15 Aug 2008, 3:00 pm - 3:30 pm

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6th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA, August 11-16, 2008

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ABSTRACT

The Chacao Channel bridge, a planned privatized/concession bridge project, is to link the island of Chiloé with continental Chile through the Chacao Channel. When completed, it will be the longest suspension bridge in South America (2,365 m) with two approximately equal spans linking three pylons. The location of the bridge presents a combination of unprecedented challenges in the design and construction making it very unique, including: extreme seismicity with the largest earthquake recorded to date anywhere in the world having occurred very close to the envisioned bridge location (the 1960 Valdivia Earthquake with a moment magnitude of 9.5!); extremely strong sea currents and tide fluctuations within the channel (up to 5-6 meters/sec and 6 meters, respectively); the general area being surrounded by active volcanoes; and a history of Tsunamis. The above extreme challenges resulted in construction cost escalations; inevitably, in 2006, after a significant portion of the investigations and design was completed, the Chilean ministry of Public Works put the project on hold. The above extreme physical challenges were the main reason for the cost escalation. This paper provides an overview of the design of the envisioned bridge, discusses the methodology to overcome the unique challenges of the bridge location and focuses on the geotechnical and seismic aspects of the design.

OVERVIEW

The envisioned Chacao Channel bridge will be located in Southern Chile, Region 10, Los Lagos, Chiloe. The bridge will connect Route 5 South of Chile between the Continent and the Island of Chiloe, crossing the Chacao Channel (Figure 1).
Several facts make this project's location unique; they are listed below

- The bridge setting is located in an active seismic region, close to the location where the 1960 Valdivia subduction earthquake occurred; this is the largest earthquake reported in modern history with a moment magnitude of M_w = 9.5. This earthquake was associated with ground deformation over a distance of more than 800 km parallel to the trench, with subsidences up to 3 m and uplifts up to 6 m. The bridge area sunk approximately 2 to 3 meters.
- Based on reports from historic earthquakes, tsunamis have occurred in the region, with reported wave heights up to 20m.
- The sea level at the project location varies significantly, with maximum tidal ranges of about 6 m. Respectively, currents reach velocities of 5-6+ m/s (about 10-12+ knots).
- There are at least ten volcanic sources within a 200 Km radius area which represent potential natural hazard centers.

The Design was setup in the following phases before the envisioned commencement of construction:

- Sub-Phase I: Complementary Geological Investigations
- Sub-Phase II: Validation of Design
- Sub-Phase III: Final Engineering

The project was put on hold at the end of Sub-Phase II after a significant portion of the design was done. This paper provides an overview of the design of the envisioned bridge, discusses the approach to tackle the unique challenges of the bridge setting and focuses on the geotechnical and seismic aspects of the design.

Fig. 1. Bridge Setting

TEAM MEMBERS

The Ministerio de Obras Publicas (MOP – Ministry of Public Works of Chile) awarded the bridge design/build contract to the concession of Hochtief, VINCI Construction Grand Projets, American Bridges, Besalco and Tesca in 2005 (Sociedad Concesionaria Puente Chiloe S.A. – CPC). 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. The Contractor is responsible for the design and construction of the bridge. The design was lead by the internal design departments of VINCI Construction Grand Projets and Hochtief; the primary designers were COWI, in collaboration with Wolfgang Romberg (Geotechnical Expert) and Alain Pecker (Geodynamique & Structure - GDS - Geodynamics expert). The design check was done by an independent engineer working for the CPC, and an independent engineer working for the MOP. The CPC Design Checker was a joint venture of the English Firm Flint & Neil (London) and the American Firm Amman & Whitney (New York), with specialty consultants Langan Engineering and Environmental Services and Ricardo Dobry; they collectively provided an independent review of the design proposed by the Contractor including design reviews, approvals and certifications. The Design checker for the MOP was primarily ARUP with the assistance of several specialty experts.

DESIGN CONCEPT

<u>Past Studies</u>

Several studies were completed before the project was awarded to the concession. These studies include a Preliminary Investment Study in 1997 by Ingenieria Cuatro Ltda, a Study of Alternatives and a Technical Feasibility Study in 2000 by ICUATRO-COWI Joint Venture. The distance to be spanned is approximately 2200 meters, and a single span suspension bridge would require two pylons, approximately 300 meters high. (see Figure 2).

