Soil-Structure dynamic interaction: application to design and construction of the facilities of a gas power plant

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ABSTRACT

This paper shows the different structural and geotechnical aspects of the design and the construction of the facilities of a gas power plant under static and dynamic loading conditions. The facilities comprehend a 50m high blow-down system, three compressor machines, a steel industrial superstructure and a reinforced concrete building. The plant is located in Sicily, a medium to high seismic zone of Italy, hence the soil-structure dynamic interaction needs to be considered. The loads acting on both the superstructure and the piled foundation system have been determined by means of a SSI analysis for the most sensitive and important structures, i.e. the 50m high blow-down system and the compressor machines.

Keywords: soil-structure dynamic interaction, earthquake, vibrations, piled foundations, structural and geotechnical engineering

1 INTRODUCTION

In common practice structural engineers use a detailed model for the superstructure and a simplified system for the ground, vice versa geotechnical engineers design the foundation system by means of a refined model for the ground, but assuming a single degree of freedom superstructure if not only the transferred static or inertial loads from the superstructure.

In this paper a case history is documented (Locatelli & Roma, 2004) in which the soil-structure interaction between the ground and the superstructure has been considered, especially when dealing with particular structures, such as towers or vibrating machines subjected to dynamic loading (Barkan, 1962, Bowles, 1996, Gazetas, 1983, Wolf, 1985).

2 GEOLOGICAL AND GEOTECHNICAL CHARACTERISATION

The gas power plant already exists (see figure 1) and belongs to ENI S.p.a. and additional facilities need to be constructed.

The gas power plant is located in Gagliano Castelferrato (Enna, Sicily, Italy) on a floodplain of the stream Gagliano on a slightly inclined slope (i=10°) from East towards West. The area is of medium to high seismicity and according to the new national seismic classification (OPCM 3274, 2003) the site is of type E and is located in zone of class II, which means a PGA=0.25g.

Geotechnical investigations have comprised both in situ and laboratory tests:

- 7 boreholes: 5 down to a maximum depth of 20m and 2 to a maximum depth of 30m
- an array of seismic refraction and an array of SAW test (Spectral Analysis of Surface Waves) (Roma, 2001, 2004)
- 2 loading plate tests and standard penetration tests SPT
- 3 Piezometers
- Triaxial compression CIU and CD test under drained and undrained conditions
- Direct Shear test under drained conditions
- Unconfined compression test
- Atterberg limits and granulometry determination
- Edometer tests

On the basis of the results of both the geotechnical investigations and the geological information three main geological units can be recognized from the ground level below:

- Alluvium deposit
- Weathered and plastic Varicolari clays
- Very stiff brown clays with blocks of quarzareniti (flysch Numidico)

Figure 1. Landscape with the gas power plant

Table 1: Geotechnical units and soil characterisation

Geotechnical unit	Soil description	γ (kN/m^3)	φ' (°)	c' (kPa)
1	Superficial ground	18.5	19	0
2	Grains into clayey, silty matrix	18.5	19	0
3	Weathered plastic clay	20	21	10
4	Stiff clay	20	24	15
5	Very Stiff clay	20	25	25

Following the geological events, the geological older varicolari clays are located at a smaller depth respect to the geological younger flysch clays, hence it is expected that the whole deposit be overconsolidated. This aspect has been confirmed by the laboratory tests performed on undisturbed samples of soil at different depths.

In the smaller geotechnical spatial scale a more refined characterization can be made with respect to the geological characterization. The geotechnical units found during in situ investigations, are reported in table 1.

The undrained shear resistance Cu has been determined based on laboratory tests as a linear function of the effective stress, which depends on stratigraphy, depth and water table position. Figure 2 reports the results of a laboratory CIU compression test on an undisturbed sample of the geotechnical unit 2 at a depth of 5m. Generally the water table has not been found during drilling, down to the maximum investigated depth of about 30m, except in a borehole where the water table position was found at a depth of about 12m. Anyway the cohesive nature of the soil, which tends to saturation after raining, suggests to consider the water table at ground level in the analysis.

Table 2: Shear wave velocity and shear modulus at very small deformations Go profiles by SASW and seismic refraction tests

Geotechnical unit	Depth from g.l. (m)	Vs (m/s)	Vp (m/s)	Go (MPa)
2	0÷7	80	305	12
3-4	7÷13	530	1425	560
4-5	13÷20	515	2000	530
5	>20	900	2230	1620

3 DESCRIPTION OF THE FACILITIES AND THEIR FOUNDATION SYSTEM

The facilities to be constructed consist of (see figure 3):

- a blow-down system high 50m and a base mat foundation realized by an inferior r.c. slab with dimensions 7.80m x 7.80mx 0.50m and by a superior r.c. hollow block, filled with gravel, with external dimensions 6.20m x 6.20m x 1.60m and lateral

walls with a thickness of 0.7m and a superior slab with a thickness of 0.50m;
- 3 r.c. foundations for the compressor machines realized by an inferior r.c. slab with dimensions 13.10m x 8.00m x 0.70m and by superior r.c. hollow block with external dimensions 9.70m x 2.60m x 1.50m and lateral walls with a thickness of 0.5m÷0.7m and a superior slab with a thickness of 0.30m;
- a steel framed shed with dimensions 44.00m x 15.00m x 12.00m constructed to cover the compressors;
- a reinforced concrete structure with external dimensions 15.50m x 12.50m x 8.00m, realized to contain several electrical devices.

