Innovative deep-mixing methods for oil & gas applications

G. Spagnoli
Department of Maritime Technologies, BAUER Maschinen GmbH, Schrobenhausen, Germany

D. Bellato
Department of International Projects and Services, BAUER Spezialtiefbau GmbH, Schrobenhausen, Germany

P. Doherty & G. Murphy
GDG Geosolutions, Dublin, Ireland

ABSTRACT: Notwithstanding the increasing investments in the use of renewable sources of energy and natural gas observed in recent years, oil still represents an important part of the global energy market. In particular, oil overall consumption and production are expected to continue growing, especially in emerging economies, though more slowly than in the past due to international policies aimed at slowing down climate changes. Plants and infrastructures related to the oil industry are often inherently related to challenging projects, which, in most of the cases, have to fulfill strict technical, environmental, and economical specifications. For instance, refinery sites located in sub-urban contexts have to face stringent environmental requirements both during production and after their dismantling to ensure that no pollution spreads across the surrounding areas. On the other hand, the construction of offshore platforms in difficult geotechnical conditions presents issues, which can be solved only by means of innovative technical solutions. This paper presents two case histories in which deep-mixing techniques were used to produce improved-soil elements for environmental and structural purposes related to the oil industry, i.e. a cut-off wall in an ex-refinery site and foundation piles for the support of offshore platforms in carbonate sands. Deep-mixing (DM) of soil is a well-established methodology in geotechnics introduced more than 50 years ago in Japan. DM methods have been used so far in numerous applications both on-shore and offshore projects. The use of DM is expected to become more and more popular in the future due to its economic and environmental advantages compared to other traditional construction techniques.

1 INTRODUCTION

Fixed platforms have been traditionally used to exploit offshore hydrocarbon reserves in water depths of less than 200 m, although there are several examples of fixed jackets installed in deeper water, e.g. Magnus platform installed in 186 m water depth in the North Sea (Clarke 1993), the Bullwinkle oil platform in the Gulf of Mexico installed in 412 m water depth (Digre et al. 1989). Offshore fixed production platforms are globally installed. In the period 2008–2012, the majority of the fixed platforms where installed in Asia and Middle East. As a matter of fact, Asia’s demand for fixed platforms is increasing especially for developments in Malaysia and China, with large proportion of platforms situated between 25 and 50 m water depth. Platforms installed in depths over 100 m are common in South America and Africa. Regarding West Africa, Angola and Nigeria are the key countries driving the market growth (Infield 2013). It is possible to observe from Fig. 1 that the majority of fixed production platforms are pile-based. In fact, the most common type of offshore foundation is the open-end driven steel tube pile (Poulos 1988) with diameter ranging from 1 m up to 6 m (for offshore renewable energy) and lengths up to 300 m (Gerwick 2007). However, there are situations where the pile installation by means of hammers is not possible, for instance in rock conditions, where boulders and/or cobbles are encountered or in calcareous deposits. Therefore, drilled piles are normally used in these conditions. George and Wood (1976) identify three basic types of offshore drilled piles:

- Single-stage drilled piles, which can be formed by drilling an oversized hole to the required penetration, inserting a steel tube pipe and grouting the annulus between pile and soil;
- Primary driven pile, which forms the casing for the upper section of the insert pile. An oversized hole is then drilled to the required penetration below the primary pile tip, and the annulus between the insert pile and soil and the primary pile is grouted;
- Belled pile.

According to Poulos (1988), Gerwick (2007) and Doherty et al. (2016) offshore drilled piles are installed with top drilling units with facilities for both direct and reverse circulation, with the latter normally used for relatively large-diameter holes. Offshore piles resist
Abbs et al. (1988). According to Randolph (1988) for 30° N, however, they are also frequent up to latitudes 50° S in coastal areas of Australia, India, Saudi Arabia. However, they are also frequent up to latitudes 50° N and 50° S (Le Tirant and Nauroy 1994). It is very well-known that they show comparable friction angles to calcareous grains.

