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PONTE SULLO STRETTO DI MESSINA

PROGETTO DEFINITIVO

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INDICE

1 Introduction

In the following a short description of the design assumption, design methodology and main calculation results for the dimensioning of the tower foundations of the Messina Strait bridge are presented.

The structural verification has been reported in the following design reports:

- General design principles
- Semi-local FE model description
- Calabria Tower Foundation
- Sicily Tower Foundation

The geotechnical verification has been reported in the following design reports:

- Calabria Tower Foundation: evaluation of foundation behaviour via 3D FE analyses and of bearing capacity
- Sicily Tower Foundation: evaluation of foundation behaviour via 3D FE analyses and of bearing capacity
- Site Stability: Evaluation of liquefaction potential for Sicily and Calabria foundation soil
- Site Stability: slope stability for Calabria tower site
- Seismic analyses for soil-foundation systems

2 Materials

2.1 Environmental exposure classes

The tower foundations are in part in contact with the foundation soil (even if they are rigorously in contact with the diaphragms on the external perimeter and with the improvement treatment of the soil on the basis) and in part are exposed to the atmosphere in an area next to the coast. The environmental exposure classes to be used to design the concrete mix-design are the following:

- Corrosion induced by carbonation : XC4 - Cyclic wet and dry (Concrete surfaces subject to water contact, not within exposure class XC2);
- Corrosion induced by chlorides from sea water: XS3 - Tidal, splash and spray zones (Parts of marine structures).

2.2 Concrete

According to the environmental exposure classes previously defined, for the tower foundation construction, the following strength of concrete are provided:

1 concrete of the cylinder and of the foundation cone, but not the upper 6 m: C30/37

2 concrete of the of the upper 6 m of the foundation cone: C60/75

3 concrete of the transversal connecting beam: C40/50

2.3 Cover

Considering the design life of the structure (200 years), a cover equal to 100 mm with respect to the ordinary reinforcement is used. At the centre of the cover thickness a wire fabric in stainless steel realized with 8 mm diameter bars, spaced 200 mm in both the directions, is provided. The wire fabric is connected to the main reinforcement throughout L-shaped bars placed perpendicular to the wire fabric itself, and also realized in stainless steel of 8 mm diameter.

2.4 Reinforcing steel

For what concerns the steel is provided:

1 ordinary reinforcement: B450C (fyk ≥ 450 MPa);

2 stainless steel: AISI 316 L (fyk ≥ 450 MPa);

3 prestressing steel:

- strands: $f_{p1k} \geq 1670$ MPa, $f_{ptk} \geq 1860$ MPa;
- bars: $f_{p1k} \geq 835$ MPa, $f_{ptk} \geq 1030$ MPa.

The use of stainless steel is provided, as specified in the following:

1 since it has to be guaranteed a design life of the structure equal to 200 years, a cover equal to 100 mm is provided. At the centre of the cover thickness a wire fabric in stainless steel realized with 8 mm diameter bars, spaced 200 mm in both the directions, is provided. The wire fabric is connected to the main reinforcement throughout L-shaped bars placed perpendicular to the wire fabric itself, and also realized in stainless steel of 8 mm diameter;

2 the connecting beam slabs of about 4.0 m in extension starting from the foundation are reinforced with stainless steel bars in order to guarantee a further reserve towards the durability of the structure in case of unexpected large cracks due to differential settlements between the two cones of the foundation greater than the ones predicted in the design phase.

3 Foundation geometry

The pier geometry, both for the Sicily side and from the Calabria side, has been modified respect to the pier dimensions enclosed in the tender design, in order to take into account the point 104.1 of Annex J of EN 1992-2, for which the distance from the edge of the loaded area to the free edge of the concrete section should not be less than 1/6 of the corresponding dimension of the loaded area measured in the same direction. Then, the dimensions of the cone become for the top face 29.60 m, while for the bottom face 39.80 m. In the following two paragraphs the foundations geometry is described, that mainly differs in the base cylinder dimension and in the connecting beam length.

