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FOUNDATION REINFORCEMENT OF THE IPANEMA BEACH OFFSHORE SEWAGE PIPELINE

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ABSTRACT

The Ipanema Beach offshore sewage pipeline was installed thirty years ago in water depths up to 31m, in Rio de Janeiro, Brazil. It has a unique design, because steel piles were driven every 41 to 50m at joint along its length, in order to support it, keeping a constant gradient flow. These piles maintain the pipeline above mudline. After only a few years a first of these pile supports collapsed, allowing the pipeline to drop on the seabed, because of the occurrence of micro-cracks triggered by corrosion and environmental loads. Similar failures took place again before corrective measures could be taken. The main challenge faced at the end of 1999 was to solve the problem, thus avoiding any major consequence to the environment in the vicinity of the City of Rio de Janeiro, without having to interrupt the sewage pumping. This challenge was met by designing a non-conventional foundation reinforcement. In December 2001 the final support was reinforced. It is estimated that the entire life of the structure was increased by at least 30 years.

INTRODUCTION

The sewage collected from the southern districts of Rio de Janeiro City, flows through a 4.3km long offshore pipeline. The sewage is launched at 31m water depth offshore the Ipanema Beach. In order to survive the breaking wave zone the initial part of the pipeline is buried, but beginning a few hundred meters from the beach the pipeline is supported by discrete 1.4m o.d. by 3/8" wall thickness steel open-ended pipe piles. A total number of eighty piles were driven up to 25m penetration in heterogeneous soil deposits that vary from soft clay to dense sand, keeping the pipeline up to 4m above the seabed. A split concrete double joint was used to make the connection between the concrete pipes and the foundation piles as shown in fig. 1.

After twenty years in use one of these piles broke, dropping both adjacent tubes onto the seabed. This same problem occurred again on three additional supports. Fortunately for the environment the pipeline never opened, thanks to the flexibility of the rubber O-ring connection and neither was there any damage to any of the concrete parts, so the steel piles were consistently identified as the weak part of the system. It was recognized, however, that a major environment hazard would be the result of such an accident in which the pipeline did open, so future accidents had to be avoided at any cost. In order to assure this, Rio de Janeiro's State Water Supply and Sewage Agency (CEDAE) contracted, in addition to the repairs of the fallen supports, the design of an overall foundation strengthening for the remained supports. A new soil investigation campaign and a geophysical survey (fig. 2)

were carried out on which to base a design concept using suction piles.

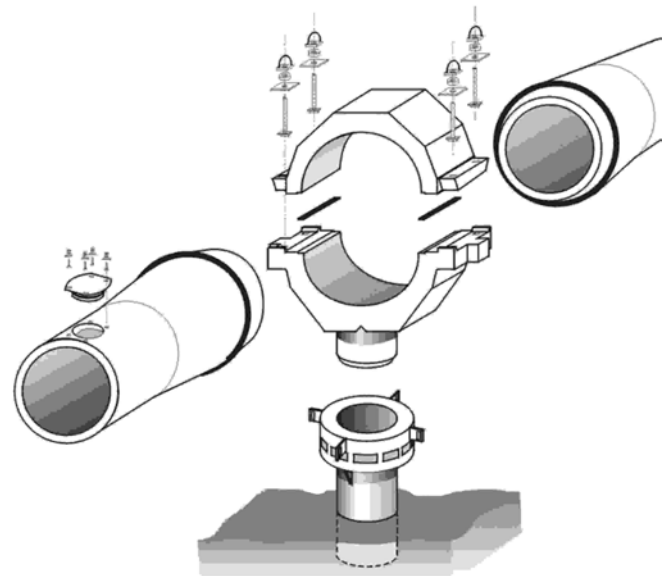


Fig. 1. Double Split Concrete Joint Configuration

While the bid to define the job contractor was in course, another support (# 505) broke. At this opportunity, however, a reasonable volume of sewage spilled on the beach and an emergency repair solution, using driven piles, was carried out. At this occasion three other supports, the adjacent #504 and 506, and a third whose stability was questioned because of its large tilt were also lined up for immediate repair.