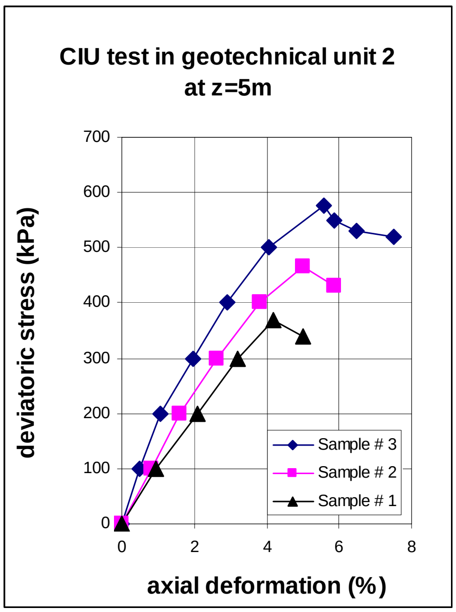

Figure 2. CIU triaxial compression test on a sample of the geotechnical unit 2

For all the facilities reinforced concrete bored piles have been adopted. The foundation characteristics of each facility are described in table 3.

Table 3: Foundation system under each facility

Facility	Foundation	D (m)	L (m)	test piles	Nmax (ULS) (kN)
Blow-down system	Mat with 8 bored piles	1.0	20	1	663
Compressors	Math with 12 bored piles	1.0	20	1	598
Steel frame	Beams on bored pile	0.6÷1.0	20	-	425
Concrete building	Beams on bored piles	0.6÷1.0	20	2	676 ÷ 480

D, L = pile diameter and length; Nmax= design axial load at ULS.

4 DESIGN APPROACH

Both a conventional approach and a complete SSI analysis have been considered for the design of the facilities under inertial loads. Also cinematic effects have been determined seperately.

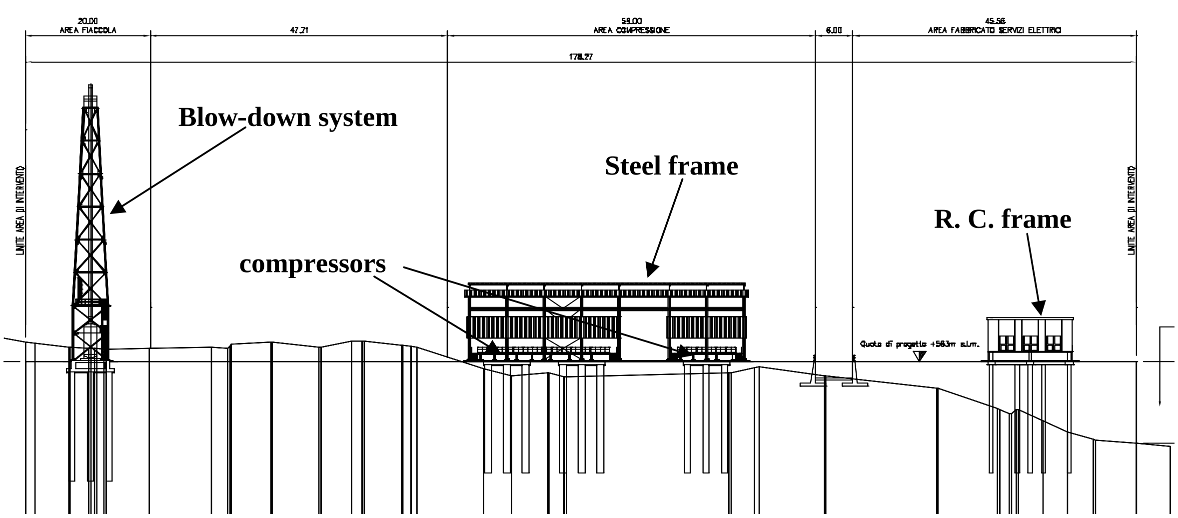

Figure 3. Section view of the new facilities to be constructed into the gas power plant.

