Costa Concordia wreck removal project – Active anchorages for the foundations of the hold back system

Projet d'enlèvement de l'épave du navire Costa Concordia - Ancrages actifs pour les fondations du système de retenue

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ABSTRACT: During the Costa Concordia wreck removal project, a hold back system was designed and successfully installed to prevent uncontrolled sliding of the wreck during winterization and parbuckling operations. The hold back system was composed by steel caissons installed on the seabed connected to the wreck by steel slings and fitted with hydraulic strand jacks to provide the required pre-tensioning to the wiring system. The 8000kN horizontal pulling design force requested by the anchorage system were sustained by the pre-tensioning of the anchor blocks against the rocky seabed by means of ten high capacity steel grouted rods. A critical aspect was the potential progressive failure of the foundation section of the active anchors, that could be induced by the different deformability of steel, grout and rock, further complicated by the cyclic nature of the applied loads. All soil anchors were then designed and installed to guarantee a tensile force applied at the lower end of the anchor in such a way that the resisting foundation section was totally compressed. Three full scale field tension tests confirmed the design assumptions and the eleven caissons installed successfully provided the requested anchorage capacity in operation.

RÉSUMÉ: Dans le cadre du projet d'enlèvement de l'épave du Costa Concordia, un système de retenue pour empêcher le glissement incontrôlé de l'épave pendant les opérations d'hivernage et de redressement a été conçu et installé avec succès. Ce système de retenue était composé de caissons en acier installés sur le fond marin, reliés à la structure de l'épave par des élingues en acier et équipés de vérins hydrauliques pour fournir la précontrainte nécessaire au système de câblage. La force de traction horizontale de 8000kN nécessaire au système d'ancrage a été apportée par la précontrainte des blocs contre le fond marin rocheux, au moyen de dix tiges cimentées en acier de haute capacité. Un point critique était la potentielle défaillance progressive de la section de fondations des ancrages actifs, qui pouvait être induite par les différentes déformabilités du acier, coulis et roche, encore compliquées par la nature cyclique des charges appliquées. Toutes les ancres de sol ont ainsi été conçues et installées pour garantir une traction appliquée à l'extrémité inférieure de l'ancrage, de telle sorte que la section de résistante soit totalement comprimée. Trois tests de tension de champ à grand échelle ont confirmé les hypothèses de conception et les onze caissons installés avec succès ont fourni la capacité d'ancrage demandée en fonctionnement.

KEYWORDS: Costa Concordia, active anchor, pre-tension, grouted rod, rock bolt, seabed anchor, field test, nearshore work.

1 INTRODUCTION

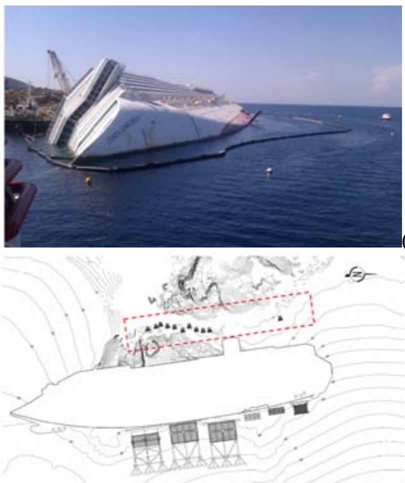
On the night of January 13th, 2012, the cruise vessel Costa Concordia shipwrecked, after hitting a rock close to Giglio island, about 15 km off the coast of Tuscany, Italy. As a result of the incident, the wreck laid with its starboard side resting on the seabed, at a water depth of about 30 m and partially emerging (Figure 1).

The solution chosen for the removal of the Costa Concordia wreck at Giglio Island foresaw first the rotation of the ship to an horizontal position ("parbuckling", see Figure 2), to be followed by its re-floating, after which the ship was tug to Genoa harbour for de-commissioning on July 2014. The parbuckling stage represented the most critical step in the all process and was successfully completed in mid September 2013.

In order to prevent uncontrolled sliding along the steep seabed during parbuckling operations, an holdback system was designed and installed, to provide a sound constraint on the landward side of the wreck.