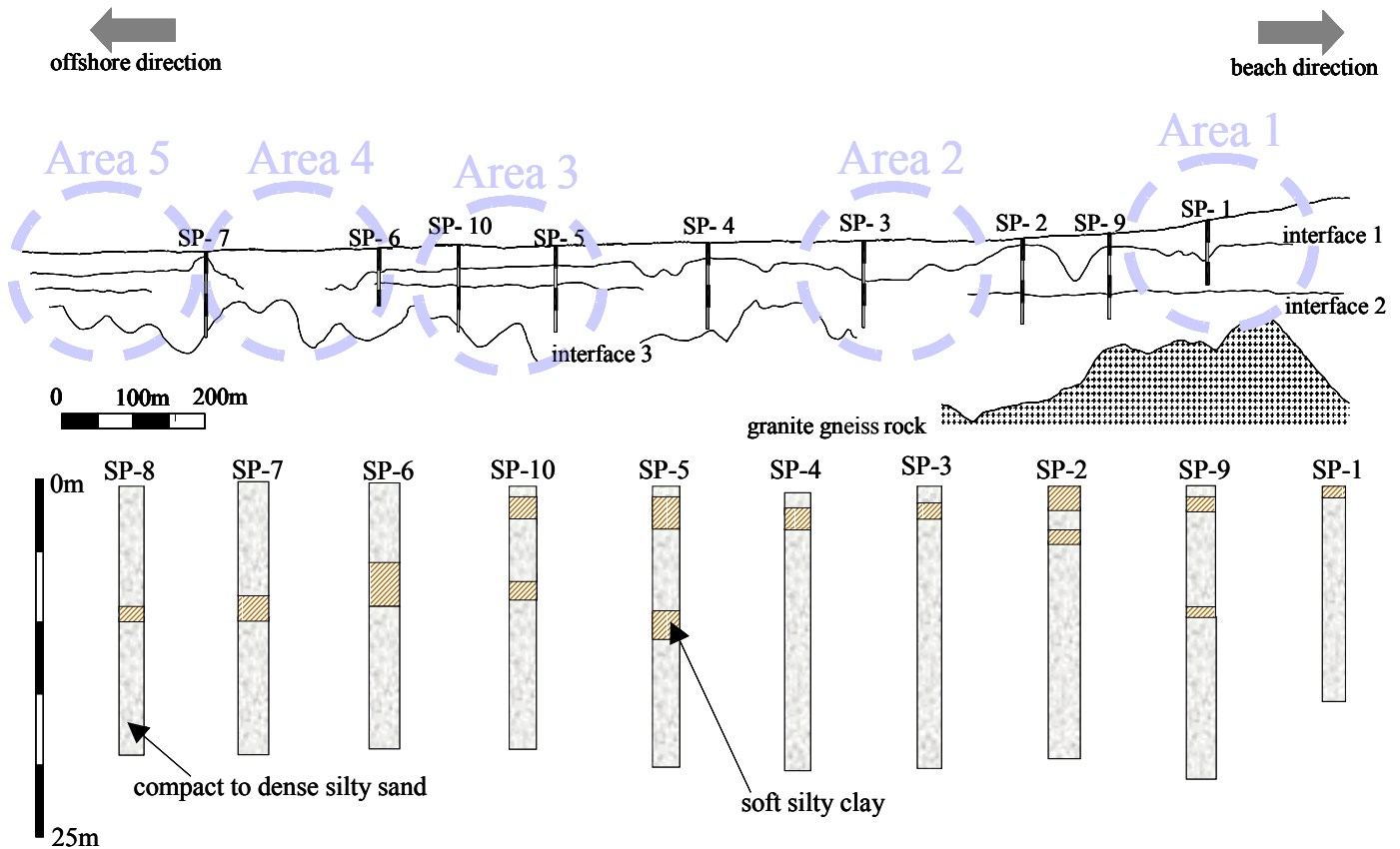


Fig. 2. Stratigraphy Identified in the Geotechnical Boreholes and in the Seismic Survey

When the piles of these repairs were driven, it was verified that the soil boreholes in the locations were not representative. These piles crossed stratigraphies totally different from those presented in the borehole logs. With this updated information the designer and CEDAE chose to change the reinforcement concept to driven piles, a more flexible solution than that posed by the suction piles, which require more refined and reliable data. Less than two years after the installation was started all planned supports had been strengthened.

This paper presents the design solutions, as well as an analysis comparing the piling records with the stratigraphy obtained from the geotechnical and geophysical data.

THE SUCTION PILE DESIGN

For economical reasons the first strengthening concept was based on a single structural solution for all geotechnical profiles identified by the boreholes. The concept included two 4m o.d. steel caissons with 2cm wall thickness and 4m penetration, centered 10m apart, each one weighing 13 tons. In addition to the piles a 20 ton steel structure was designed to transfer the loads from the pipeline joint to the caissons. This assembly was designed to resist vertical loads up to 145 tons and lateral loads as high as 45 tons. The main beams (girders) of the load transfer structure were 13m long and 1.1m high. They were supported on the caissons in such a way to allow

rotations, but restricting horizontal motions. The connection with the existing joint is provided by a bolted brace, half of which was connected to each of the beams (1.9m diameter). Norwegian Geotechnical Institute (NGI) was hired by the designer to validate the solution.