The conventional approach, commonly used in practice for inertial loads, does not consider soil-structure dynamic interaction, since the superstructure is considered fixed at the base and the reactions at the fixed base are successively used as actions on the foundation system. Soil-structure dynamic interaction by means of FEM (Bowles, 1996) allows for a complete model of both the superstructure, the foundation system and the surrounding soil. (see figure 4 and figure 5).

The interaction between the pile and ground has been modelled by means of dynamic Winkler's springs, which are characterized by an elasto-plastic behaviour. Both the limit yielding pressure p_{lim} (equation 1) and the dynamic elastic stiffness $k_{dynamic}$ of the Winkler's springs (equation 2) have been calculated as a function of the undrained shear resistance of the soil Cu, when transient loading events, such as earthquakes occur.

$$p_{\lim} = 0.72 \cdot 0.9 \cdot Cu \text{ (Poulos \& Davis, 1987)} \tag{1}$$

$$k_{dynamic} = \eta \cdot k_{static} \text{ (Bowles, 1996)} \tag{2}$$

$$k_{static} = \frac{E}{\left(1-\nu^2\right)} \tag{3}$$

the dynamic factor η depends on the dimensionless frequency

$$a_0 = \frac{\omega \cdot 0.5 \cdot D}{V_S} \tag{4}$$

which contains information about the pile diameter D, the shear wave velocity of the soil Vs and the circular frequency ω of the external load (earthquake, wind, machines). E and υ are the Young modulus and the Poisson ratio of the soil.

By comparing the two methods of analysis it has been observed that when using a conventional approach the stress level and the deformations of the structural elements (beams and columns) of the superstructure are approximately 30% lower with respect to the results obtained with a complete SSI analysis under the same soil and loading conditions. This means that using fixed constraints instead of the actual foundation system at the base of the superstructure may cause a significant underestimation of the safety level of the structure, especially when dealing with soft superficial soils.

5 NATURAL FREQUENCIES OF THE SOIL DEPOSIT

Also the natural frequencies of both the ground and the foundation-superstructure system have been calculated, to assure that resonance phenomena will not occur for travelling shear S waves (fs) and superficial Rayleigh waves (f_R).

$$f_S = \frac{V_S}{4h} \tag{5}$$

$$f_R = \frac{(a+bj)V_s}{H} \text{ (Roma et al., 2001)} \tag{6}$$

where a=-0.09, b=0.65 are two constants and j is the Rayleigh mode of reference.

Two different situations have been considered:

Case A) only the soil layer of soft clay (geotechnical unit 2) has been considered with Vs_1=80 m/s and thickness h_1=6.6m

Case B) the first 30m of soil have been considered with an equivalent shear wave velocity Vs_{30}= 251m/s, according to Eurocode 8.

Table 4 reports the natural frequencies of the deposit for the cases A and B, when considering both vertically propagating shear waves (S) and Rayleigh waves (R) travelling on the free surface.

The higher natural frequency of the soil deposit f_{site}=6.8Hz has been compared with the natural frequencies of the whole system (superstructure and foundation system) $f_{min\_structure}$=10.9Hz (the lower frequency is the horizontal translational mode) and the operating frequencies of the compressors f_{exc}=16Hz (lower frequency of exercise) (see table 5). It has been verified that resonant conditions do not occur with the soil deposit, since f_{exc}/f_{site} =2.36 and $f_{min\_structure}/f_{site}$ =1.61.

Table 4: Natural frequencies of the soil deposit (type E according to OPCM 3274, 2003)

Case	Vs (m/s)	h (m)	fs (Hz)	f_R (Hz)
A	80	6.6	3.0	6.8
B	251	30	2.1	4.7

h=thickness of the layer

Table 5: Verification of no resonant conditions

f_ecc (Hz)	f_site (Hz)	f_structure (Hz)
16	6.8	10.9

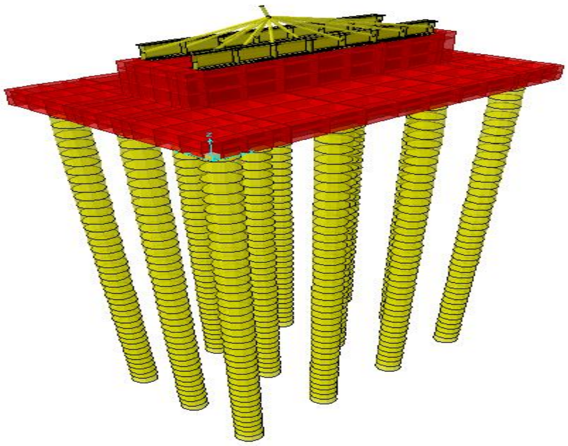

Figure 4. FEM for the soil-structure dynamic interaction between ground and compressors.