The Sicily foundation has, under the cone, a cylinder with a diameter of 55 m. The cylinder is 13 m high and its bottom face is placed at -15 m a.s.l., while the cone is 20 m high and its upper face is placed at +18 m a.s.l.. The total height of the foundation is thus 33 m. The foundation has two cone-cylinder blocks joined by a transversal connecting beam. The transversal beam is constituted by three cells 11.50 m high and 4.40 m wide, which webs have a thickness of 1.2 m, while its upper and bottom slabs have a thickness of 2 m (see Figure 3.1).

The Calabria foundation presents the same features of the Sicily foundation for what concerns the cone, while differs from Sicily foundation for the cylinder diameter dimensions. In fact, the Calabria foundation has a diameter of 48 m, as it can be seen in Figure 3.2. Also the length of the connecting beam, measured in axis at the top and at the bottom slab changes. The height of the cylinder and of the foundation will remain the same.

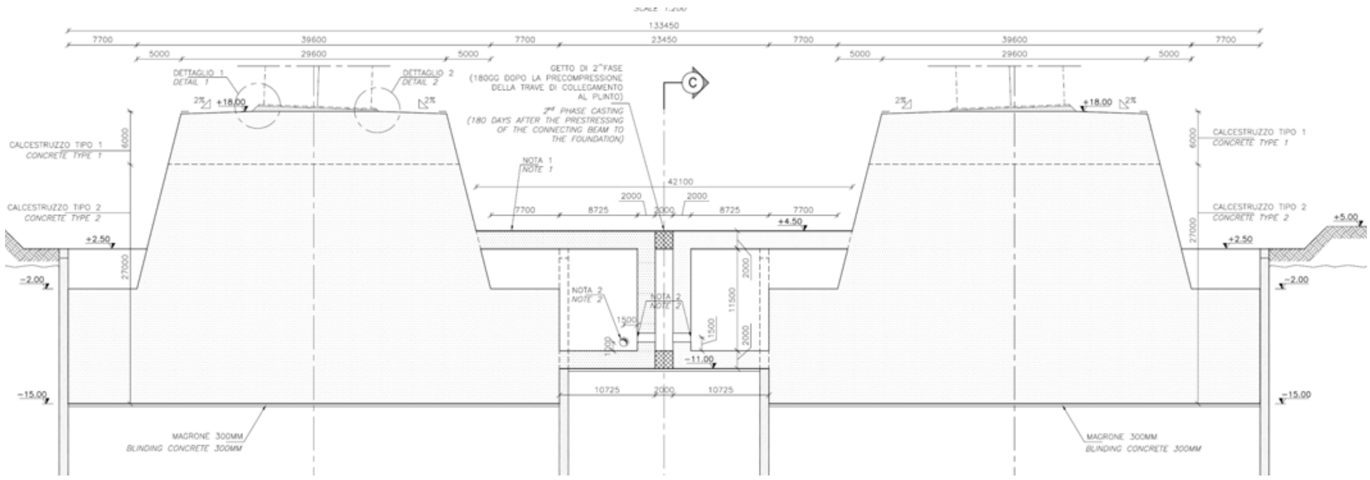

Figure 3.1 Front view of the Sicily foundation

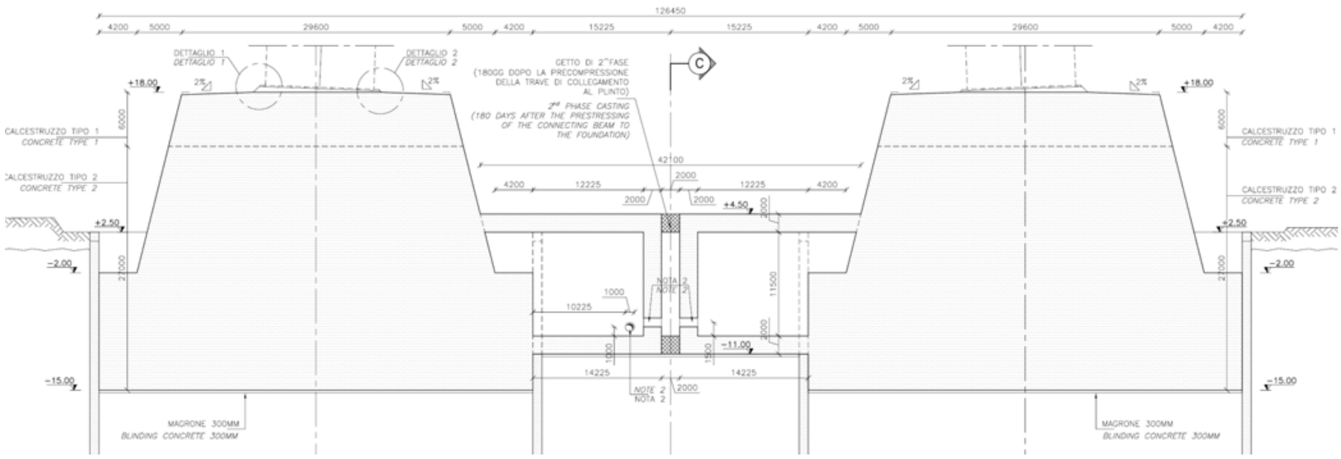

Figure 3.2 Front view of the Calabria foundation

4 Prestressing tendons

The prestressing tendons are used in the foundation construction in two situations:

1 in order to guarantee the transfer of stresses transmitted from the tower to the deepest layers of the foundation plinth. They are vertical prestressing tendons, anchored on the bottom flange plate of the tower with strands number ranging between 12 and 55. The tendons have a U-shaped path with a minimum radius equal to 5 m in the deviation regions. The ducts are realized with a galvanized steel pipe of 8 mm thickness in the deviation regions, while in the other regions are realized with HDPE ducts. The use of galvanized steel pipes is necessary to appropriately transfer the concrete stresses without risk of splitting in concrete. The tendons are comprised of individually greased and polyethylene sheathed strands that provides two barriers against corrosion and allow the replacement of tendons over time. The tendons in the U-shaped ducts are post tensioned from both ends.

