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Foundations Conditions Study for Aeolian Power Units on Soft Soils under Static and Seismic Loads. Case Study

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FOUNDATIONS CONDITIONS STUDY FOR AEOLIAN POWER UNITS ON SOFT SOILS UNDER STATIC AND SEISMIC LOADS. CASE STUDY

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ABSTRACT

The paper presents the analysis of foundations conditions under static and seismic loads for a wind farm located in the Eastern part of Romania.

From lithological point of view, the location is characterized by a soft cohesive strata alternation over 40m deep. Some design considerations for obtaining the most economical foundation options are discussed.

By taking into account the static and seismic conditions, the soil – structure interaction (S.S.I.) is also revealed together with conclusions on the aeolian towers' foundations solutions.

INTRODUCTION

The paper presents the analysis of foundations conditions under static and seismic loads for a wind farm located in the Eastern part of Romania.

Because European Union ask that every country from EU has to have at least 20% from its energy generated from renewable sources, Romania prepare itself allowing big wind farms to be located near Black Sea coast in Dobroudgea and beyond Danube in Romanian Plane.

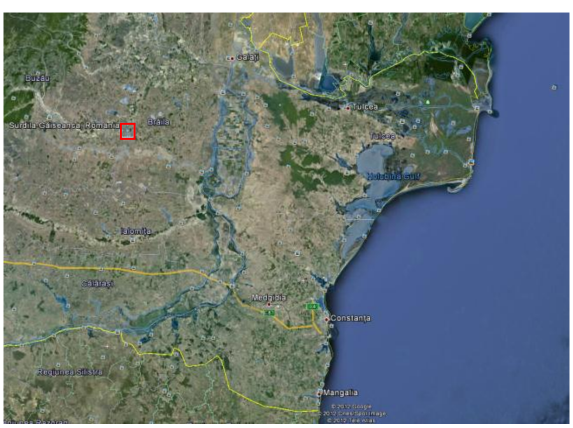

Fig. 1. Surdila-Gaiceanca location in Romania.

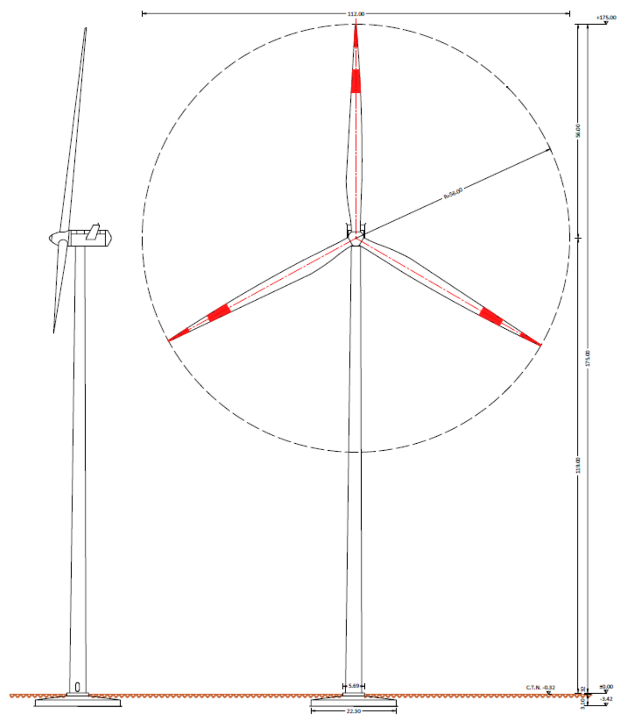

Fig. 1. Lateral view of WT – 3MW, h=119m.

From seismic point of view, Romania has a design code named “*Cod de proiectare seismică – Partea 1, Prevederi de proiectare pentru cladiri P100-2006*”. Shear base forces are defined with:

$$F_b = \gamma_I S_d(T_1) m\lambda \,, \qquad (1)$$

where:

γ_I - importance/exposal factor, depending on structure,

$S_d(T_1)$ - response spectrum corresponding to the first period of the structure.

m total weight of structure,

λ correction factor.