Figs. 3a to 3d present, step by step the installation phases. The total installation was estimated to be done in less than four working days per support. The installation sequence included first lifting the installation frame fully loaded with the suction caissons and the load transfer beams (fig. 3a). In order to make the solution possible, in this step the caissons needed to penetrate the superficial soil by self-weight. After the frame was adequately positioned above the pipeline joint, divers would close the caisson valves to start the driving procedure. For the driving of the caissons up to the maximum penetration, two centrifugal pumps were sized in order to discharge all confined water trapped inside the caisson chambers (fig. 3b). When the caissons reached the maximum penetration the pumps would be turned off, the installation frame would be removed and the load transfer beams locked into the support joint, one in front of the original pile and the other behind it (fig. 3c). The brace would then close around the original pile and bolts installed and tightened.

To transfer the pipeline loads to the reinforced structure the centrifugal pumps would then be reversed in order to fill up the caisson chambers with pressurized water.

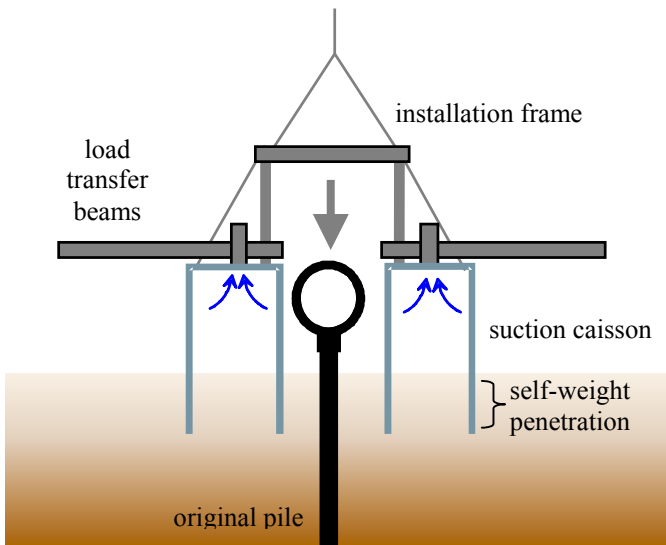


Fig. 3a - Suction Caisson Self-Weight Penetration Phase

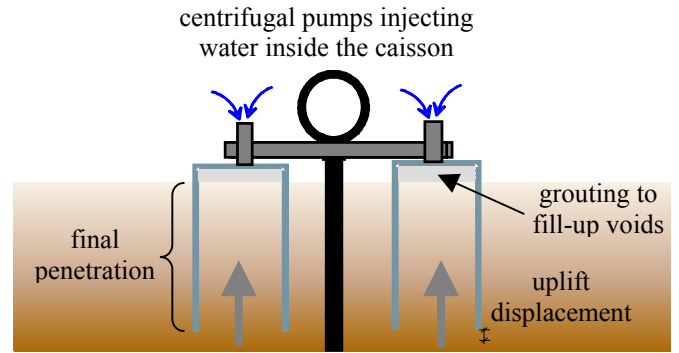


Fig. 3d - Lifting and Grouting Phase

This would cause the caissons to be lifted, thus assuming at least part of the support load (fig. 3d). To prevent any future settlement of the caissons, the gap formed between the caisson top plate and the soil plug will be totally filled with cement grouting.

EMERGENCY REPAIR

Immediately after the winning bidder for the suction project was declared, but just before the corresponding service order for the contractor was issued, support #505 (located only 900m away from the beach) in 23m water depth, collapsed. Immediate repair was required, but the urgency to start construction of the proposed strengthening solution made it clear that an alternative design using the available material would have to be undertaken, even if this meant doing so at a higher cost.

Less than a week after the accident a crane barge was mobilized to the offshore site carrying 30" o.d. open-ended pipe piles to be driven. Although the vessel had been mobilized to strengthen support #505, it was decided that three other supports should be strengthened first: support #606 (in the sewage discharge zone), because it had already tilted approximately 7° and was thus considered to be failing and supports #504 and #506, adjacent to #505, to prevent a domino effect type failure during the repair of support #505.