2 in order to introduce compression stresses in the connecting beam. They are horizontal prestressing tendons characterized by 37 strands anchored in correspondence of the two transversal walls realized in the central part of the beam. Each tendon is realized with individually greased and polyethylene sheathed strands in order to guarantee the possible substitution of the tendons. Also in this case the tendons show a U-shaped path with a variable radius, function of the path, in the deviation regions; the minimum radius is about 2.70 m. The sheaths are realized with a galvanized steel pipe of 8 mm thickness in the deviation regions, while in the other regions are realized with HDPE ducts. The use of galvanized steel pipes is necessary to appropriately transfer the concrete stresses without risk of splitting in concrete.

5 Construction phases

The construction of each plinth can be summarized in the following steps:

1 foundation diaphragms; soil consolidation under the foundation plate realization; excavation inside the diaphragms until the foundation placing quote;

2 starting of the two plate foundations construction, throughout a single casting of nominal thickness equals to 3000 mm, realized with C30/37 concrete;

3 continuation of the foundation realization throughout sequential casting of nominal thickness of 600 mm, realized with C30/37 concrete. Each casting is reinforced with one or two layers

of reinforcement, that therefore results placed with a 600 mm or 300 mm spacing in the vertical direction and placed directly on the casting previously realized, after the introduction of an appropriate spacer of about 50 mm thickness in order to guarantee an optimum bond to the concrete;

4 foundation cones realization completion through a final cast of 6000 mm realized with C60/75 concrete, because of the contact stresses under the anchorage plate and of the transferring mechanism of the tower foundation actions.

At the end of each sequence casting the superficial scratch of a “donut” of 5 m thickness is provided, in order to improve the shear resistance of the casting joint. The scratch should be achieved by raking, exposing of aggregate or other methods giving an equivalent behavior in order to obtain a surface with at least 3 mm roughness at about 40 mm spacing.