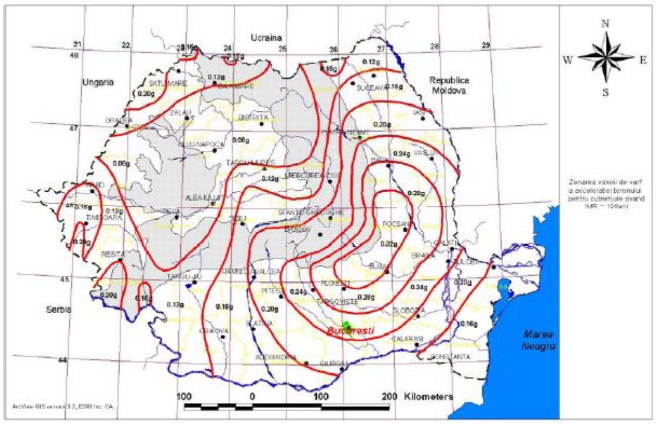

Fig. 3. Peak ground acceleration according to P100-2006.

As in can be seen in Fig. 3 seismic hazard is described through peak grouns acceleration a_g determined for average recurrence interval IMR=50 years for ultimate limite state a_g = 0,28 g, where g=9,81m/s^2, **a_g =2,747 m/s^2**.

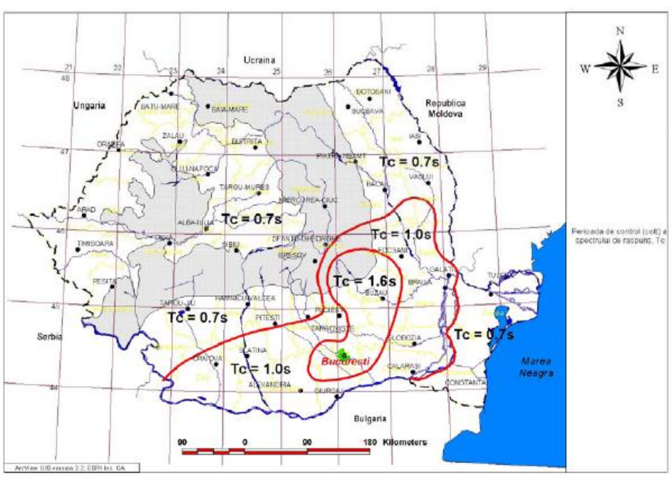

Fig. 4. Corner period according to P100-2006.

Corner period for response spectra for the site is T_c=1 s (*Fig. 4*).

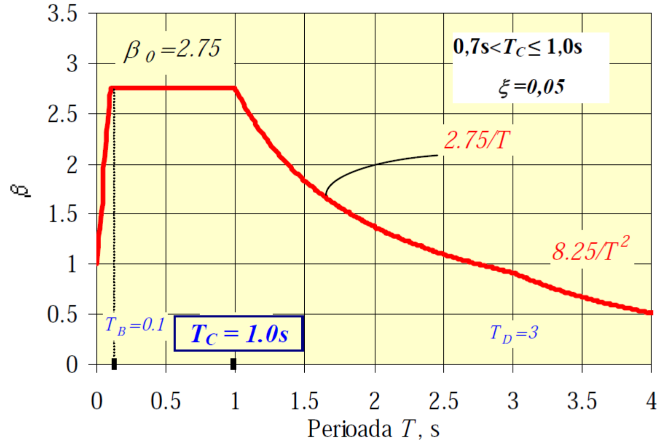

Fig. 5. Normalized spectrum for acceleration’s elastic response according to P100-2006

From 2012 in Romania is proposed another design code named “*Cod de proiectare seismică – Partea 1, Prevederi de proiectare pentru cladiri P100/1-2012*”. Main difference is that it design value of seismic force for is defined for 225 years average recurrence interval (probability of exceeding 20% in 50 years).

A new map of peak ground acceleration is proposed (see Fig. 6). As in can be seen in Fig. 5 seismic hazard is described through peak grouns acceleration a_g determined for average recurrence interval IMR=225 years for ultimate limite state a_g = 0,30 g, where g=9,81m/s^2, **a_g =2,943 m/s^2**.