In addition, some among the anchor blocks were selected to be connected to the wreck in October 2012, well before parbuckling operations, to fasten the wreck during the 2012/2013 winter and the related hard weather conditions.

The hold back system was composed by 11 steel caissons (anchor blocks), connected to the wreck structure by means of steel cables (Figure 3). At the top of each caisson a steel truss structure is mounted, on which large hydraulic winches (strand jacks) are installed that provide the required pre-tensioning to the wiring system.



(a)

(b)

Figure 1. Pictures of Costa Concordia: bow (a) and plan view (b), prior to parbuckling, with the anchor blocks area marked by a red dashed line

Pre-tensioning by means of the strand jacks leads to the application of an initial loading to the anchor blocks, which was subjected to an increase during parbuckling. The pulling action estimated for each anchor block for design was about 8000 kN. Being the external actions and the anchor blocks bases mainly horizontal, a pre-compression against the base rock was needed, in order to provide each block with sufficient shear capacity. The design solution required the installation of 10 steel tendons for each caisson (Figure 4), pre-tensioned to a force of 1750 kN. The anchor block construction sequence was completed with the grouting of the internal chambers of the caisson by means of cement mix. Being the caissons hollow at their bases, direct contact between grout and rocky seabed was established.

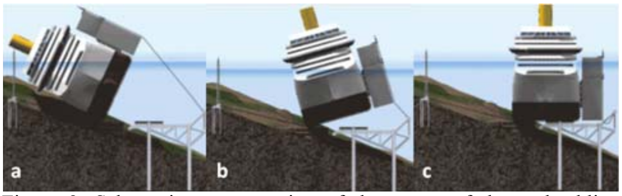


Figure 2. Schematic representation of the stages of the parbuckling project (see www.theparbucklingproject.com), with the hold back system shown on the left-hand side of each picture.

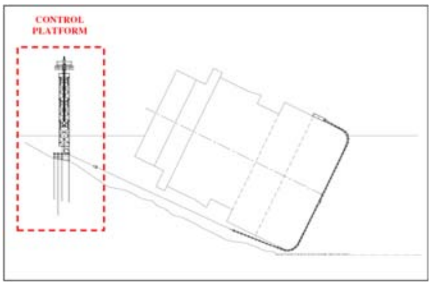
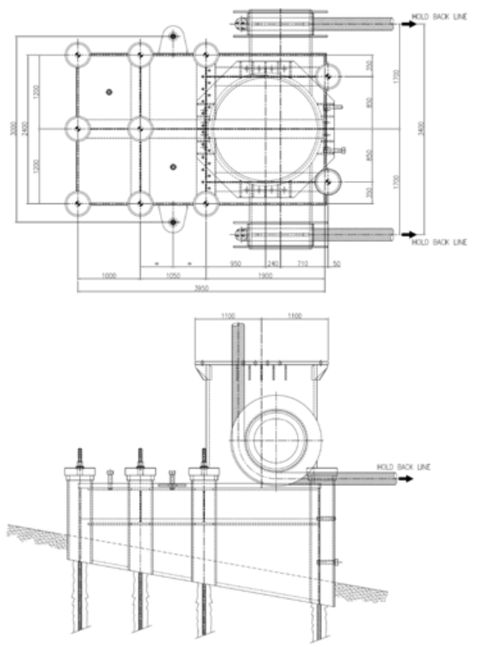


Figure 3. Cross-section of the Costa Concordia wreck, with a typical anchor block and its steel tendons marked by a dashed rectangle



(a)

(b)

Figure 4. Plan (a) and side (b) views of an anchor block

2 DESIGN OF THE ACTIVE TENDONS

As required by the high design values of the pre-stressing actions, thread bars were chosen with a diameter of 63.5 mm. These are manufactured with a steel featuring a yield stress of 670 MPa and a failure stress of 800 MPa.

The seabed within the nearshore areas of the wreck features granite rock outcrop, characterised by limited presence of fractures. RQD values are generally of the order of 80%, while Unconfined Compression tests carried out on a number of cores indicated strengths of 70 MPa, on average, with peak values exceeding 100 MPa, so that bearing capacity of the rock mass under the anchor block was confirmed not being an issue.