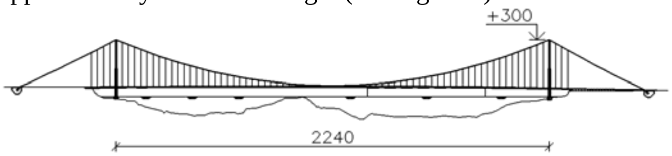

Fig. 2. Single-Span Bridge Concept (Ingeneria Cuatro/Cowi, 2001)

This very long span would drive the cost too high for the bridge to be built. However, roughly in the middle of the channel there is a submerged islet (called Roca Remolinos, see Figure 3) that could potentially be used to support a third pylon and cut the span in half. The biggest concern from the beginning of this project was if Roca Remolinos was strong enough to support a bridge pylon. Geotechnical investigations and several studies provided a design where the bridge with three pylons could be constructed.

The bridge scheme is a suspension bridge, with two main spans supported by three Pylons (North, Central and South) and having a continuous deck. The North (Continent) and South (Island of Chiloe) Pylons are to have a distance of about 1050 m and 1100 m from the Central Pylon. The three Pylons will rise about 180 m above the mean sea level (Figure 4 and 5). The positioning of the central pylon has to be in the middle to provide the symmetry necessary for the design (equal spans), and symmetry is what drove the positioning of the North Pylon in the water. If Roca Remolinos was closer to either shore a different scheme would be needed.

Fig. 3. Top of Roca Remolinos. Picture taken from a boat during low tide.

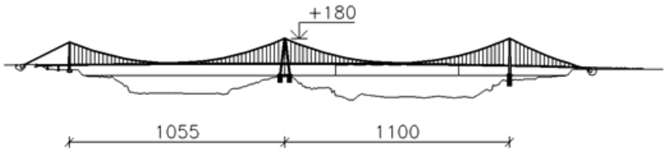

Fig. 4. Final Bridge Concept (Ingeneria Cuatro/Cowi, 2001)

Fig. 5. Rendering of Final Bridge Concept (Ingeneria Cuatro/Cowi, 2001)

The Big Unknowns

A number of very important questions were raised before the bridge was awarded to the concession. As mentioned earlier, the biggest unknown was if Roca Remolinos was strong enough to support the Central Pylon. A relatively weak Roca Remolinos could drive the foundation cost to a point which is prohibitive to the bridge. It will be shown that the concession demonstrated the feasibility of the project by performing additional geotechnical investigations, in-situ and laboratory testing and numerous desk studies focusing on the questions regarding Roca Remolinos; such questions include its genesis and susceptibility to erosion, especially given the very strong currents.

A different set of concerns regards the seismicity of the area. Initially an active fault was suspected to cross under the bridge. This would drive up the bridge cost as well to seismically proof it.

A combination of the strong currents and high seismicity raised concerns regarding the slope stability of the shores, especially at the locations of the North and Central Pylon.

All the above concerns were the primary focus of the preliminary design phase. The following sections provide a summary of the work performed during the first two sub-phases of the project, and target those concerns.

GEOLOGICAL ANALYSES

It is noted that very limited information exists in the literature regarding the formation of the channel and there were no substantiated theories explaining the genesis of Roca Remolinos. At the start of the project, the prevalent theory was that Roca Remolinos was the remains of a pyroclastic flow through the channel, but this theory was never substantiated. The above lead the designer to make an independent investigation to determine how Roca Remolinos was formed. Desk studies, field visits and geophysical investigations were performed and supervised by GDS to answer the questions regarding the existence and activity of the Golf of Ancud Fault (FGA), the Chacao channel formation and the genesis of Roca Remolinos (Figure 6). Regarding the FGA, no major deformation was evidenced in the recent plio-quartenary deposits of the Chacao channel area (with the exception of the elastic rebound after the major 1960 Valdivia subduction earthquake) and therefore FGA's activity was not substantiated. Based on the additional geophysical surveys (Marine Seismic Reflection and Multibeam survey), it was found that there is no active fault crossing the channel. FGA's location is only partially understood, but it is believed to be seated in the basement rock.

Regarding the Chacao channel formation, the geology and the geomorphology are the results of the large amounts of sand and gravels deposited by the glaciers, at different periods, over the neogene basement. The channel seems to be the expression of a major erosion feature probably formed during the last Llanquihe deglaciation when the sea level was much lower than the present one.

Regarding the genesis of Roca Remolinos, Roca Remolinos corresponds to glacial deposits of unknown age but probably older than the last Llanquihue glaciation found on both banks of the chanel. The cementation agent in the Roca Remolinos samples is described as ferric oxides, and could be created due to a pozzolanic reaction. The project was put on hold before the origin of the cementation was fully understood. Once the project is reactivated, a cement specialist will be needed to give an expert opinion on the stipulated process.