The tendency to contraction during shearing has therefore serious implication for pile shaft resistance, which depends on the development of lateral stress along the pile wall (e.g. Murff 1987; Colliat et al. 1999; Gerwick 2007). The skin friction of driven piles in carbonate sands (where the estimation of K and tan δ is uncertain) is given by (Le Tirant and Nauroy 1994):

\[ \tau = \text{Inf} (\beta \sigma'_{v}; \tau_{lim}) \]  

where β varies from 0.05 to 0.2 (Dutt and Cheng 1984; Abbs et al. 1988). According to Randolph (1988) for long piles, the very low β values suggest that it is not correct to relate the value of skin friction to the overburden pressure. It seems more appropriate to think in terms of absolute values of unit skin friction, which lie in the range of 5 to 15 kPa. Drilled-and-grouted (D&G) piles overcome the difficulties associated with particle crushing and compression at the pile interface and are frequently used for offshore foundations in these deposits (Lee and Poulos 1991). As a result, the radial effective stress will remain close to the in situ horizontal stress, yielding significantly higher values of shaft friction than for the driven counterparts. This has resulted in the prominence of D&G piles in calcareous sand deposits (e.g. King et al. 1980; Gerwick 2007). However, D&G piles is a costly foundation solution and therefore, despite the geotechnical properties of carbonate sands, driven piles with closed-ended piles have been installed in the past for saving costs (e.g. De Mello et al. 1989) or grouted driven piles have been suggested, where cement grout after driving is injected (Barthelemy et al. 1987). Barthelemy et al. (1987) tested a 762 mm diameter pile equipped with grout pipes and instrumented with 36 strain gauges was driven into calcareous sands down to 24 m. The following paper presents and discuss the latest data about a novel mixed-in-place pile (MIDOS) already described in Igoe et al. (2014); Spagnoli et al. (2014); Doherty et al. (2016). As the MIDOS is based on the mixed-in-place technology, an onshore test with the Cutter Soil Mixing (CSM) in an ex-refinery site is presented. Laboratory tests of MIDOS pile in carbonate sands will also be briefly discussed.

2 THE DEEP MIXING METHOD

2.1 Introduction

The Deep Mixing Method (DMM) is an in-situ soil treatment technology whereby binding materials are added and blended with soils in order to improve their hydraulic and mechanical properties. Deep Mixing techniques, originally developed in Sweden and Japan during the 1960s and 1970s, are today well-established procedures in the geotechnical engineering practice of an increasing number of countries. The reasons for this success can mainly be due to the several engineering purposes they serve as alternative, more economic (e.g. Topolnicki 2004), and environmental friendly solutions with respect to the traditional methods involved in ground improvement works and to the constant technological development carried out on mixing rigs. The performance of deep mixing structures depends significantly on the mixing process implemented at the site, which is much more effective when a homogeneous distribution and uniform blending of the binding material with the soil is achieved (Mitchell 1981). First attempts to classify in a rigorous way deep mixing techniques were made by Bruce (2000). This classification depends on several fundamental operative features as follows:

- The method with which the binding material is introduced into the subsoil, namely in a Wet (pumped as a slurry) or a Dry (blown in pneumatically) form.
- The approach adopted to penetrate the ground and/or to blend and homogenize the chemical agents with the soil: purely by Mechanical methods adding the binder at relatively low pressures, or by a rotary method aided by Jet systems which injects fluid slurry at high pressure. The classification does not include jet-grouting, as it does not involve any mechanical mixing to improve the stabilized mass.

![Figure 1. Fixed platforms installed in the period 2008–2012 (modified after Infield 2013).](image-url)
The vertical position of the mixing tools along the drill shaft: the incorporation of the binder and its homogenization with the soil can be carried out at the end (or within a distance comparable with the characteristic size/diameter of the tool) or along a significant portion of the shaft.

In practice, not all the combinations provided by this flowchart are available for ground improvement applications since wet slurry, jetted shaft mixing (WJS) and dry binder, rotary, shaft mixing (DRS) do not exist, and no jetting introducing dry binder (DJS or DJE) have been already developed. Furthermore, recent technologies based on driven mechanisms different from those involving rotary systems cannot be correctly identified using this scheme.

In order to overcome such limitations, Topolnicki (2004) elaborated a more general classification which allows distinguishing between different in-situ soil mixing treatments (Fig. 2). The basic indicators used to operate this distinction are the same considered by Bruce (2000), even if the mixing principles are here extended to include recent advancements in deep mixing machinery.