The construction phases of the connecting beam can be summarized in the following stages:

1 demolition of the circumferential diaphragms in correspondence of the intersection area of the beam with the foundation plate, until the basis foundation;

2 construction of the intrados slab of the connecting beam, of the webs and of the extrados slab. In particular, in the construction of the extrados slab, in order to avoid the use of formworks of difficult assembling and removal due to the high height of the intrados slab, the use of nonreturnable formworks realized with pre-cast planks shall be provided. The realization of portholes to access one of the cells of the connecting beam and for the access between the cells shall be provided. The connecting beam is realized with the exclusion of a central segment of 2.0 m length. In this way it’s possible to efficiently prestress the two half-beam to their respectively foundations. The prestressing is realized with prestressing tendons, with centroid coincident with the centroid of the connecting beam and anchored in correspondence of the two transversal walls of 2000 mm thickness;

3 realization of the central segment of the connecting beam after 180 days from the introduction of the prestressing on the two lateral parts and its subsequently prestressing with bars of 120 mm diameter. The number of prestressing bars guarantees the same prestressing force of the unbonded tendons of the first phase. The temporal phase offset of the single parts of the construction, the length of the single parts and the prestressing force, are studied in order to guarantee, once the rheological phenomena (shrinkage, creep,

relaxation) are exhausted and taking into account the redistribution of the actions caused by a change in the static scheme, a residual stress on the connecting beam equal to 1.5 MPa at time infinite.

6 Structural design

From a general point of view, having elements very massive and with a remarkable shape effect, the best way to proceed is to perform an analysis with solid finite elements (3D), operating in linear field and using a resistant plastic criterion for the ultimate limit state and seismic verifications to evaluate the required reinforcement and the corresponding stresses in concrete. Such criterion is based on the application of the static theorem of the plasticity theory, therefore it constitutes a lower bound solution, that certainly rounds down the real evaluation of the available resistances in the foundation blocks. Knowing the required reinforcement in each finite element, the provided one can be defined and the serviceability verification can be carried out.

The accuracy level that is presented in the following pages is surely high, because the 3D elements dimension is on average less than 2 m^3.

The design reinforcement that is proposed should be completed at shop-drawings stage with an accurate study both of the concrete mix design and of the stresses due to the effects of hydration heat, differential shrinkage and creep between different castings with which the foundation will be built.

The structural calculations of the tower foundations have been carried out on the basis of the loads transmitted from the tower to the foundation calculated in the global IBDAS model version 3.1a. The design dominant internal load combinations from the latest global IBDAS model version 3.3f differ less than 10 % from those used for the dimensioning. The increased loads from the tower have been briefly considered and it appears that the new loads could still be accepted within the design shown in the drawings.

6.1 Structural model

The model used in the foundation design should be able to gather in an adequate way the stress flow, starting from the area of the application of the actions, given by the tower legs to the foundation until it reaches the soil. Being the structure very massive, therefore characterized by an important shape effect, it has been meshed using a 3D finite element model (see Figure 6.1).