Preliminary engineering evaluations of the limit shear resistance available for anchoring applications in this type of material were performed. As summarised by O'Neill et al (1996), it is possible to compare the formulations adopted by several researchers, generalising their relevant empirical relations with the following formulation:

$$\tau_{max} = \alpha \times q_u^{\beta}$$

where τ_{max} is the limit shaft adhesion, q_u is the average UCS (both expressed in MPa) of the weaker materials between rock and grout (i.e. grout, in this case, with assumed UCS equal to 42 MPa), α and β are the empirical factors for the evaluation of the limit shaft adhesion.

The methods considered are listed in Table 1, together with the relevant values of the empirical factors used and the calculated limit shaft adhesions. The method proposed by FHWA (1999) that features a slightly different formulation is also attached at the bottom of the table. Calculated values range from a minimum of 1296 kPa given by the Carter & Kulhawy (1988), Horvath (1982) and Serrano & Olalla (2004) methods, to a maximum of 12600 kPa, given by the Reynolds & Kaderabek (1980) method. For the evaluation of the tentative length of the active section of the anchor, a cautious estimate of 1300 kPa was chosen.

A critical aspect to be faced by the design was the potential progressive failure of the active tendon section of the anchors, that could have been induced by the large difference in deformability (in tension) between the steel reinforcement and the grout, further complicated by the cyclic loading, in particular during the period of hard weather, when sea conditions worsen and higher waves hit the wreck on its portside. This repeated loading is immediately transferred to the anchor blocks and if not properly taken into account, progressive failure of the grout might have lead to a marked reduction of the bonding capacity, by accumulating before the start of the parbuckling stage. A solution was found by transferring the axial load from the steel tendon to the region of the active bulb farther from the seabed (Figure 5). A set composed by two thick steel plates 1 m spaced is rigidly connected to the lower end of the tendon and grouted for a firm connection to the surrounding rock mass. On the other hand, a lubricated sheath isolates the tendon from the injected grout everywhere, but within the plates section. No contact and as consequence no shear stresses are thus transferred directly from the tendon length to the surrounding grout along most of the bar length. Nevertheless, the entire grouted length of the active section contributes to the re-distribution of the axial force to the rock mass. This scheme results in a compressed active section, rather than the traditional tieback active section subjected to tensile loads. Besides, a beneficial Poisson's effect is also developed, as the tendency to lateral expansion of the compressed plug is counterbalanced by the stiff confinement that granite rock can provide. As a result, additional normal stresses are produced, which have a positive contribution to the overall shear resistance of the element.

The length of the drillhole is variable from 12 to 15 m, according to the anchor location within the anchor block layout, in order to transfer the pre-stressing actions to a larger volume and prevent the possibility of failure at depth, while its diameter was set equal to 203 mm.

Table 1. Summary of the results obtained from the preliminary evaluation of the limit shaft adhesion

METHOD	α (-)	β (-)	τ_{max} (kPa)
Carter & Kulhawy (1988)	0.2	0.5	1296
Coates (1967)	0.05	1	2100
Gupton & Logan (1984)	0.2	1	8400
Horvath & Kenney (1979)	0.21	0.5	1361
Horvath (1982)	0.2	0.5	1296
Meigh & Wolski (1979)	0.22	0.6	2072
Reese & O'Neill (1988)	0.15	1	6300
Reynolds & Kaderabek (1980)	0.3	1	12600
Rosenberg & Journeaux (1976)	0.34	0.51	2287
Rowe & Armitage (1984)	0.4	0.57	3368
Serrano & Olalla (2004)	0.2	0.5	1296
Toh et al. (1989)	0.25	1	10500
Williams et al. (1980)	0.44	0.36	1690
FHWA (1999)	$\tau_{max} = 0.65 p_a \cdot \left(q_u / p_a \right)^{0.5}$	with: p_a = atmospheric pressure	1332