**2.2 Cutter Soil Mixing (CSM)**

The Cutter Soil Mixing (CSM) technique is a wet mechanical mixing technique developed in 2003 that differs significantly from conventional DMMs, since it makes use of two sets of cutting wheels rotating around horizontal axis (Fig. 3A) to produce rectangular panels of improved soil rather than one or more vertical rotating shafts creating circular stabilized columns. Therefore, CSM can be considered a WME technique.

The counter-rotating mixing wheels and the cutting teeth push the soil agglomerates through narrow gaps between vertical steel plates, named “shear plates” (Fig. 3B). This produces a sort of forced mixing action which enhance incorporation and homogenization of the binder into the soil.

The CSM unit can be mounted on a guided Kelly bar or a wire rope suspended cutter frame equipped with special steering devices. Depths up to 60m have been successfully reached using wire rope-suspended rigs provided with four sets of mixing wheels. Panels having a width ranging from 2.4 m to 2.8 m and a thickness between 0.55 and 1.5 m can be created.

**2.3 The MIDOS technology**

The MIDOS pile is a hollow steel pile, which is lowered into the soil while at the same time a mixing tool, which is advanced inside the hollow pipe, mixes the in situ soil with cement slurry (Fig. 4). Hence, a mixed soil cement body is created.

A test pile has been installed in the Bauer’s field test facility in Bavaria (Germany) in a silica sand deposit with the following dimensions: the steel pile had an external diameter of 1.5 m with a wall thickness of 15 mm. The steel has at its lower end a mixing chamber with an outer diameter of 1.9 m. Holes in the steel pile and in the mixing chamber allow the soil-cement-mix to communicate between inside and outside of the pile. The steel pile with the mixing chamber remains in the soil, surrounded by a soil-cement-mixture. The pile length was 17 m and it was installed in less than 3 hours (see Igoe et al. 2014).

Because the MIDOS reached a static uplift capacity of about 9 MN (Igoe et al. 2014) and a static bearing capacity mobilized by the dynamic pressure of the load test of 15.4 MN, it was decided to extend the investigation to calcareous sands, which as previously described, present foundation issues.
site in Leuna (Germany) in order to limit the groundwater contamination in the surrounding area. The wall was 450 m long (6400 m²) and 203 panels were necessary for its completion.

In addition to the cut-off wall, a retaining wall was produced for a separate test purpose.

3.2 Geotechnical characterization of the site

A comprehensive geotechnical investigation was accomplished in advance to the CSM wall construction. It included several boreholes and dynamic penetration tests. Furthermore, laboratory tests for the determination of grain size distribution, permeability and triaxial unconsolidated and undrained strength were carried out on soil samples retrieved from the boreholes. Some of these samples were also classified by means of Atterberg limits and chemical analyses. Inside each borehole a piezometer was installed to monitor the groundwater level and the quality of the water before and after treatment.

The resulting grain size distribution curves are depicted in Fig. 5. Each curve in the graph is individuated by an alphanumeric string reporting the borehole number, the sample number, and the depth at which the sample was taken. The curves show the great heterogeneity characterizing the subsoil at the site of Leuna.

Triaxial undrained tests were performed on undisturbed cohesive samples collected from the site. The undrained shear strength varied from 120 kPa to 247 kPa, denoting the high overconsolidation of the clay deposits.

The classification of these soils based on the Casagrande plasticity chart is shown in Fig. 6. The finest fraction could be classified as inorganic silty clay of medium plasticity (CL). Moreover, hydraulic conductivities of the order of magnitude of $10^{-10}$ to $10^{-11}$ m/s were measured on soil specimens representative of the fine deposits of Leuna.

Close to each borehole, a dynamic penetration test was executed in order to obtain a more effective correlation between test results and visual inspection. An example is presented in Fig. 7, in which the DP results are compared with the information contained in the corresponding borehole report.

From the in-situ and laboratory investigation campaign the following geotechnical profile was derived.

The first layer consisted of loose to medium-dense sandy fillings, with a variable thickness of 0.0–2.2 m from the ground level (up to 2.64 m close to B5413).