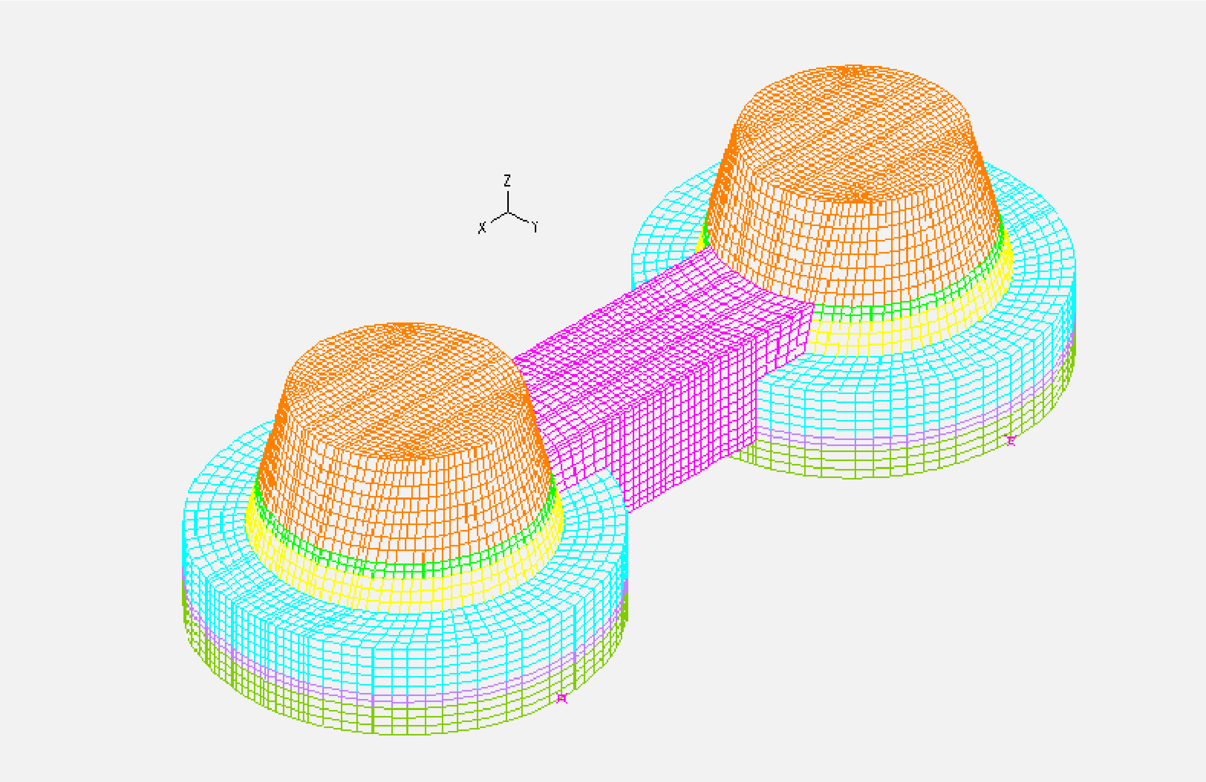

Figure 6.1 Finite element model - 3D view of the Sicily foundation

The structure analysis is carried out using the finite element automatic code ADINA (Automatic Dynamic Incremental Non-linear Analysis) version 7.1. This code, widely used in several fields of the structural engineering, offers great accuracy and reliability of results also for systems with hundreds of thousand of degrees of freedom, both for the linear analysis and for the nonlinear analysis.

Considering the design level (Progetto Definitivo), it has been decided to use a linear-elastic analysis both for the serviceability combinations of actions and for the seismic and ultimate limit state combinations of actions, leaving at more accurate levels of design the eventual refinement and optimization of the structural solution, passing through non linear analysis methods. In the current design level the non linearity in the materials behaviour has been taken into account just in the evaluation of the resistances, as it is described in the following paragraphs.

The model has been loaded with 2 systems of forces. The first system is related to the following actions: loads transmitted by the tower, self weight, seismic accelerations, etc.; the second system of forces corresponds to the soil reaction under the foundation cylinder, and it is able to equilibrate the previous actions. Under such systems of forces, the foundation results self-equilibrated, and therefore it doesn't need, in principle, any restrains. In any case, in order to eliminate the six degrees of freedom corresponding to the rigid body motions, the model has been externally restrained with a system of four fictitious springs in each of the three directions with a very low stiffness compared to the structure one.

Since the soil response is influenced by the stiffness of the connecting boxed beam, the two limit cases of nil stiffness and infinite stiffness for such beam have been considered.

6.2 Action combinations

The foundation dimensioning has been carried out starting from the action coming from the tower and transferred to the foundation itself. In particular, have been considered the combinations of actions able to:

1 produce at the basis of the tower the maximum axial force $F_{z,max}$ (positive if in tension);

2 produce at the basis of the tower the minimum axial force $F_{z,min}$;

3 produce at the basis of the tower the maximum bending moment acting in a perpendicular direction respect to the longitudinal axis of the bridge $M_{x,max}$, taken in absolute value.