It was decided that the installation procedure of the support #505 strengthening would be based on the following phases:

- first a four piled strengthening structure (with grouted sleeves), used also for the supports #504 and 506, would be installed close to the former position of the #505 pile;
- a spare base of the concrete joint (called a "chevretre") would be mounted on top of this structure;
- on a day with ideal sea conditions, the damaged joint would be opened and the two 50m concrete pipes lifted from the mudline;
- before transferring the pipes to their new support the old 10m long rubber O-rings would be replaced by new ones;
- with both pipes placed on the new "chevretre", the top part of the concrete joint ("chapeau") would be recovered from the mudline to close the new joint.

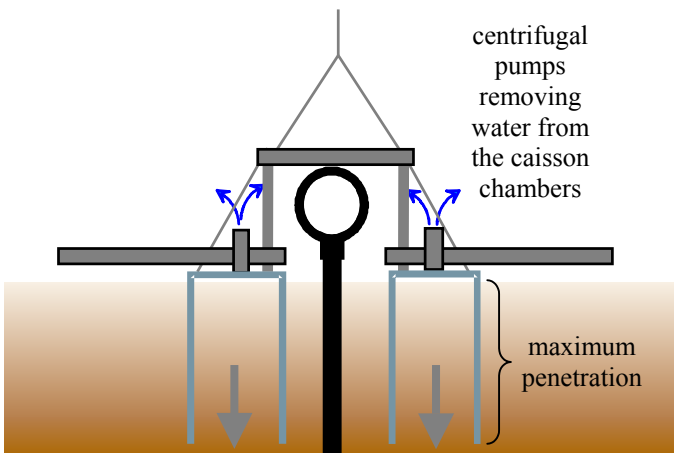


Fig. 3b - Suction Phase

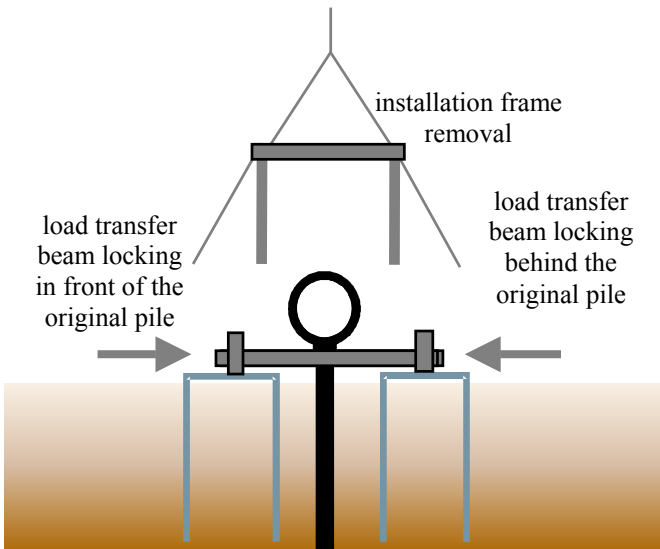


Fig. 3c - Locking Phase

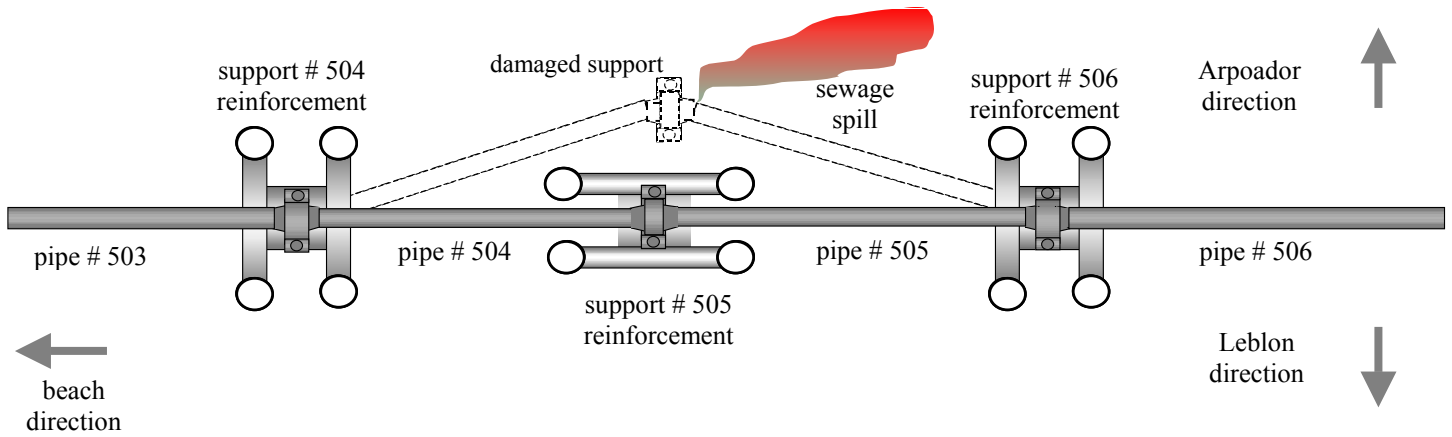


Fig. 4 - Support #505 Area Before and After Repair

Figs. 4a and 4b show a sketch of the support #505 area before and after the intervention.