Fig.6. Aerial photo SAF-060453-Fondef of the Chacao Channel above Roca Remolinos (GDS, 2006)

SITE INVESTIGATION AND SOIL PROFILE

Preliminary Geotechnical Investigations

During the preliminary design, ICUATRO-COWI completed a preliminary geotechnical investigation in two phases:

• January 2000 to March 2000, at which time, four inland borings were drilled by Geovenor close to the locations of the anchor blocks.

• October 2000 to March 2001, at which time, thirteen borings were drilled by Geovenor; nine of which were drilled inland, and four were marine borings, drilled at the locations of the Central and North Pylon.

The four marine borings were done from a 185-ton, floating platform. At the time there was no jack-up platform available. The borings were drilled using an HQ-3 core barrel with a core diameter of 61 mm. In addition, an NQ-3 core barrel was used (with an inner diameter of 45mm) to drill a hole with the required diameter to perform Pressuremeter tests. It is noted that the recoveries of the four marine borings were low, and the Pressuremeter tests performed were limited.

Laboratory tests included Sieve analyses, unconfined compression, Consolidated isotropic undrained (CIU) Triaxial tests on saturated samples, Cyclic Triaxial tests (25 cycles), CIU Triaxial tests following Cyclic Triaxial tests, Shear wave velocity tests on unconfined samples, and Shear wave velocity tests on confined samples. Pressuremeter and Maritime Seismic Reflection tests were performed to complete the preliminary geotechnical investigations.

Complementary Geotechnical Investigations

Following award of the project to the CPC additional geotechnical investigations were performed, primarily to verify the strength of Roca Remolinos and make sure the project is feasible. As part of Sub-Phase I, an additional geotechnical investigation was performed at Roca Remolinos (Central Pylon) comprising six new marine borings (three borings for each of the two Central Pylon legs). An additional two marine borings were performed at the location of the North Pylon. In all borings a series of in-situ tests was also performed.

The borings were drilled using state-of-the-art drilling equipment from a jack-up platform (Figure 7). Two different core barrels were used, the NQ 3 (required for the pressuremeter tests), and the GEBOR S with an inner diameter of 101 mm. The GEBOR S was chosen for increased core recovery to identify the different soil layers. This larger diameter of sampling, together with the use of a jack-up platform were the two critical improved factors over the preliminary investigation, and proved that the soil cementation in Roca Remolinos is generally higher, and consistently so, than presented in the preliminary investigation.

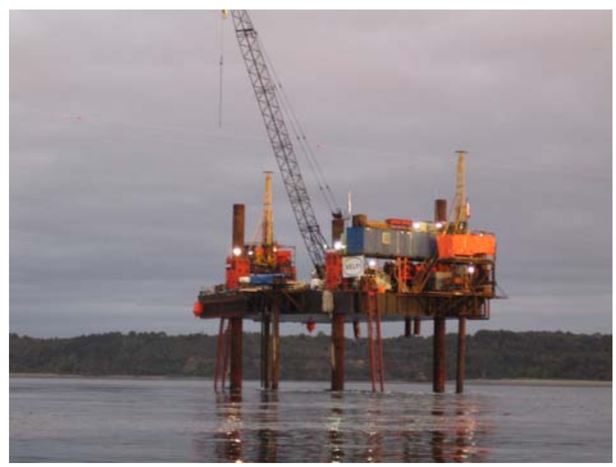

Fig. 7. Jack-up Platform on top of Roca Remolinos

In addition to the pressuremeter tests, suspension logging techniques were used to obtain additional in-situ data. These tests were performed with state of the art equipment (such as the P-S suspension logger by OYO). The parameters measured were: temperature, void ratio (Neutron log), density (gamma-gamma log), shear wave velocity and compressional wave velocity. The temperature data appear to have been influenced by the grout hydration in the borehole, but they are not important nor do they influence the other parameters measured.

In particular, the measurement of compressional and shear wave velocities are of paramount importance for earthquake design; the values reported in the original investigation were based on laboratory tests on samples some of which seem to have been significantly disturbed and were not considered to provide reliable results representative of the field conditions.

Laboratory tests on samples collected from the complementary geotechnical investigation were performed at the IDIEM laboratory at the University of Santiago, according to ASTM standards. The following tests were completed: unit weight, water content, grain size, unconfined compression, triaxial CIU, triaxial cyclic, direct shear and x-ray diffraction test. Discussions on the test results (both in-situ and laboratory) are provided below.

Results of complementary investigations

Roca Remolinos. Based on the sample visual inspection and test results, four different layers were identified (W. Romberg, 18 January 2005, Figure 8). All elevations given herein are below the Mean Sea Level, in meters.