To each action coming from the tower have been added the self weight, the seismic acceleration on the foundation mass, the vertical prestressing of the foundation plinth (after losses), the actions given by the prestressing bars used to connect the tower to the foundation (after losses), and the actions given by the connecting beam prestressing, as equivalent horizontal stresses (after losses).

To each of the previously mentioned actions a combination coefficient has been applied, function of the actions transmitted by the tower (for example in case of actions from the tower able to give the maximum axial force and the minimum axial stress, to self weight has been applied a factor respectively of 0.95 and of 1.25).

6.3 Resistance verifications

The resistance verifications are carried out for each finite element in which the model has been meshed, starting from the stress state in the combination analyzed. Considering the three combinations of action described in the previous paragraph ($F_{z,max}$, $F_{z,min}$ and $M_{x,max}$) and the two behaviour hypotheses of the connecting beam (nil stiffness and infinite stiffness), the six combinations used in the dimensioning are obtained.

In the resistance verifications the amount of reinforcement to be provided in correspondence of each finite element is carried out, as well as the maximum stress acting on the concrete, taking also into account its eventual cracking state. The analysis of the results highlights that the provided reinforcement is always greater than the one strictly required, except than 510 elements out of a number of total elements equals to 59656, corresponding to less than the 1% of the elements. In these elements, the provided reinforcement is less than that required mainly because of the type of analysis carried out (linear elastic) and the mesh dimension, not suitable, in some areas of the foundation, to gather the effective distribution of the stresses (in particular in the areas of intersection between the connecting beam and the foundation block). Serviceability verifications

Also the serviceability verifications are carried out for each finite element in which the model has been meshed, starting from the stress state in the combination analyzed.

In particular the stress verification have been carried out checking that the stresses in the reinforcement is limited to a value of $\sigma_{s,\max} = k_3 f_{yk} = 0.8 f_{yk} = 360MPa$ and the stresses in concrete is limited, operating on the safety side, applying a coefficient k_1=0.6 to the maximum stress at which the concrete can work under tangential stresses (νf_{ck}), taking also into account the resistance reduction due to the re-orientating of the compression fields after the cracking.

Despite that cracking verification should be carried out in the quasi-permanent or in frequent combination of actions, in our case the verification has been carried out for the six SLS2 combinations analyzed (situation surely more severe compared to what is required from the EN 1992-1). The cracking verification has been made without direct calculation, using the approach described in section 7.3.3 of EN 1992-1. According to this approach, in order to guarantee a cracking opening less than 0.3 mm (as required for the environmental exposition class XS3), the steel tension, evaluated in the cracked section, is limited to values that are function of the reinforcing bars diameter and of their spacing. In particular, in order to limit the crack openings on the outside surface, in contact with the soil or with the atmosphere, reinforced with bars of 25 mm diameter and 150 mm spaced, the steel stress should be limited to 280 MPa.

The analysis of the results of the serviceability verification highlights that it is necessary to increase the reinforcement in some limited areas of the foundation in order to comply with the limitation previously described.

7 Geotechnical Design

The geotechnical calculations of the tower foundations have been carried out on the basis of the loads transmitted from the tower to the foundation calculated in the global IBDAS model from the tender design. An important update has been carried out on the basis of the new loads provided by the global model IBDAS 3.3 b.

Also the results from the new global IBDAS model version 3.3f have been compared and discussed in separate sections in the geotechnical design reports. It is found that the new loads could still be accepted within the design shown in the drawings.

7.1 Soil conditions

The soil stratigraphy and the geotechnical characterisation are described in the report: “Updated geotechnical characterisation based on the 2010 site and laboratory investigations, Annex” (CG1003-P-RG-D-P-SB-G3-00-00-00-00_01_A) prepared by Professors S. Rampello and A. Flora under an agreement with Eurolink, The report includes the results from the in situ tests planned for the Progetto Definitivo, together with laboratory tests carried out on undisturbed frozen samples and on samples reconstituted at values of relative densities in the range of 40 to 80 %. .