Due to the summer storms, the large barge motions, the reduced pipe connection clearance (the gap between adjacent segments inside the pipe joint was only in the order of 20 to 50cm) and finally, because the repair had to be performed in less than 24 hours (the limiting time to accumulate sewage in the onshore storage tanks), it was decided that the job had to be carried out with the aid of a fixed structure. A 20 ton steel jacket, fixed to the mudline by four 30" driven piles, was designed, fabricated and installed specially for this job in less than one month. To control the maneuvering of the two 50m concrete pipes on their way to the new support, an 80 ton capacity constant tension winch was mounted at the top of the jacket.

All repairs were done in less than four months. At all four support locations it was observed that the stratigraphy evaluated from the geotechnical boreholes did not match the pile driving records. This observation triggered some doubt regarding the risk involved with the use of the suction pile concept as the general repair solution.

IMPROVING THE DRIVEN PILE SOLUTION

Although good engineering is the result of carefully planned design, based on solid technical judgment, it is also a fact that one learns with experience. This is definitely the case, considering the rush in which the emergency project mentioned above had to be conceived. The following aspects seemed to be of utmost importance and would have to be addressed if the authors were to improve their previous design:

- the two halves of the strengthening structure installed at supports #504 and 506 should be able to remain in the upright position during the interconnection to the original pile. The fact is that they fell over very easily, making this step a time consuming item;
- the 45 tons vertical load and 3 tons horizontal load transferred to the four piles were low, but the 30" o.d. by 1" wall thickness piles could not be reduced, because this

- diameter was governed by geometric limitations, high stress developed during installation and also due to the bond strength of the grouted connection used to transfer the load from the sleeves to the piles;
- the bolted connection of the two parts of the reinforcement structure were difficult for the divers to adjust, which also turned out to be time consuming.

Based on the items pointed out an improved 19 ton steel structure with only three open-ended piles was conceived (fig. 5). The advantage of this concept is that it could be set directly into position without tipping over. In addition, instead of connecting two heavy halves it required just a small 1 ton add-on piece used to brace the original pile, thus making the bolting operation much easier for the divers, besides weighing considerably less than in the emergency design.

The contractor used a 2000 ton jack-up platform to install all sixty-seven reinforcements. The planning was prepared to execute two reinforcements per jack-up position. To make this possible the jack-up operated centered above the pipeline. As shown in figs. 6a and 6b, two frameworks were positioned, one on each side of the platform, together with two lattice



Fig. 5 - Reinforcement Structure

boom cranes, which could maneuver on the main deck in order to perform the installation together.

Delmag D-44 diesel hammers drove the two hundred and one piles to penetrations ranging from 7m to 28m below the mudline.

The installation procedure started with the structure carefully inserted under the pipeline joint until the open brace moved into position against the original pile (fig. 7a). The structure add-on was then lowered, put into position and bolted to the main part. At this time the foundation strengthening was sitting on the seabed. Guided by the fixed template placed on the jack-up steel framework, the piles were inserted into the structure sleeves and driven straight down to final penetration (fig. 7b). Once all piles were at final penetration the reinforcement structure could be lifted until it reached the pipeline bottom of the joint (fig. 7c). Steel wires kept the structure in position for the cement grouting of the annular between the pile and the sleeve, thus fixing the new foundation to the reinforcement structure (fig. 7d).