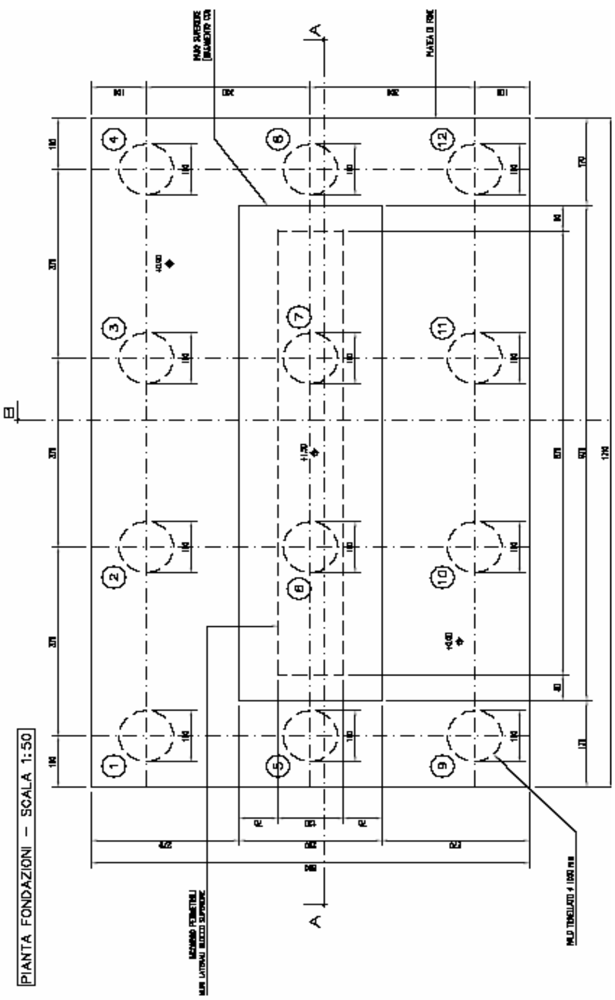

Figure 5. Plant of the piled foundation of the 3 compressors.

6 TESTING ON FULL SCALE PILES

After design of the piles 4 full scale tests have been prescribed on piles, which are representative of the different soil-foundation conditions. The loading test is increasing monotonic up to a value of 1.5 times the maximum expected load in exercise. Table 2 summarizes the location and the characteristics of the tested piles. Also in figure5 and figure 6 the test equipment and the load-settlement curve are shown regarding the loading test on the pile # 5 of the blow-down system. In the 1st cycle and 2nd cycle of loading the exercise axial load Ne=432 kN and 1.5·Ne have been reached with the maximum settlements respectively of w1=0.29mm and w2=0.48mm. The residual settlements are around 0.1mm. The results of the loading test prove that soil-pile behaviour is essentially elastic under the exercise loading conditions.

Figures 6. Loading test on pile # 5 of the blow-down system.

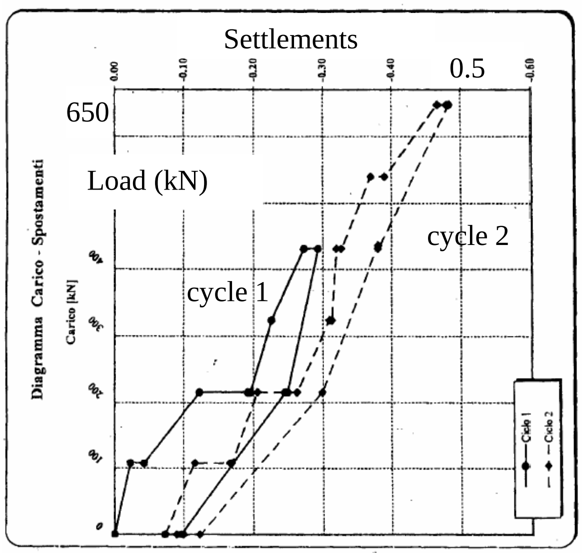

Figure 7. Load-settlement curve from 2 cycles of loading on pile # 5 of the blow-down system.

7 CONCLUSIONS

A methodological approach has been adopted for the design and the construction of the piled foundations of the facilities of a gas power plant under static loads and different types of dynamic loading (seismic, vibrations generated by machines). After the geological and geotechnical ground characterisation by means of a series of in situ and laboratory tests, soil-structure dynamical interaction has been considered to evaluate the loads on the superstructure and the foundation system of the most sensitive and important structures , i.e. the vibrating compressors and the 50m high blowdown system. A comparison between a complete SSI analysis with FEM and a simplified conventional approach with the superstructure fixed at the base reveals that with the simplified conventional method the safety level of the whole structure could be underestimated of about 30%, on average over all the structural elements. A series of loading tests on full scale piles has been prescribed to verify the correctness of the soil-pile behaviour predicted during the design phase.

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