1. A Tuffite (Caprock) layer exists (grey color), about 40-meter-thick, comprising of fine to medium sands, well to highly cemented, with low to medium silt content.
2. Below the Tuffite layer is a layer comprising of Pleistocene Sands about 35-meter-thick, comprising of medium sand, well to low cemented, with a few layers of gravel. Within the same layer are sublayers with concentrated gravel and low silt content. The layer colors vary from medium to dark grey and grey-brown.
3. Below the Pleistocene Sand layer, there is a layer of Pliocene/Pleistocene Silt about 30-meter-thick, comprising of hard or cemented Silt, with light brown to yellow color.
4. Below the Pliocene/Pleistocene Silt there is a layer of Pliocene/Pleistocene Sands, comprising of very densely compacted, partly cemented medium sands. The soil investigation was terminated about 20 meters inside this layer.

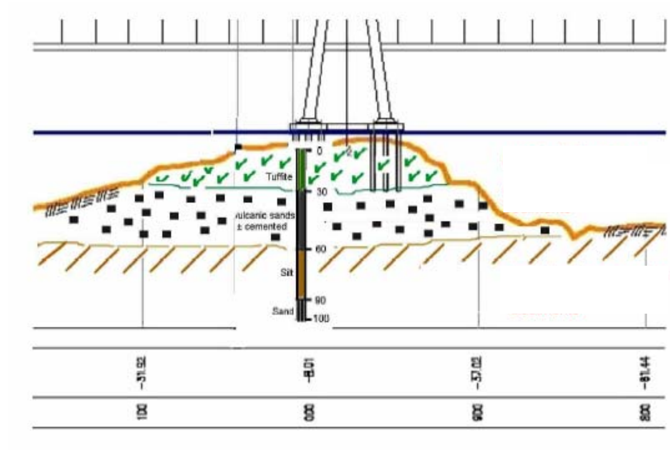

Fig. 8. Roca Remolinos soil layers (W. Romberg, 2006)

Some representative results from the in-situ tests at Roca Remolinos include the following:
• The P-S suspension logging results showed shear wave velocities with typical values between 1000 and 1400 m/s for the Caprock layer, between 600 and 800 m/s for the Upper-Sand layer, and about 700 m/s for the Silt layer. No measurements were taken for the Lower-Sand layer (see Figure 9). The high shear wave velocities for the Caprock layer are indicative of a rock material. It is noted, however, that the material cementation varies (see below) and once broken the material behaves like soil rather than rock.
• The mass density varied from about 1.7 to about 2.5 gr/cm^3 with typical values of about 2.1 gr/cm^3 for the whole depth.
The laboratory test results contained in the IDIEM factual report, showed the following:
• The unconfined compression strength from all tested samples varied widely. Specifically, for the Caprock layer, the unconfined compression strength varied from about 0.35 to 12.9 Mpa, with typical results between about 5 to 11 Mpa. The large scatter is attributed to various degrees of cementation for the different samples and different disturbance levels.
• The Direct Shear tests were performed for three levels of vertical stress (0.05, 0.15 and 0.25 Mpa). Tests were done in the Upper-Sand layer, the Silt layer and the Lower-Sand layer. For all layers, the effective friction angle (ϕ') varied from about 38° to about 44°, and the cohesion (c') ranged from about 0.01 to 0.026 Mpa.

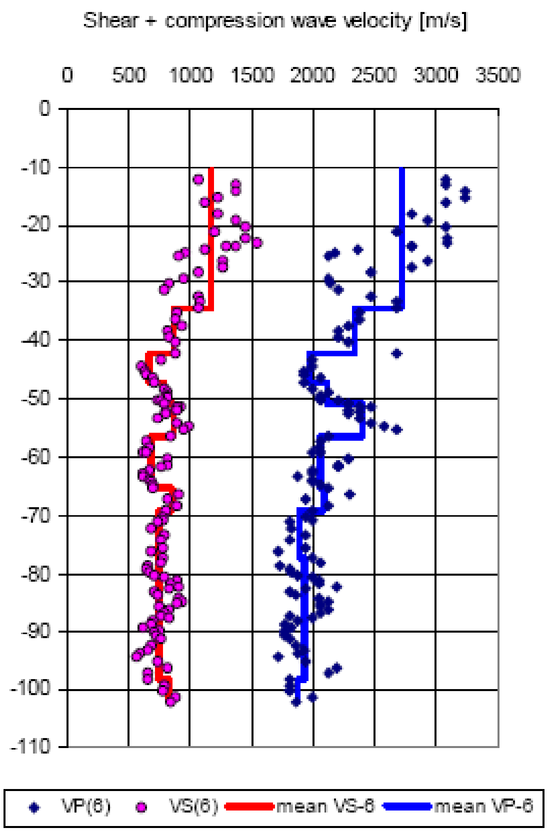

Fig. 9. Shear and compressional wave velocity profile at Roca Remolinos from suspension logging (boring A.SM-6, W. Romberg, 2006)