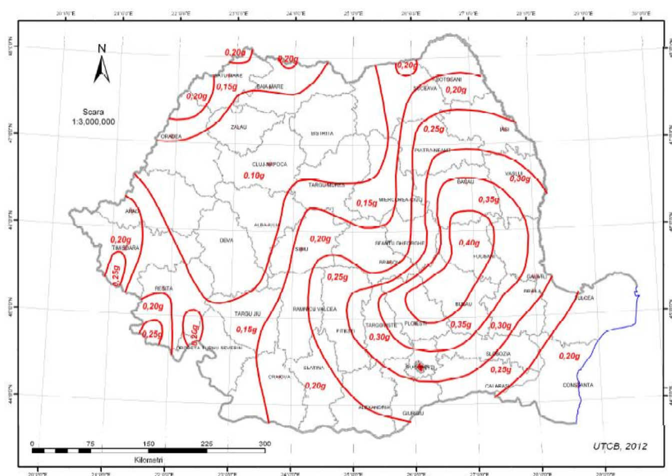

Fig. 6. Peak ground acceleration according to P100-2012.

Main differences are at T_B=0,2T_C, so that T_B=0,2 s.

From natural frequencies point of view (Fig. 9) it can be seen that almost every type o wind turbine (with steel or concrete tower) has natural period over 3,2 s, even if the soil is stiff, average or soft.

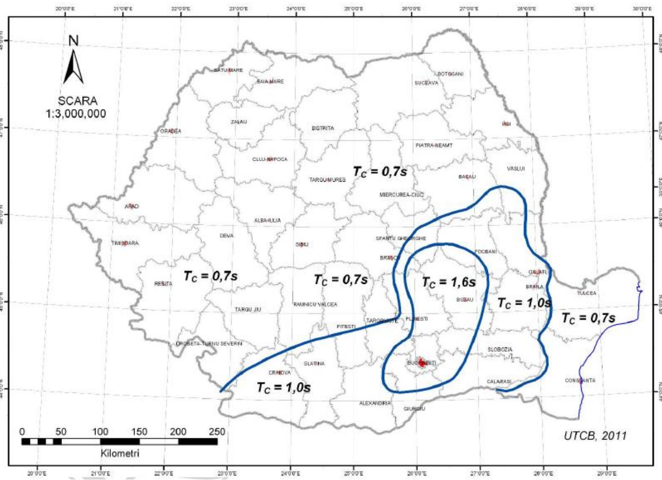

Fig. 7. Corner period according to P100-2012.

Regarding corner period (Fig. 7) the same zones will be used.

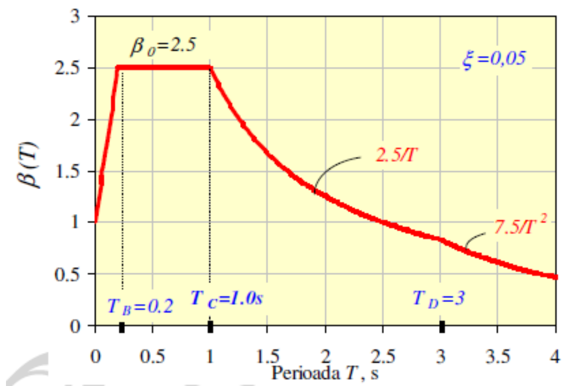

Fig. 8. Normalized spectrum for acceleration's elastic response according to P100-2012.

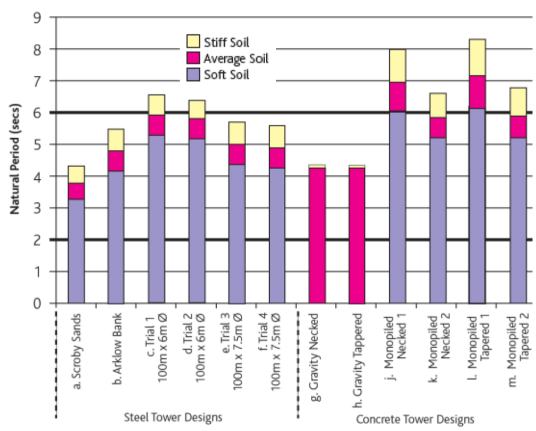

Fig. 9. Natural periods for WT (Gifford – Concrete towers for Onshore and Offshore wind farms)

2. GEOTEHNICAL INVESTIGATIONS

Geothnical investigation were made respecting national standard NP 074/2007. 9 drillings were made, corresponding to each of G1-G9 wind turbine location during November 2011 to march 2012 using WIRTH HD 10S and L100 machines.