The soil properties and soil stratigraphy from the above report have been used for the geotechnical verification for the Progetto Definitivo.

7.2 Evaluation of foundation behaviour for Calabria tower foundation

It has been verified that the ultimate limit state considering failure of the ground beneath the tower foundation has not been exceeded.

Vertical displacements of the foundation have been calculated by a 3D FEM model. Settlements below the foundation for the structural integrity limit state loading condition defined in the tender documents (SLS2) range between 14 mm and 33 mm, showing some rotation only towards the seashore.

The foundation bearing capacity for loading condition defined by NTC 2008 (ULS-STRU and ULS-GEO) has been calculated both by a 3D FEM model and by a hand calculation using J. Brinch Hansen's formula for shallow foundations. Adequate overall safety factors are deduced by both methods.

7.3 Evaluation of foundation behaviour for Sicily tower foundation

It has been verified that the ultimate limit state considering failure of the ground beneath the tower foundation has not been exceeded.

Vertical displacements of the foundation have been calculated by a 3D FEM model. Settlements below the foundation for the structural integrity limit state loading condition defined in the tender documents (SLS2) range between 72 mm and 98 mm, showing significant rotation only towards the seashore.

The foundation bearing capacity for loading condition defined by NTC 2008 (ULS-STRU and ULS-GEO) has been calculated both by a 3D FEM model and by a hand calculation using J. Brinch Hansen's formula for shallow foundations. Adequate overall safety factors are deduced by both methods.

7.4 Slope stability for Calabria tower site

2D slope stability analyses have been performed for the Calabria shore. The seismic actions considered for the slope stability analyses is based on the seismological study by Faccioli (2004).

The stability analyses were carried out under static and pseudo-static conditions on two longitudinal sections intersecting the Calabria Tower foundations using two different commercial codes, namely Slope/W and Plaxis 9.0. The former code allows to perform limit equilibrium analyses with different stability methods; the latter is a finite element code allowing to analyse the soil stress-strain behaviour through advanced constitutive models.

Under static conditions the lowest values of the safety factor SF were obtained for relatively shallow failure mechanisms located in the middle of the submerged slope, far from the foundation of the Calabria Tower. The calculated values of the safety factor were for the Slope/W analyses in the range 2.13-2.71, and for the PLAXIS analyses around 1.95, the differences in results being due to different geometrical and loading conditions assumed in the analyses as well as to the procedure adopted to evaluate the safety factor.

Pseudo-static analyses performed to consider the seismic action provided, as expected, lower values of the safety factor SF, ranging between 1.23 and 1.37 in the Slope/W analyses, and around 1.0 in the PLAXIS analyses.

Stability analyses were also carried out under both static and pseudo-static conditions to calculate the safety factors (for both the investigated sections) corresponding to slip surfaces passing underneath the Tower foundations, which are the only ones of real concern. The level of safety of these slip surfaces is larger than the previously mentioned minimum one, found for shallow slip surfaces along the submerged slope. The slip surfaces under the Tower foundations are much deeper, because of the beneficial effect of the jet grouted soil mass. Values of the static and pseudo-static safety factors for such mechanisms were about 2.7 and 1.5 respectively, denoting large margins against the achievement of an ultimate limit state of the slope close to the Calabria Tower.

In order to evaluate slope seismic stability conditions according to more rational performance-based criteria, a large number of dynamic displacement analyses were also carried out. In the displacement analyses, consistently with pseudo-static analyses, two different failure mechanisms were examined: mechanism M1 corresponding to a shallow failure surface approximately parallel to the submerged ground surface and entirely developing in the coastal deposit layer; a deep seated circular failure mechanism M2, extending into the continental deposits passing below the bridge tower foundation.