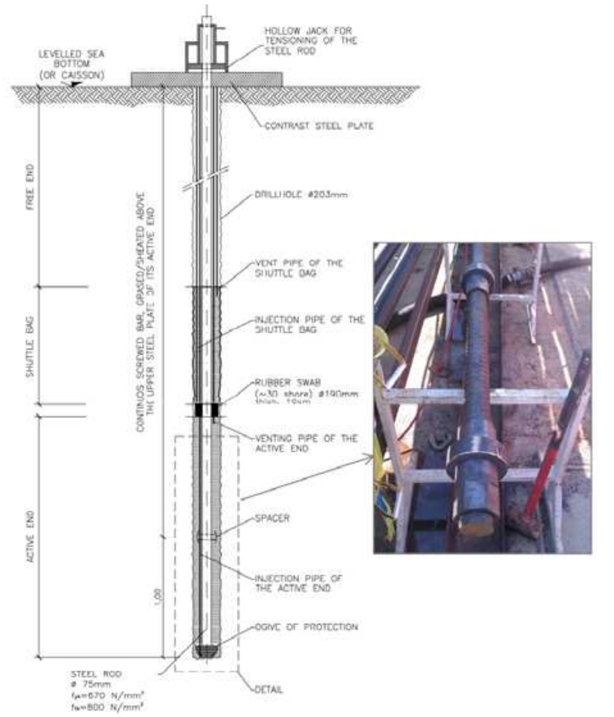


Figure 5. Details of the steel test tendon and of the steel plates for the connection between the unsheathed section and the injected grout

3 PROGRAM OF PRELIMINARY TESTS

With the scope to validate the initial design assumption on the value of the limit shear stress available at the contact between the cement grout and the rock mass and to test the transfer of the axial force to the grouted volume by means of the steel plates set, a series of three load tests was carried out (TDP1, TDP2, TDP3). TDP1 and TDP2 had the scope to bring to failure conditions the contact between the grout and the rock mass, so that an indication on the real Factor of Safety against pull-out failure could be obtained. The following steps were carried out (Albert et al. 2015; Asioli and Del Monte 2015):

- the bar was tensioned to an “alignment” load of 286 kN;
- the load was then increased at steps of 286 kN and the correspondent initial and deferred (60 seconds) elongations measured, to a maximum force equal to 2668 kN;
- the maximum force was decreased from 2668 kN to 286 kN in three steps and the correspondent initial and deferred (60 seconds) elongations measured.

Three of the above cycles were applied for TDP1 and two for TDP2.

The third test (TDP3) was carried out according to a more complex loading procedure, with the scope to evaluate any viscous response of the anchor system:

- the first load cycle was carried out similarly to the test procedure for TDP1 and TDP2, but assuming lower alignment load (175 kN) and peak load (2096 kN);
- starting from 175 kN, the tension load was increased step by step up to reach the maximum applied load of 1747 kN and deferred elongations measured (till 24 hours), followed by detensioning to 175 kN;
- the load was then increased up to the maximum load of 2668 kN and the deferred elongations measured (till 30 minutes), followed by final detensioning to 175 kN.

The third cycle was to be repeated five times. However, due to adverse sea conditions, the last load cycle was not executed. Due to the high values of the loads applied during these tests, much greater than expected in service conditions, larger steel bars were adopted (outer diameter of 75 mm), which allow for a yield load of 2960 kN. The length of the active section was assigned equal to 3 m (TDP1 and TDP2) and to 5 m (TDP3).

Drilling was performed by means of a water hammer, driven by a water pump able to provide a constant hydraulic pressure of 180 bars. Grouting of the anchor volume located below the shuttle-bag was carried out at 1 MPa pressure. Care was taken in this stage to insert right below the shuttle bag a compressible rubber disc, with the scope to leave an empty chamber, so excluding any contribution of the shuttle-bag to the pull-out resistance. An hydraulic jack was used for the application of the load (Figure 6). The type of pressure unit adopted was able to maintain a chosen steady pressure value when needed also throughout the long duration load steps, such as occurred for the TDP3 test.

Figure 6. A picture of the test set-up, clearly showing the hydraulic jack

The results obtained from tests TDP1 and TDP2 went beyond expectations, as no failure was reached even for the maximum force applied. Load-displacement diagrams are shown in Figure 7. The average value of the shear stress τ induced along the grouted section of is calculated as:

$$\tau = \frac{F_{max}}{\pi \cdot D_{drillhole} \cdot L_{grouted}} = \frac{2668\ kN}{\pi \cdot 0.203\ m \cdot 2.9\ m} = 1443\ kN/m^2$$

where F_{max} is the maximum pull-out load applied to the steel tendon, $D_{drillhole}$ is the nominal diameter of the drillhole and $L_{grouted}$ is the contact length between steel tendon and grout (net length of the active end). The above value (1443 kPa) is larger than assumed during the preliminary design stage (1300 kPa), so that the preliminary design was validated and confirmed.