The second level, containing a larger fine fraction with respect to the first, was composed of Loess, glacial marl, or a less weathered gravel terrace depending on the considered site area. The cohesive part had a soft to hard consistency, while the granular material was characterized by a loose to medium-dense state. Some organic formations of soft to stiff consistency were encountered between 2.2–6.1 m from the surface.

Sand and gravel of the alluvial terrace, prevalently at a loose to medium dense state, formed the third level. In some DP tests, $N_{DP}$ was over 60 blows per 10 cm. These high values were probably related to the presence of boulders or cobbles. The thickness of this layer was variable throughout the jobsite area and was between 6.1 m and 10 m below the ground level. On the north (B5407 to B5409 and B5500), the surface quaternary deposits described so far were found to lie directly on the bedrock.

In the remaining area, a tertiary layer was located in between. The tertiary deposits incorporated a sequence of non-carbonate sands, silts, and clays with
a thickness ranging from 13.5 m to 41.0 m. The cohesive formations had a stiff to solid consistency, which led, in some cases, the DP tests to refusal. The tertiary sands were denoted by a medium dense state increasing in density with depth. In some boreholes, local organic formations of lignite were identified. Finally, the weathered and disaggregated sandstone bedrock with a stiff to semi-solid consistency was reached at a variable depth, which in the North-western area of the jobsite was approximately comprised between 5.0 m and 7.0 m.

### 3.3 Mix design, production data, and results

The cut-off CSM panels had a maximum depth of 16 m below ground level and a thickness of 0.64 m. The construction was performed following the one-phase system and the back-step procedure (fresh-in-hard), with the realization of 2.8 m wide primary panels and 2.0 m wide secondary panels, i.e. with an overcut of 0.4 m at each side.

As far as the cut-off wall production concerns, the mix parameters provided in Tab. 1 were adopted at the site. In order to introduce into the ground an amount of cement corresponding to 152 kg/m³ of natural soil, a flow rate of 858 l/m³ was assumed. The cement used was a CEM III 32.5 N NW/HS (85% slag and 15% OPC). The retaining wall, on the contrary, was created with a higher amount of cement to ensure the achievement of a predefined strength level (Tab. 1).

The same flow rate of 858 l/m³ was adopted to introduce into the ground an amount of cement corresponding to about 550 kg/m³ of natural soil. Furthermore, the same binder as in the cut-off wall was used.

Hydraulic conductivity values of the order of magnitude of $10^{-9}$ to $10^{-11}$ m/s were obtained from permeability tests performed on cut-off wall samples. Unconfined compressive strength (UCS) values ranging from 0.5 MPa to 6 MPa were measured on samples collected from the structural wall.
4 THE MIDOS TECHNOLOGY FOR OFFSHORE PILED FOUNDATION APPLICATIONS

4.1 Laboratory tests in calcareous and silica sands

To investigate the feasibility of using MIDOS in calcareous sands, a laboratory study was required to assess both the geotechnical performance of the pile and the structural behavior of the Mixed-in-Place (MIP) grout, which forms the pile body. Doherty et al. (2015; 2016) showed that the silica sand tested (Blassington sand) was chosen as a control or reference sample. This was deemed particularly important because the MIDOS system has already been shown to be an effective pile installation technique in silica deposits, with the load carrying capacity shown to be in-line with predicted values for conventional D&G piles. As a result, direct comparison of the silica and calcareous sand behavior was deemed essential for this study. The testing regime focused on both the potential geotechnical and structural failure modes. A carbonate sand from Dog’s Bay (west coast Ireland) was used. Portland CEM II cement was used throughout the testing regime and a water/cement (w/c) ratio of 0.4 was adopted for the tests as suggested by ISO 19902 for fixed steel offshore structures. Different cement-to-sand ratios (c/s) were used for the tests, i.e. 15-25-35%. These values were chosen to simulate a high installation (15% c/s) and low installation time (35% c/s), where the cement would have more time to be mixed with the soil. The 25% c/s is a compromise value between the low and high installation time. A series of tensile tests and UCS tests were performed at 3 days, 7 days and 28 days and across the range of c/s ratios noted above. The shear rate for the tensile test was 50 N/s whereas for the compression test was 0.5 MPa/s (Doherty et al. 2015). The results demonstrated identical trends in all cases, regardless of c/s ratio and curing time. This is particularly evident in Fig. 8, which directly compares the compression and tension tests of grout samples with the same testing time and c/s values (Doherty et al. 2015). Over the entire range of parameters considered, the grout samples demonstrated comparable behavior, suggesting that the structural performance of the MIDOS pile body should be similar in both calcareous and silica deposits.