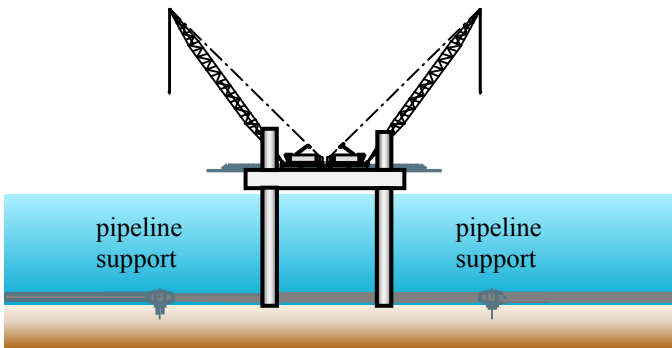


Fig.6a - General View of the Jack-up Positioning

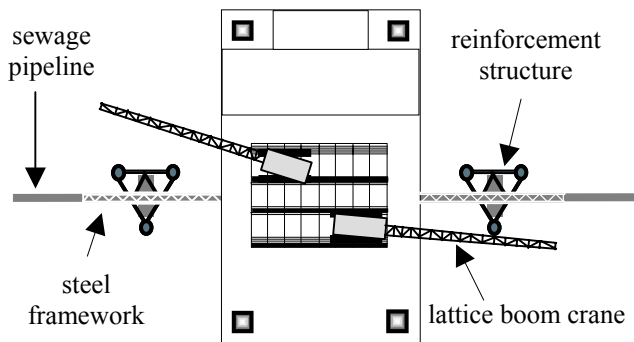


Fig.6b - Plant View of the Jack-up Positioning

During the installation process the erratic nature of the soil deposits along the pipeline axis was confirmed. Generally speaking it was possible to verify five areas of different pile driving behavior:

- Area 1, the closest to the shore (approximately 500m), around 20m water depth, where after a 1m self-weight penetration the piles were driven in compact to very dense sand layers, with a blowcount linearly increasing to around

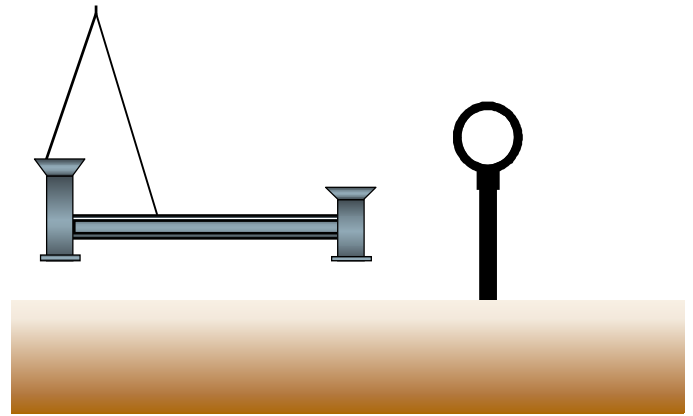


Fig. 7a - Positioning Phase

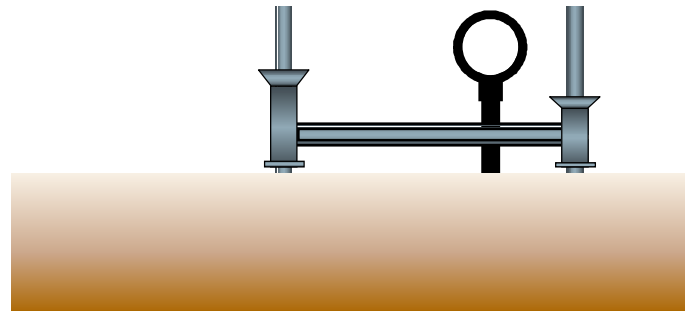


Fig. 7b - Pile Driving Phase

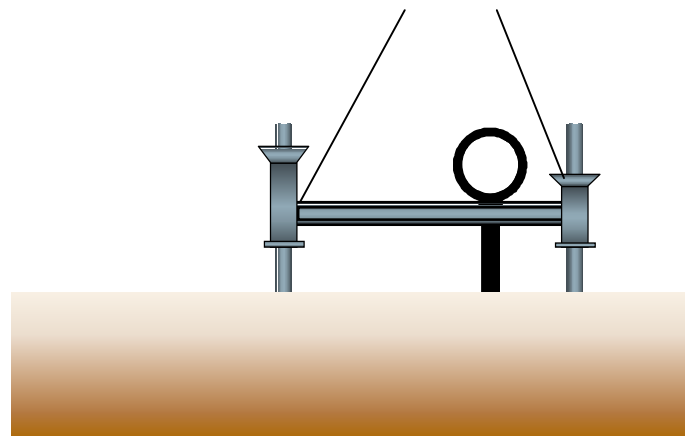


Fig. 7c - Lifting Phase

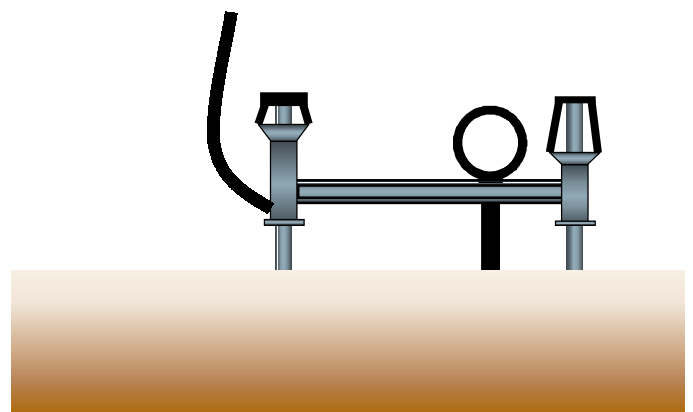


Fig. 7d - Grouting Phase

100 blows per 25cm from 4m penetration to the final penetration of 8-10m below the mudline. The seismic survey identified two intermediate interfaces (the shallowest at 5 to 10m and the other at 15m) and a granite-gneiss rock at less than 30m from the seabed;