<u>North Pylon.</u> The complementary investigation revealed the following two soil strata; an approximately 60-m-thick sand layer and a layer of Silt below. The sand layer consists of very dense, mostly non-cemented sands (with a few embedded thin layers of silt) which have been geologically preloaded by glacial ice. This is confirmed from the recovered samples, and the in-situ and laboratory tests results which indicate high shear wave velocity values, and high angles of friction. The sand underlying the North Pylon is different from the sand layers encountered in the Central Pylon location (Roca Remolinos), mostly because it is uncemented. The shear and compressional wave velocities as well as the lab tests indicate that this sand is a weaker soil material than the sands encountered in Roca Remolinos, and much weaker than the Tuffite appearing in the top 30 m or so in Roca Remolinos. The Silt layer encountered in this investigation is also different from the Silt layer encountered at Roca Remolinos.

Based on the lab tests, ϕ and c values for the Sand layer were about $\phi = 40°$ and c = 0.0 MPa. For the Silt layer ϕ was estimated varied between 22.5° and 27° and c was estimated between 0.2 and 0.6 MPa.

The above investigations provided sufficient information to the CPC to proceed with the design of the bridge.

PROPOSED FOUNDATIONS

The general foundation scheme was setup in the technical feasibility study by ICUATRO-COWI. Based on the given at the time geometry and soil conditions, the bridge foundation will include two anchor blocks, a shallow foundation for the south pylon, and deep foundations for the central and north pylon.

The complementary investigations and studies showed that in general, the soil strength parameters were the same or better than those assumed in the feasibility study, thus the same foundation concept was kept.

The two envisioned anchor foundations are massive concrete blocks, approximately 33 and 29 m high (south and north respectively), with a width of 35 m and a length that varies, reaching 54 m. The foundation of the South Pylon consists of two square footings 19 m x 19 m, 7-meters-thick, connected together with a beam 4 m wide by 3 m deep by 10 m long.

The Central Pylon foundation is to be placed on a combined pile cap, with each pile cap being supported by 16 piles (Figure 10). The combined pile cap consists of two separate pile caps connected with two transverse beams. These beams are hollow with dimensions 4 m deep by 11 m wide by 24.2 m long. The piles are cast-in-place concrete piles, 3 m in diameter at the top with an external steel casing, 70-mm-thick. The pile diameter is reduced to 2.7 m after the top 13.5 m and continues for another 28 m. The total length is about 42 m.

The North Pylon is to be placed on one pile cap supported by 16 piles. The pile cap and piles have the same dimensions with those of the Central Pylon foundation. The pile diameter is reduced to 2.7 m down to 6 m into the ground. The total length is about 56 m.

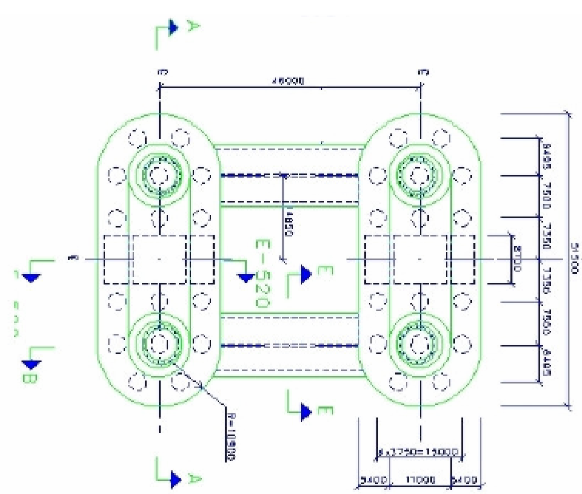

Fig .10. Plan view of Central Pylon foundation (COWI, 2006)

SEISMIC ANALYSES

During the preliminary studies it was obvious that the seismic hazard at the bridge setting is very significant given the past earthquakes in the general area. Seismic hazard studies were carried out during the technical feasibility study and two main seismic sources were identified to influence the bridge location. These were an interplate source corresponding to the subduction zone of the Nazca plate beneath the South American plate (located offshore in the Chilean trench), and an intraplate source, represented by the Gulf of Ancud fault (FGA) which was believed to be an active fault crossing the bridge. The FGA was believed to be responsible for a 6.8-magnitude earthquake associated with an aftershock of the 1960 Valdivia earthquake. In those studies, three earthquake intensity levels were identified relating to different performance objectives of the bridge: A Safety Evaluation Earthquake (SEE-collapse), a Functional Evaluation Earthquake (FEE-service) and a Construction Earthquake (CE), all corresponding to different probabilities of exceedance. The various seismic parameters corresponding to the different events are presented in Table 1.

