameter and foundation subgrade and to account for the length and spacing of inclusions. The stress distribution was computed to a depth of 120 m taking into account the unloading stresses due to excavation. Compressibility parameters of the soil were determined coinciding with the available CPT records. The direction cosines of the vector normal to the plane were computed to obtain the direction and magnitude of the maximum foundation tilt for each pier.

Inclusion installation records were kept during driving. Unlike typical piles, there were no driving criteria for the steel inclusions. Instead, hammer blows and total transferred energy were plotted with depth on an inclusion basis and most importantly on a quadrant basis for each pier. The intent was to identify potential weaker areas that could result in excessive settlement and/or tilt. See typical graph for Pier M2 in Fig. 19. Water ballast preloading begun soon after each pier was in position and movements were monitored. Recorded settlements and tilt would be accounted for as the pylon superstructure construction progressed.

Table 1 gives a summary of the predicted settlement and maximum tilt of the foundation piers along with actual monitoring results during preloading. Actual movements are typically significantly less than predicted. Furthermore, there is a remarkable correlation between the recorded inclusion total driving energies and the corresponding tilt as can be seen when comparing Fig. 19 with the graph given for Pier M2 in Fig. 20. Due to the lower than predicted settlements, the top of Piers M1, M2 and M3 are actually higher than designed; the difference in elevation being corrected within the capping slab below the bridge deck.

Table 1. Pier Base Settlement Performance

Pier Base	Predicted Settlement (cm)	Actual Settlement (cm)	Predicted Tilt(%)	Actual Tilt (%)
M1	17	8.1	0.073	0.063
M2	28	13.2	0.126	0.095
M3	22	8.0	0.109	0.031
M4	2.7	6.1	0.181	0.010

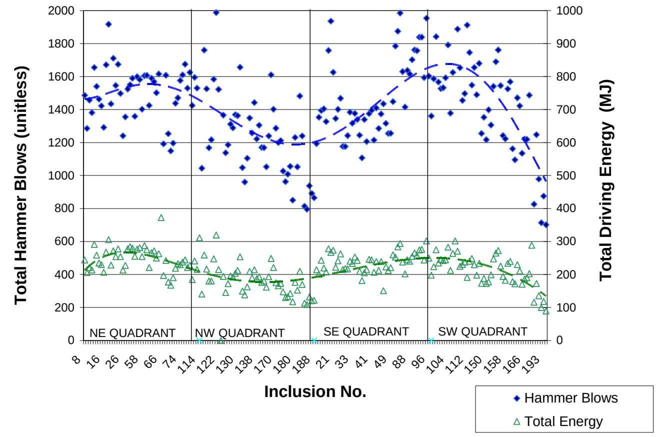

Fig. 19. Inclusion Driving Records for Pier M2

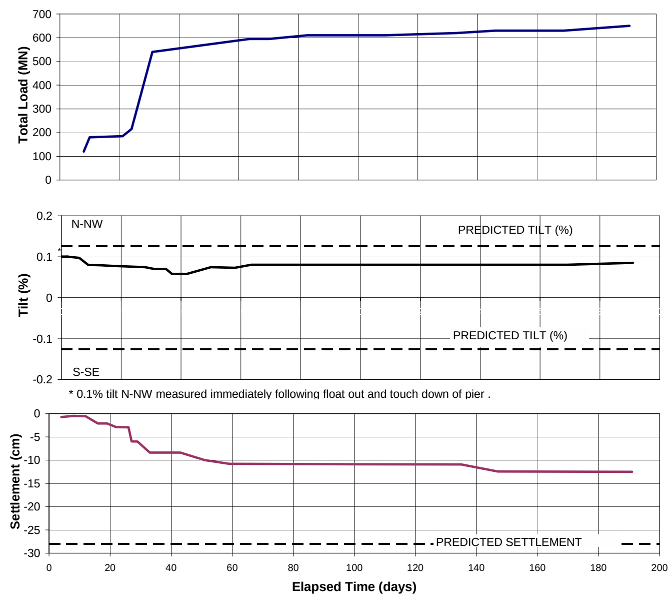

Fig 20. Measured Pier Settlement and Tilt under Preload (Pier M2)

CONCLUSION

Foundation construction for a bridge spanning the Gulf of Corinth, founded in 60 m deep waters on marginal soils was not without risks. The key for the Contractor to mitigating these risks was identification, assessment of probability, and development of contingency and/or risk management plans. Risks due to construction cost overruns were mitigated by the fact that the Concessionaire and the Contractor were solely responsible for all design and construction methods and associated costs and had the forthsight to heavily invest in the design and achieve a combination of minimum cost and practical time allocation.

The Contractor obtained critical highly specialized and often unique/build-to-suit pieces of equipment at the start of the project to achieve the desired results. The availability and capabilities of this equipment were factored in the design. The risk of potential accidents that could result in short term or permanent loss of this equipment was covered by insurance policies. Finally, another form of risk that had to be addressed was the shortage of skilled laborers for the unique type of work involved in this project and the strong labor unions in Greece. To mitigate these risks, the Contractor undertook a pro-active approach with the establishment of an on-site training center and program designed to develop a skilled labor pool of foremen, gang leaders and laborers necessary to meet the demands of the project. The Contractor opted to train locally rather than import skilled labor due to the language advantages and the local workers' good spirit and willingness to learn. While proper training may have caused some initial delays in the early stages of construction, the long term benefit has been justified.

In closing, we cannot overstate that partnering among the various team members was instrumental in achieving the desired end results. The Concessionaire fostered an unprecedented, in our opinion, spirit of collaboration and focus to a common goal. The design and construction process has been a remarkable experience that allowed significant challenges to been identified, solutions prepared and construction executed.

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