First general conclusion of this studies is that a layer of about 25m of dusty sandy clays saturated with low consistency thin intercalations non-cohesive nature is on each site. This is also confirmed in geoelectrical studies (Fig. 9).

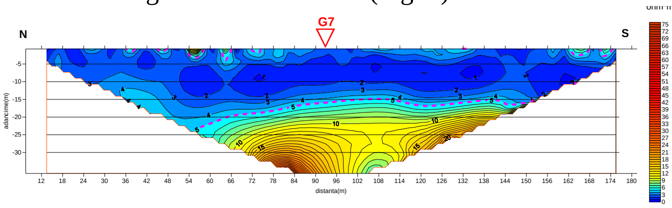

Fig. 10. Geoelectrical studies made at location G7.

It is confirmed that resistivities for the first 22m of soil is very low between 1.5 ÷ 5.5 ohmxm. From 22m depth resistivities are increasing to values between 25 ÷ 40 ohmxm which suggest cohesive lithological formations with better consistency.

For low consistency clays were made also dynamic tests in cyclic triaxial compressive machine and in resonant column.

Compression tests were performed in edometer in order to obtain edometric modulus and unit deformation. The maximum vertical effort of 300kPa was applied to identify stress - strain relationship in compression area (charge) / decompression. In terms of compressibility characteristics of site materials fall into the category of soil with high compressibility in natural conditions and very high in flooded conditions. M_{2-3} edometric module and value of unit deformation under normal specific effort level of 200kPa were considered for flooded samples.

In order to study the effect of treatment with (Portland) cement for soft clay samples were also performed compression test-settlement. Samples were treated with 8% and 12% cement were tested in terms of site saturated with salt water after 6 days. An increase in the amount of edometric modulus M_{2-3} from 4.167 kPa to 5.262 kPa in mixed with 8% cement (20% increase) and 6897 kPa mixed with 12 % cement (60% increase).

On samples from drilling were also conducted tests to determine the shear strength parameters, both by direct shear test and triaxial tests.

In direct shear apparatus, saturated samples were tested taking into account site conditions (nature-soil and geological effort applied) being sheared in conditions CU (consolidation effort geological area, v = 0.5mm/minut - sheared undrained) and in CD conditions (geological consolidation effort, v = 0.05m/minut - sheared drained).

On undisturbed samples belonging to the main geological formations were performed following types of tests:
- Type direct shear tests consolidated - undrained (CU) and consolidated-drained or type (CD);

- Compressive load and strain controlled triaxial stress conditions measured anisotropic type CKoD and CKoU, respectively;
- Triaxial compression tests with strain imposed and effort to follow those measures pore water pressure variation CKoU;

It is presented the stress-strain relationships recorded during shear.

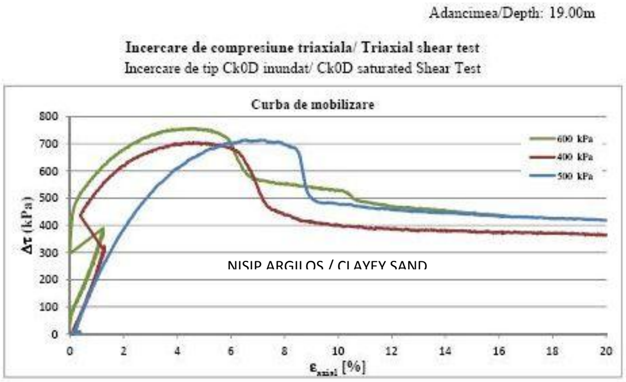

Fig. 11. Triaxial shear test CK0D - Δτ.

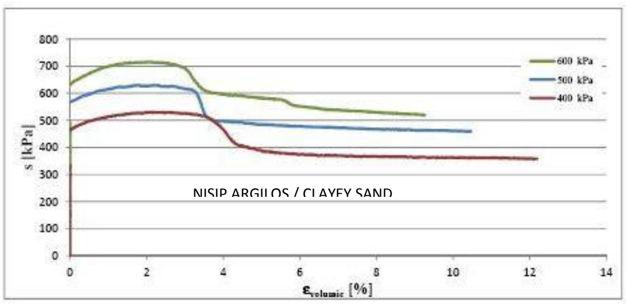

Fig. 12. Triaxial shear test CK0D - s.