Table 1: Parameters for the assumed earthquake intensity levels (including the crustal event in the initial studies)

Seismic Source	Magnitude Ms	Depth (km)	Epicentral distance (km)
Subduction (SEE)	8.5	30	80
Crustal (SEE)	7.0	10	22.5
Subduction (FEE)	8.0	30	80
Crustal (FEE)	6.0	10	10
Subduction (CE)	7.5	30	80
Crustal (CE)	(*)	(*)	(*)

(*) The CE-crustal event was negligible.

One of the objectives for CPC early on was to investigate the existence and activity of the FGA. In the initial stages of design, the existence and activity of the FGA were taken as a working hypothesis assuming the worst case scenario, and assigning conservative seismic parameters to the fault due to lack of data. Just before the project was put on hold, the marine seismic reflexion investigation performed by CPC provided evidence that the FGA is not an active fault and its location is probably much deeper than originally thought.

As part of the geotechnical seismic design the designers performed state-of-the-art seismic hazard, soil response and soil-foundation-interaction analyses to determine the design spectra at each foundation location. The rock design spectra for the subduction event were defined at the referential study (ICUATRO-COWI 2001). GDS estimated the acceleration time histories that matched the rock spectra and using wave propagation theory estimated the acceleration time histories at the surface of the shallow foundations. Using SASSI (Lysmer et al., 2000), a finite element computer program, the designer accounted for soil-foundation interaction and estimated the acceleration time histories at the center of the pile caps (central and north pylon) and at the top of the anchor blocks. Figures 11 and 12 show the finite element models of the central pylon foundation and the anchor blocks. Figure 13 shows one of the many design response spectra at the top of the foundation level (central pylon) accounting for soil-foundation interaction. GDS used SASSI to estimate the acceleration time histories, spectra and foundation impedances (springs and dashpots) and provided them to the bridge designer to be used in a global finite element bridge model and identify the seismic forces at the bridge.

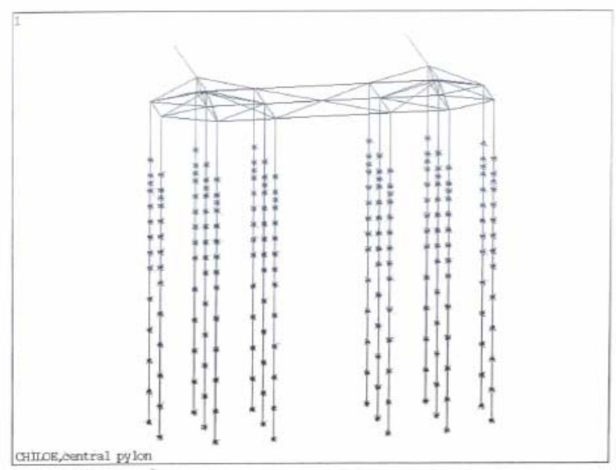

Fig.11. SASSI finite element model of Central Pylon (GDS, 2006)

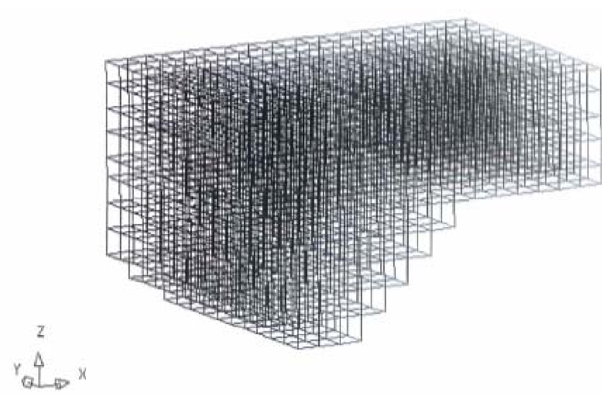

Fig. 12. SASSI finite element model of south anchor block (GDS, 2006)

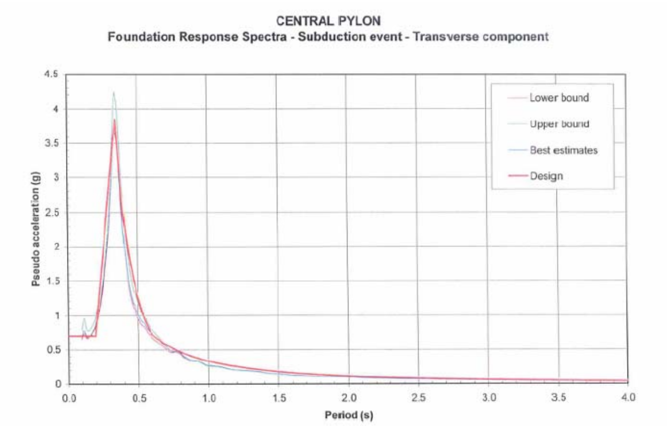

Fig. 13. Indicative design acceleration response spectrum at foundation level of the Central Pylon, accounting for soil-structure interaction (GDS, 2006)

SLOPE STABILITY

<u>North Shore</u>

As mentioned earlier, the north pylon was placed close to a relatively steep slope at the north bank (Figure 14) and the slope stability and slope displacements during the design seismic event were estimated.