Two sets of accelerograms were used as input motions: 22 records of natural earthquakes and 20 horizontal synthetic accelerograms. The natural accelerograms were used as input motions in their original form and also scaled to a design value of the peak horizontal acceleration equal to 0.58 g.
For mechanism M1, the computed permanent displacements are parallel to the potential sliding surface considered in the analyses, inclined 26^0 to the horizontal. The maximum displacement is in the range 4 - 60 cm, with an expected value of about 14 cm.
For mechanism M2, permanent displacements were computed referring to a rotational failure mechanism; the maximum displacement is in the range 6 - 12 cm, with an expected value of about 2.5 cm.
Further displacement analyses were carried out accounting for seismic induced excess pore water pressures, considering two more seismic records: one relative to the Kocaeli (Turkey) earthquake (1999) and one relative to the New Zealand earthquake (2010) (see par. 6.2 of Report CG1000-P-CL-D-P-ST-F3-TO-00-00-00-01_A_Towers_Liq_pot_ANX)**.** The excess pore pressures in the slope on the Calabria side were evaluated through fully coupled dynamic analyses performed using a finite element code (par. 6.4 of Report CG1000-P-CL-D-P-ST-F3-TO-00-00-00-01_A_Towers_Liq_pot_ANX). Peak values of the maximum seismic induced excess pore pressures were used to evaluate the corresponding reduction of the critical acceleration for the two failure mechanisms assumed in the analyses. Conservatively, the minimum value of the critical acceleration was introduced in the displacement analyses.

For mechanism M1 a large scatter can be observed in the values of maximum displacements that range from 1.6 cm to 263 cm. The latter value was obtained using the New Zealand accelerogram, that induces large excess pore pressures in the coastal deposits. However, this mechanism takes place far away from the tower foundations, and so it is not affecting their behaviour.

For mechanism M2, involving the foundation of the tower, displacements are practically negligible, if natural records are considered. Maximum displacement up to 10 cm are estimated using the New Zealand record scaled to 0.58g. In fact, the excess pore pressures induced by the assumed seismic records affect only the portion of the failure surface developing in the cohesionless coastal deposits.

7.5 Liquefaction potential

An estimate of the susceptibility of the sites potentially sensitive to liquefaction (namely Calabria Tower foundation site, Sicily Tower foundation site and Sicily terminal structures site), have been carried out. The adopted calculation methods used are consistent with the indications given by Italian Code (NTC 2008).

The seismic actions considered for the liquefaction analyses is based on the seismological study by Faccioli (2004).

Calculations using semi-empirical methods for the simplified analyses, based on in situ test results (SPT, CPT, Cross Hole tests) were initially carried out. The results are presented for the different sites, and for the Calabria Tower location a further distinction is made to take into account a difference in density and shear strength between the Southern and Northern part. Average values of N_{SPT}, q_c and V_s have been considered for each site and then compared to the values of CSR related to the twenty reference seismic actions. Synthetic charts of the liquefaction susceptibility were then plotted. Liquefaction potential is low for the Sicily tower site and for the Northern part of the Calabria tower site. In the Southern part of the Calabria Tower foundation site a high potential was evaluated and for the Sicily terminal structures a very high potential was evaluated for most of the 20 seismic actions,.

In addition a complex elasto-plastic constitutive model was adopted to represent soil behaviour, and used in a FEM code for fully coupled analyses. First seismic input motion is described (in this case, the whole accelerogram and not only the maximum acceleration is used) for the two shores. Then, the parametric study carried out to get the calibration of the model parameters is shown, considering both monotonic and cyclic actions. Since the information coming from laboratory tests are meagre at the moment, two sets of parameters were selected.The results of the advanced analyses are presented, in terms of $\Delta u/\sigma'_{m0}$ and permanent displacements of some selected points.

On the Sicilian side, the advanced analyses indicate that, in the worst considered conditions, liquefaction takes place for z < 13m. . The value of the coefficient of permeability has an effect on both consolidation time and on the depth of occurrence of liquefaction; for the lower values of k drainage mainly occurs after the earthquake in the form of post-seismic consolidation. On the Calabria side, the advanced analyses indicate that liquefaction takes place to a larger depth (about 20 m). Due to the very low coefficient of permeability of the Pezzo Conglomerate the consolidation time also increases.