4.2 Finite element analysis

Spagnoli et al. (2015) showed MIDOS pile models were developed with Plaxis 3D to analyze a range of pile length to diameter ratios in silica and calcareous sand deposits. In order to investigate the behavior of MIDOS piles in calcareous sand, suitable soil parameters were calibrated in order to reflect the load bearing behavior of this type of soil more precisely. The range of model dimensions and the corresponding model numbers are presented in Table 2. Six geometry models were analyzed for each soil type, to cover the range of diameter/length combinations presented.

In tension the shaft resistance is the primary resistance component. The radial effective stress at failure is highly dependent on the initial horizontal effective stress (at rest condition) $\sigma'_{rc}$, as the MIDOS pile aims to maintain a constant horizontal lateral stress during installation the simplistic “wished-in-place” model construction should provide a reasonable initial installation stress condition – compared to that of driven piles where the driving stresses can be impossible to accurately model. The soil models used in this study are assumed to be normally consolidated and the lateral earth pressure coefficients are automatically determined by Plaxis as $K_{nc} = 1 - \sin \varphi'$. In practice the
initial soil stresses would be assessed and inputted directly to Plaxis using the results of in-situ geotechnical testing such as CPT, to simulate the specific conditions encountered at a real site. As a simplified method of allowing for some strength reduction a “soft soil” layer was modelled underneath the chamber to a depth equal to the chamber diameter. The soft soil was given stiffness parameters equal to half of the regular sand parameters and a friction angle equal to the constant volume friction angle of the soil in the respective model. This has only been considered in the silica sand, as the soil beneath the chamber in calcareous sand is modelled using the crushed parameters. The silica and calcareous sand models were calibrated using triaxial tests. For the calcareous sands, triaxial tests conducted at varying effective confining stresses (100 kPa and 1000 kPa) found that sand stiffness deteriorated significantly at higher confining stresses, subsequent grain size distribution on those samples showed clear evidence of particle degradation during the 1000 kPa triaxial tests. Due to the limitations of the hardening soil model, the soil degradation and stress could not be calibrated into a single soil model. Therefore two sets of soil properties were adopted and the soil properties were optimized using triaxial tests on uncrushed and crushed samples. A limit stress of 400 kPa was selected as the pressure at which the sand will begin to deteriorate. Once this stress was reached, the soil properties were changed to that of the crushed sand to model the stress softening behavior of the calcareous sand. This process required the model to be first run using the intact soil only. The resulting soil stresses were then examined and the depth of soil where the stresses exceed the 400 kPa determined. The soil body below this depth was assumed to be crushed, while above this depth, the non-crushed soil model was used. Therefore, in the next stage of analysis, a mixed soil model was generated, where the soil properties at depths below the identified depth were modified to represent the behavior of crushed soil, and the model was re-run. This methodology was found to provide a more realistic response than adopting a fully crushed or non-crushed approach (Spagnoli et al. 2015). Comparison of the tension and compression performance of the various pile geometries showed some scatter in the results obtained, however in general the simulations highlight similar performance between the piles in silica and calcareous sand (Spagnoli et al. 2015).

The results of numerical analysis of the piles in silica sand were also compared with those predicted using the API method, see API (2014), and a CPT-based approach, see Igoe et al. (2014), assuming three different values for the average $q_c$ of the seabed (see Fig. 9). The comparison shows that the Plaxis models predict significantly lower capacities than the analytical predictions. It should be noted that Finite Element Modelling is not particularly suited to large deformation analysis and convergence issues may stop the model before the ultimate limit state capacity, as predicted in the analytical methods, is reached.

5 CONCLUSIONS

Offshore driven piles are the most common foundation type. However, drilled-and-grouted piles, although time consuming, are used where driven piles reach their limits. The Deep Mixing Method (DMM) is an in-situ soil treatment technology whereby binding materials are added and blended with soils in order to improve their hydraulic and mechanical properties. This paper analyzed two case histories, where this technology successfully was used for an onshore and for a potential offshore application.

REFERENCES