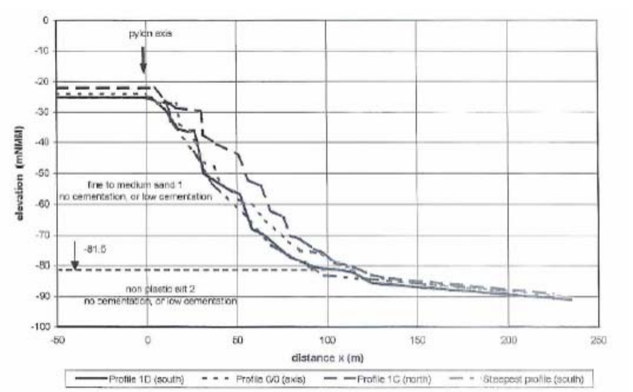

Fig.14. North pylon- longitudinal profiles (GDS, 2006)

Limit equilibrium methods were used to estimate the factor of safety for static conditions. The assumptions and methods used were simple and conservative given that the static slope failure was not a driving factor in the design. This is true for the static and seismic stability as well as the calculation of permanent displacements. A slope of about 35.5 degrees was considered in the calculations.

For the analyses, a ϕ angle of 40 degrees and zero cohesion were used and were deemed reasonably conservative. The overall factor of safety for a slope failure was approximately 1.2. A factor of safety of 1.5 was achieved by moving the north pylon foundation 20 m to the north.

The method of Makdisi and Seed was used to calculate preliminary permanent ground displacements A reasonably conservative peak ground acceleration was chosen (0.3g) assuming a constant acceleration throughout the slope without any out-of-phase motions. The slope displacements were estimated to be of the order of 30 cm.

Post-earthquake static slope stability checks were performed assuming conservative residual soil parameters based on the triaxial results at very large strains. The pore pressure ratio buildup value due to shaking, is taken as 0.2 and is reasonable for this dense, dilative soil. The use of a reduced friction angle of approximately 34 degrees and zero cohesion was probably quite conservative. A minimum factor of safety of 1.15 was found to be reasonable.

From the above analyses, the conclusions where to move the north pylon 25 m to the north towards the shore to ensure a factor of safety of about 1.5 against slope failure for static conditions and 1.15 for post-earthquake conditions.

<u>Roca Remolinos</u>

The Central Pylon is envisioned relatively close (30 to 40 m) to a relatively steep slope at Roca Remolinos (Figure 8 and 15). The lateral capacity of the foundation was estimated conservatively during the design. The stability of the steep slope of Roca Remolinos was not examined in such detail as for the North Pylon mostly because of the generally high cementation of the caprock material. However, during the final design, a more complete study will be carried out to assess the slope stability given that in a case of a slope failure, the cantilevering length of the piles will increase, along with the scour susceptibility of the foundation. Such a situation would worsen in case of repeated lateral loading before or after a potential slope failure. The interconnection of slope stability, large cantilevering lengths and scouring is further discussed later in the "future steps" section.

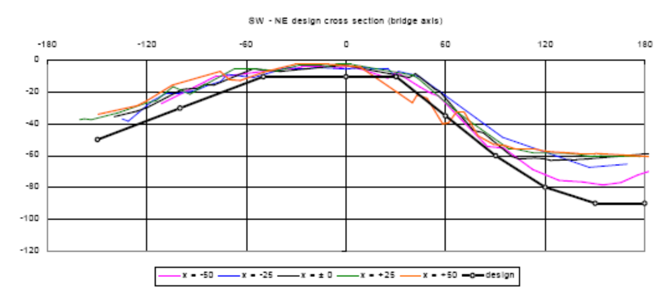

Fig. 15. Cross-sections with the steepest slopes at Roca Remolinos (W. Romberg, 2006)

SCOURING STUDIES

The scouring potential was estimated based on bottom tracking measurements and ADCP (Accoustic Doppler Current Profiler) measurements at the edge of the slope at the north pylon location. Based on the above data and the north pylon soil material information collected during the geotechnical investigation, it was estimated that the rate of material loss the North shore will suffer in the next 100 years, is about 0.2 to 0.5 m per year. Changes in the bathymetries taken in 2000 and 2005 suggest that 0.5 m per year is the more reasonable value to follow. Figure 16 is representative of the strong currents in the channel. These findings suggested moving the North Pylon 50 m to the North. This 50-m displacement due to scour considerations is independent of the 25-m displacement due to the slope stability considerations. However, since both displacements consider extreme events that will not occur simultaneously, a 50-m displacement of the north pylon to the north was decided. This distance would provide adequate factors of safety at all times for slope stability and give the necessary time to repair the scour protection when necessary. Figure 17 shows a recommendation for the scouring protection measures.