In the third test (TDP3), no viscous response was observed, which otherwise might have jeopardised the duration of the anchor blocks pre-stressing against the granite seabed (Figure 8). Similar conclusions as previously stated for TDP1/TDP2 can be drawn about the first load cycle and the corresponding permanent settlements measured, which recorded a slight increase following the load peak increment from 2096 kN (first cycle) to 2668 kN (third cycle).

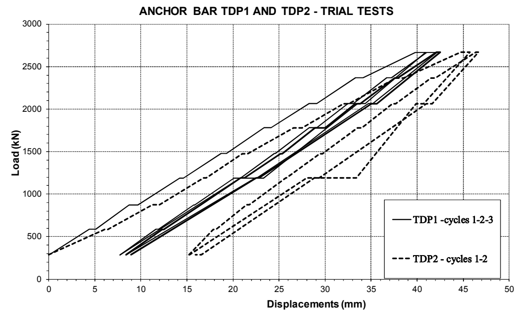


Figure 7. Load-displacement diagram for tests TDP1 and TDP2

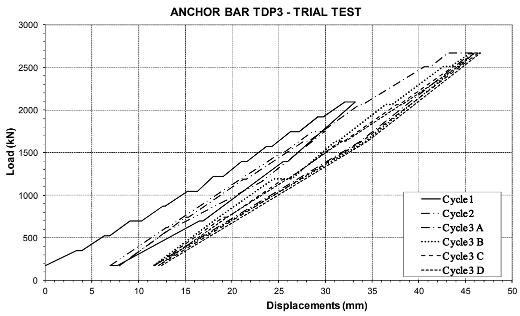


Figure 8. Load-displacement diagram for test TDP3

4 INSTALLATION OF THE SERVICE TENDONS

One-hundred and ten anchors were installed as part of the following procedure, that summarises the construction steps of an anchor block system:

- site preparation of the seabed at the anchor block location;
- deployment of the steel caisson onto the levelled seabed;
- first stage filling of the internal volume of the steel caisson with grout, to about half its height;
- installation of the ten steel tendons, following the same procedure previously listed about the preliminary test tendons;
- hand-tightening of the reaction nuts;
- second stage filling of the internal volume of the steel caisson with grout, to its complete filling;
- test of the ten tendons, with the load test 10% higher than the maximum design tension, followed by detensioning to their design pre-stressing value.

A time gap of one week was left in between the second stage filling and the tensioning stage. This interval was found sufficient from laboratory tests for the development of the minimum UCS of the grout required by design (42 MPa).

For the tensioning of the tendons a different type of jack was used, composed by a power unit connected to ten jacks. This system was able to tension simultaneously the ten tendons, avoiding any out of balance effect on the anchor block.

All tendons were thus brought in eleven steps to a maximum axial load of 1925 kN each, which was maintained for ten minutes (measuring the related deformations), and subsequently unloaded to their final service pre-stressing value of 1750 kN. All tests were successful, as all tendons were able to carry the loads applied and no unacceptable large elongations were recorded.

5 CONCLUSIONS

As part of the Costa Concordia wreck removal project, a hold back system was designed and installed, with the main function to prevent uncontrolled sliding of the ship along the steep seabed during storm and "parbuckling" operations. In order to provide the required shear capacity, an adequate pre-stressing of the caissons against the granite seabed was engineered. This was accomplished by means of a group of ten thread steel bars for caisson, each pre-tensioned to a design value of about 1800 kN. Anchors design was done in such a way as to keep their active section always in compression and was validated by means of a preliminary programme of pulling tests on single tendons, which confirmed the initial design assumptions and in particular the limit value of the shaft adhesion. Besides, all service tendons, for a total number of more than one hundred, were successfully tested and put into operation.

6 ACKNOWLEDGEMENTS

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