- Area 2, at 26m water depth, is approximately 1,200m away from the beach. The piles were driven up to 20-22m penetration below the seabed reaching 20 to 40 blows per 25cm. They penetrated practically due to self-weight for up to 10m (possibly in very soft to soft clay soils). From 10 to 14m a compact sand deposit was found. According to the geophysics, as the distance from the shore increases the rock deepens and along a 300m extension the second interface even vanishes from the records (the first interface varies between 3 to 10m);
- Area 3, around 29m water depth, approximately 2,200m from the shore. In that location the piles were shorter than in the other parts. After a 2m self-weight penetration, between 4 to 8m penetration the blowcounts remained constant at around 30-40 blows per 25cm. Suddenly they increased to approximately 80 to 110 blows per 25cm at 8-9m penetration below the seabed. The soil profile below 2m seems to be a compact to very dense sand. The seismic survey identifies three interfaces, at 5m, 10m and below 20m (this deeper interface possibly represents the same rock identified in Area 1);
- Area 4, around 29m water depth too, approximately 2,800m away from the beach. This is the area where the presence of soft clay deposits becomes more intense. Self-weight penetrations of up to 12-13m were observed. After that the blowcounts increased to as much as 50 blows per 25cm at 15-18m penetration below the mudline (possibly in compact to dense sand). The geophysics showed just the third interface, varying between 12-25m (the two shallower interfaces disappeared along a length of 200-400m);
- Area 5 is the pipeline diffuser zone. This is the most distant area from the shore, at 31m water depth. Pile penetrations varied from 18 to 21m below the seabed. No self-weight penetration was verified. The driving possibly happened entirely through sandy layers. In the superficial layer the blowcount reached 50 blows per 25cm (dense to very dense deposit), but at 6-7m it decreased to the level of 10-20 blows per 25cm (medium compact to compact granular deposit). As penetration increased so did also the blowcount reaching 40-70 blows per 25cm at final penetration. The seismic survey identified the three interfaces at 5m, 10m and between 15 and 25m.

All cited areas are identified in fig. 2. Figs. 8a to 8e shown records of piles driven in each one of these areas (supports #405, 515, 539, 552 and 607, respectively).

CLOSING REMARKS

Two automated procedures for the repair of the piled foundation of the Ipanema Beach offshore sewage pipeline have been outlined, one based on a non-conventional piling technique using the suction pile concept and the other a