Fig.16. Currents at the legs of the jack-up platform at Roca Remolinos

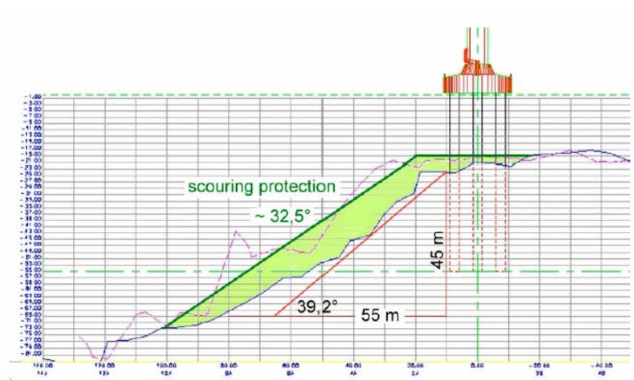

Fig.17. Recommended scouring protection measures for the North Pylon (W. Romberg, 2006)

CONCLUSIONS AND FUTURE STEPS

The complementary investigations and desk studies by the CPC verified that Roca Remolinos is stronger than originally thought during preliminary designs and therefore the design concept of using three pylons is feasible. Several questions still remain to be answered because the project was put on hold before all the necessary work was completed. These questions are provided below:

1. Regarding the erosion processes in the Chacao channel, for the North shore, an erosion rate of 0.2 - 0.5 meters per year was estimated. Based on the geo-morphology of the area, the North shore appears to be much more susceptible to erosion than the South shore and Roca Remolinos, therefore the above mentioned rates should be an upper bound for these locations. To verify these numbers a more detailed assessment of the erosion rate remains to be performed for the South Shore and Roca Remolinos.
2. Regarding the geologic genesis of Roca Remolinos the most plausible scenario for the cementation of the caprock of Roca Remolinos appears to be a pozzolanic reaction involving volcanic ash, sulfur and water. This reaction created the “glue” between the gravels and the sand that compose the caprock. The project was put on hold before the above mentioned scenario was validated by a cementation expert. Similar materials to the caprock may have also existed at the shores, and have probably been eroded. As mentioned in item 1, the rate of erosion of the caprock materials is not yet estimated.
3. The need for a detailed slope stability analysis for Roca Remolinos can not be overemphasized. A possible slope failure would have severely adverse effects on the foundation’s lateral capacity and scour resistance.
4. It is important to assess the behavior of the cap-rock before, during and after lateral-loading events (such as earthquakes, wind, ship collision); this item is connected with the rate of erosion and the cementation of the cap-rock addressed in items 1 and 2, and the slope stability in item 3. The Central and North Pylon are supported on groups of piles which are cantilevering above the ground surface for about 10 and 25 m accordingly. These long lengths will result in increased stresses and strains at the top few meters of the rock and soil (during the lateral-loading events) and will probably break the cementation and result in loosening of the surface material. Subsequently, the strong currents may result in increased scour around the piles, and without proper maintenance there will be a weakened condition at the upper portion of the rock and soil around the piles the next time a lateral loading event occurs. The actual accumulated effect of such loading conditions on the top few meters around the piles needs to be assessed, including the possibility that the total depth of loosened material near the piles may increase with time as a result of the combined lateral loading-scouring sequences. This is a classic case where construction methods and design are tied together and an innovative protection system is required to ensure good

performance of the foundations throughout the project's useful life of the project.

4. Additional investigations at the new position of the North Pylon may be necessary. So far, the information for those locations is taken from the two additional (marine) borings. However, since the envisioned Pylon position changed (moved 50 m to the north based on the slope stability and scouring studies), the marine boring information available is about 50 m to the south of the currently envisioned Pylon location. There is geologic information and on-shore borings at the North shore that could help interpolate properties at the location of the North Pylon. However, there are no shear wave velocity measurements for the Silt layer and this information should be gathered to confirm the current assumptions. The additional borings could be performed during construction to verify the soil conditions.

Answering the above questions is important for finalizing the design for what will be a world-class record-setting signature bridge.

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