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Behavior of Direct Tower Foundation on Improved Soil

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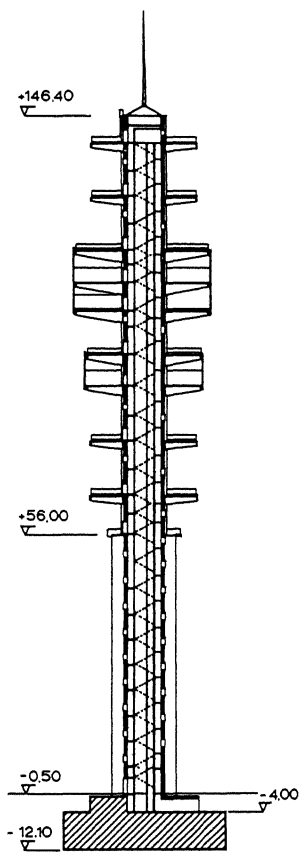
YNOPSIS The paper presents the foundation behaviour of a 146-m telecommunications tower situated ear Verona in Italy, with foundation soils characterized as a thick deposit of alluvial gravels nd sands. In order to limit total and differential settlements of the tower it was decided to mprove the foundation soil with jet-grouting columns. Predictions on the behaviour of the tructure and the foundation soil were performed with an axisymmetric finite-element model dopting an elastic linear constitutive law. Soil moduli were obtained from the results of a ross-hole test and from Standard Penetration Tests; jet-grouting columns behaviour was estimated rom load and borehole tests. Calculated settlements in the construction stages are compared with he measured ones.

NTRODUCTION

'he paper presents the foundation behavior of a 46-m telecommunications tower near Verona in orth Italy. The tower has a diameter of 14 m at he basis and 10 m at the top (fig. 1).



ig. 1.

The foundation of the building has a square section of 36 m x 36 m and lies at a depth of 12 m from ground level. Maximum actions involve a vertical load of 630.27 MN, including the weight of the foundation itself, and a moment of 777 MNm due to wind. Stresses transmitted to the soil have an average value of 0.52 MPa and a maximum, in worst conditions, of 0.62 MPa. In order to limit total and differential settlements, it was decided to consolidate the foundation soil with jet-grouting columns down to -26 m from ground level. 450 columns were arranged according to a 1.2 x 2.4 m grid. Figs. 2 and 3 show the cross-section and distribution of the columns in the treated soil. Three rows of touching jet-grouting columns going down to -17 m were arranged along the perimeter of the foundation, in order to support the walls of the excavation.
Predictions on the behaviour of the structure and the foundation soil were performed with an axisymmetric finite-element model adopting an elastic linear constitutive law. The volume of treated soil below the foundation footing was considered as a single equivalent material with an elasticity modulus obtained by imposing that no slip should occur between soil and jet-grouting columns.
Soil moduli were obtained from the results of a cross-hole test and from Standard Penetration Tests using empirical correlations, while for the jet-grouting columns estimations were made on the basis of load and borehole tests.
This paper presents the results of field, control and load tests on the jet-grouting columns. Calculated settlements in the construction stages are also compared with the measured ones.

GROUND CONDITION

The foundation soils of the Adige valley (Italy) are characterized as a thick deposit of alluvial gravels and sands with high relative densities. Fig. 2 shows a typical stratigraphic column with the N_{SPT} values at the side, measured with a conical tip in the various boreholes, and the velocities of longitudinal waves V_P and trans-