From this analysis some conclusions can be drawn valid for the entire site of the wind farm, namely:
- The mean experimental deformation module where a degree of mobilization of shear strength of 60% are:

$E_n = (10 \div 11) \times 10^3$ kPa for clayey sands located at depths of over 20 m;

$E_n = (5 \div 6) \times 10^3$ kPa for clay dusts located at depths of over 10 m;

$E_n = (6 \div 7) \times 10^3$ kPa for clays located at depths of over 10 m;

Graphic processing geotechnical data field and laboratory investigation is shown in Figure - Correlation lithological column highlighting the systematic sequence.

In cyclic triaxial test machine anisotropic samples were consolidated on effort path k_0. Each sample was tested as follows: in the first stage from a vertical axial displacement imposed to 0.01mm is determined dynamic modulus at different frequencies from 0.5Hz to 2.0Hz, then for the other steps axial displacement is changed from 0.02mm, 0.05mm, 0.1mm, 0.2mm, 0.5mm, 1.0mm, 1.5mm, 2.0mm, 3.0mm, 4.0mm, 6mm and 8.0mm respectively, keeping the same type of frequencies. Modulus of deformation values in linear dynamic conditions in the analyzed frequency decreases greatly on clay samples saturated soft with plastic strain values.

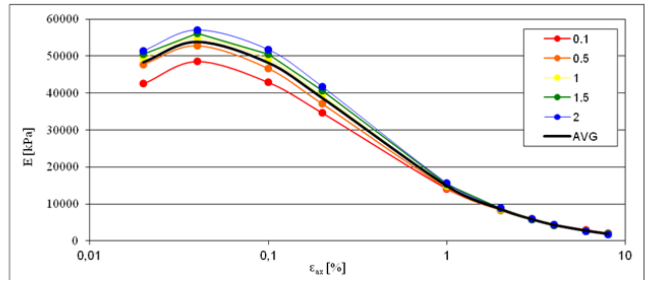

Fig. 13. Mobilization curves for G-2, P6 – 15.0m (F2) (G3, F3, 15m)

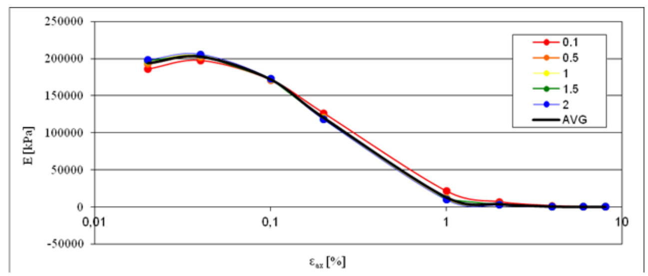

Fig. 14. Mobilization curves for G-2, P9 – 22.0m (F2; F3)

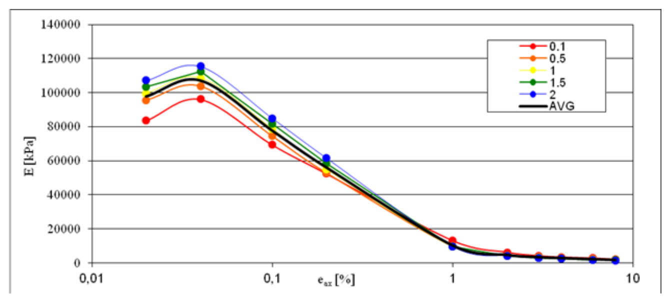

Fig. 15. Mobilization curves for G-2, P10 – 25.0m (F2) G-3, P18-25 (F3)

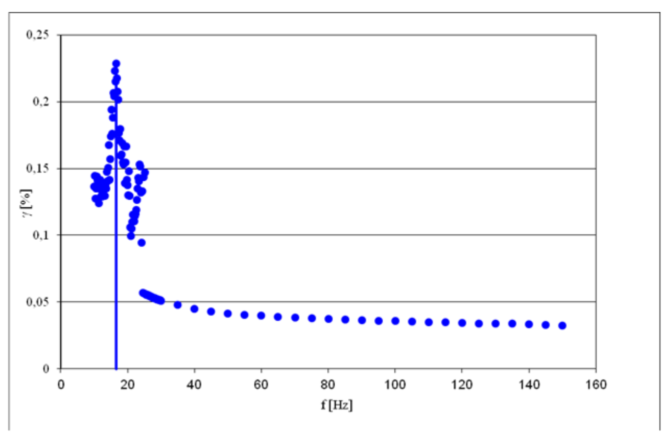

Fig. 16. Resonant frequency for torsion is 16.6Hz for the sample silty clay G-2, P06 – 15.0m; (F2); G-3 (F3)