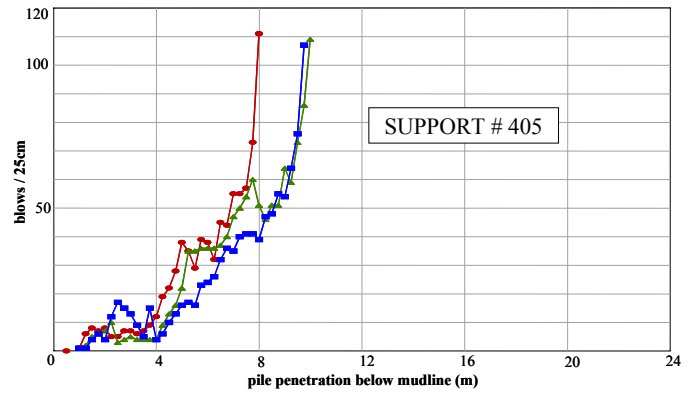


Fig. 8a - Typical Blowcount Records of Area 1

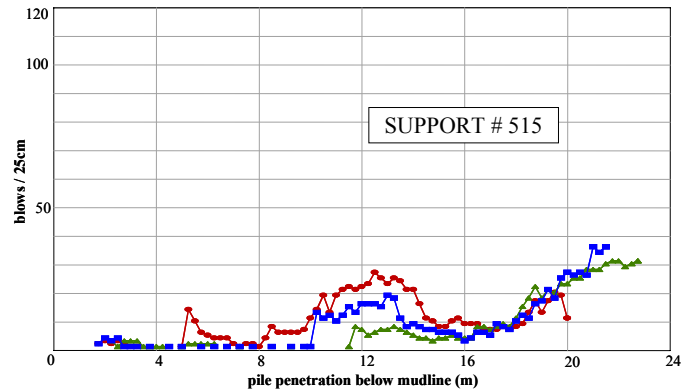


Fig. 8b - Typical Blowcount Records of Area 2

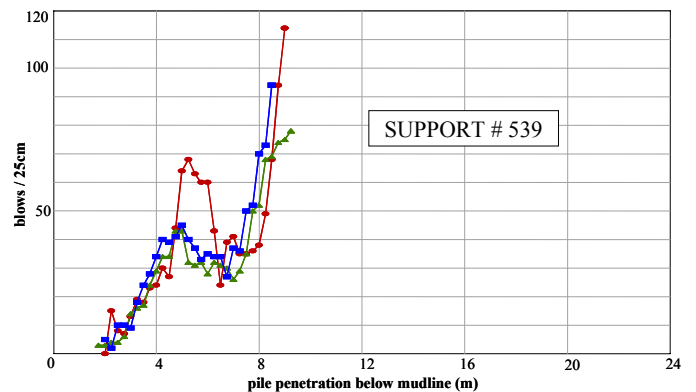


Fig. 8c - Typical Blowcount Records of Area 3

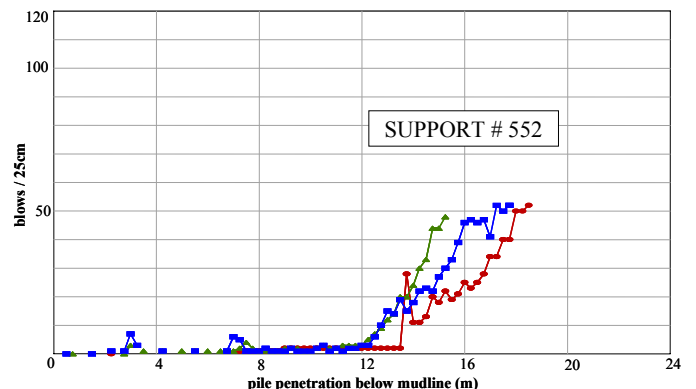


Fig. 8d - Typical Blowcount Records of Area 4

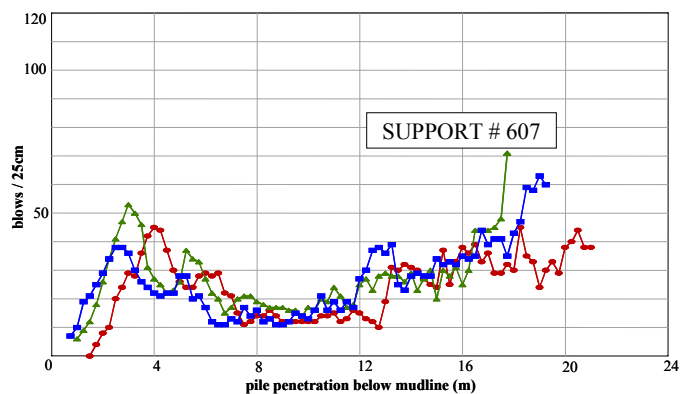


Fig. 8e - Typical Blowcount Records of Area 5

conventional driving pile solution.

The main advantages of the suction pile solution were:

- low cost due to the reduced installation time;
- immediate load transfer to the new piles;
- reduced risk of failure of the damaged existing foundations by avoiding the vibrations induced by pile driving;
- no development of high magnitude settlements due to the grout filling of the void between the caisson steel top plate and the soil plug;
- possible reestablishment of the original pipeline geometry in the zone affected by the foundation collapse;
- single strengthening solution for both granular and cohesive soil conditions.

This concept was abandoned, because during the emergency repair there were strong indications that the geotechnical boreholes were not representative for the project and a high risk of refusal was intrinsic. Due to this doubt a driven pile solution was developed. During the installation it was verified that the problem was not the boreholes, but the erratic nature of the soil deposits. Soft clay pockets and compact to dense sand layers are distributed along the pipeline longitudinal axis. Only a more comprehensive extended geotechnical campaign could provide enough information to evaluate in advance these behaviors. Without enough geotechnical investigation a digital mono-channel reflection seismic survey helped the foundation designer to estimate the final penetration in each one of the reinforced supports.

Although driven piles are widely used in all kinds of foundation design, the concept using a tripod type structure proved to be advantageous for offshore use.

ACKNOWLEDGEMENT

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