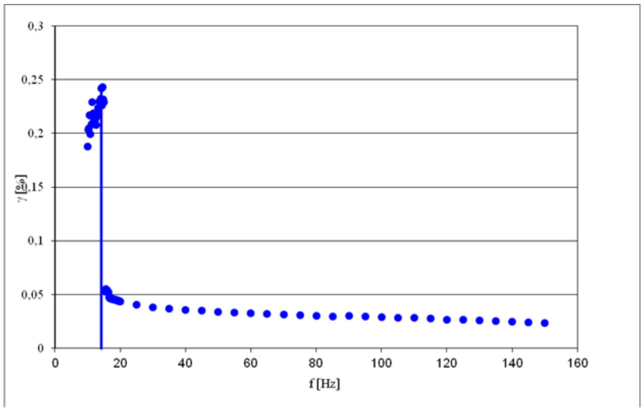

Fig. 17. Resonant frequency for flexure is 14.2Hz for the sample clayey silt G-2, P06 – 15.0m, (F3; G3 – F3)

3. CONCLUSIONS

Given the physical and mechanical characteristics of the subsoil in static and dynamic conditions as presented in the previous chapters, 3 types of foundation is recommended for generator:

a) Direct foundation on improved soil by methods compatible with saturated soft clay with salt water (see Fig.); the minimum height for slab is 5.00 m, bottom slab will be designed as a rigid compensating box with a "skirt" on outline circular for plastic yielding reduced growth areas and rotation. The wind tower foundation can be optimized by making "slurry" wall with thickness of 80 cm on circular diameter contour ~ 20 m to 20 m depth before land improvement. Also this kind of improvement reduce the pressures on soil and increse the active surface from 54% for direct foundation without skirt to 70% for direct foundation with skirt as it can be seen in Fig. 17.

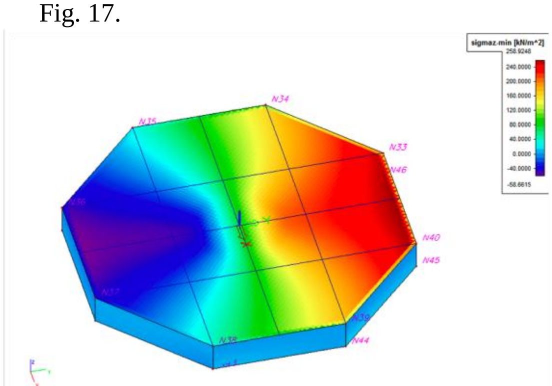

Fig. 18. Pressure under foundation with skirt.

a) indirect foundation through a system of piles - large diameter piles connected by a slab designed as a rigid compensating box for reduced deformations.
b) Mixt foundation: piles foundation on improved soil.

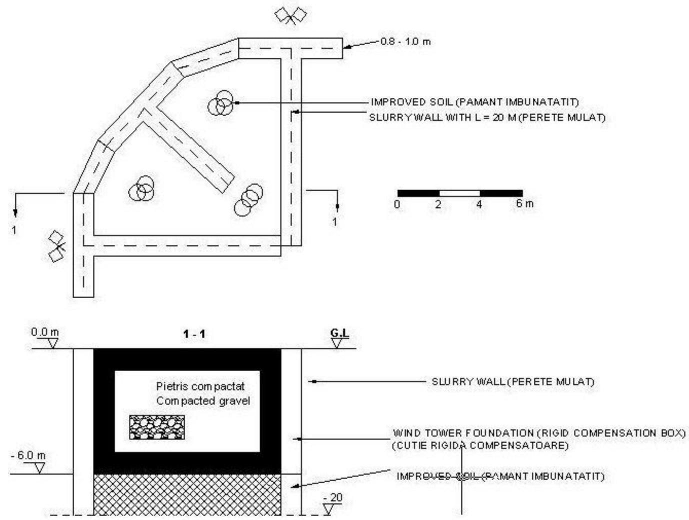

Fig. 19. Direct foundation details on improved soil

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