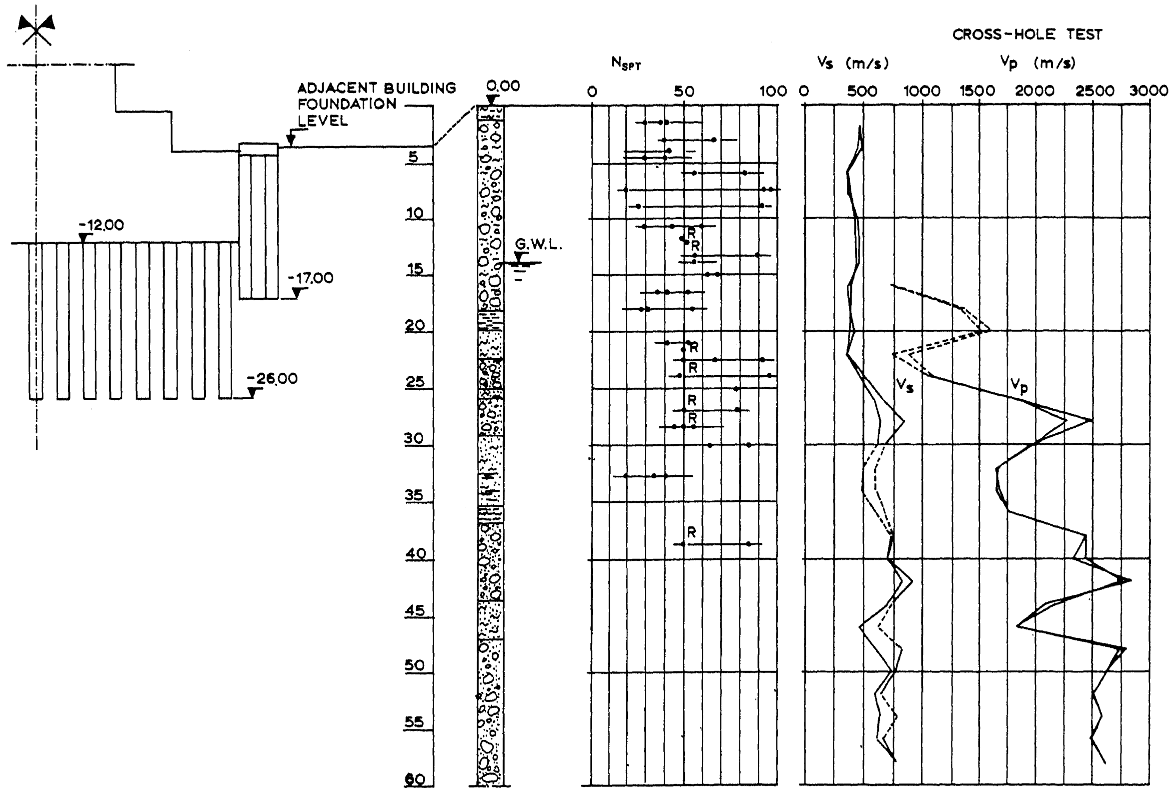


Fig. 2.

versal waves V_S measured with cross-hole tests. Boreholes were taken to a depth of -60 m.

Down to 18.0 ÷ 18.5 m from ground level, a gravel layer was found, mixed with pebbles and sometimes large boulders (maximum diameter 0.2 ÷ 0.5 m), in a matrix of weakly silty sand. The grain-size curves obtained in the laboratory indicated prevailing gravel with sand and the poor silty matrix.

Standard Penetration Tests carried out with a conical tip gave N_{SPT} values on average exceeding 50, thus indicating high soil density.

Using the usual correlations of Gibbs & Holtz (1957), although bearing in mind the reduction due to the use of the conical tip, a relative density of 80÷90% was hypothesized, as a function of effective vertical stress.

When assessing relative density according to S.P.T. test results, we considered the average of test values in the various boreholes (fig. 2).

From depths of 18.0 ÷ 8.5 m to 19.20 m, a layer of compact clay and clayey and sandy silts (C_U = 100 ÷ 200 kPa) was found, varying in thickness between 1 and 2 m. This soil was of low plasticity (liquid limit W_L = 29 ÷ 37; index of plasticity I_P = 11 ÷ 15), with an oedometric compression index Cc of 0.13 ÷ 0.14.

From depths of 19 ÷ 20 m to 60 m, the maximum depth of boreholes, incoherent soils were found, in mainly gravelly and sandy layers, intercalated with thin layers of sandy and clayey silt.

The deeper layers had high and very high density values, with relative density of 80 ÷ 90%.

Wave velocities measured during cross-hole tests turned out to increase with depth, with average

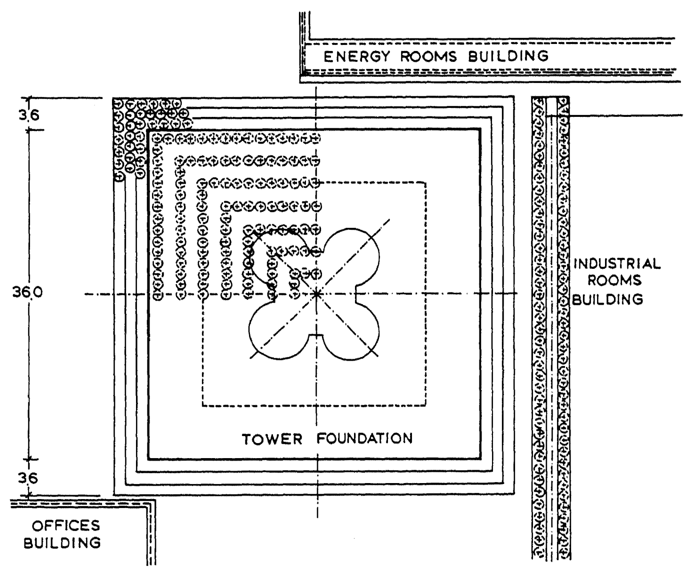


Fig. 3.

values of 350 m/s at the surface and 800 m/s at depth for transversal waves V_S and a range of 800 ÷ 2600 m/s for longitudinal waves V_P.

Permeability tests in the boreholes, carried out in the gravelly and sandy layers down to -15 m, showed very high permeability values, with coefficients exceeding 10^{-3} m/s in the sandier layers and more than 10^{-2} m/s in the more gravelly ones.

Piezometers emplaced in the shallow gravelly layer, bounded between -18 and -20 m by the continuous clayey layer, showed a water-table at about 13.50 m from ground level.

JET-GROUTING STUDIES AND TESTS

The injection system used here was of the mono-fluid type (C.C.P. method), chosen after comparing the results obtained in terms of final diameter and resistance to monoaxial compression achieved with this system and with the bi-fluid system (Kajima method).
The final diameter with the mono-fluid system measured in some test columns was on average higher than the value of 1.2 m predicted during the design phase.
Jet-grouting quality in the foundation soil of the tower was verified by means of continuous boreholes, samples being taken both along the axes of the columns and between them. Samples were then subjected to monoaxial compression tests. The strength values obtained showed a certain amount of scatter, due to the way the columns are made. Values ranged between 5.35 and 28.87 MPa, with an average of about 13 MPa.
A load test was also carried out on one jet-grouting column of the tower foundation. It gave good results, with settlement values of 0.448 mm at a load of 150 t, equal to the working load value, and 0.582 mm at 225 t. Residual displacements were 0.042 mm and 0.1 mm respectively, at first and second unloadings.

NUMERICAL ANALYSIS FOR SETTLEMENT PREDICTION

In order to analyse the stress and strain states arising in the soil as a consequence of applied loads, finite-element models were used during the design phase. Due to the nature and characteristics of the soils in question, a constitutive law of elastic linear type for reproducing the behavior of gravels and sands, was considered to be most suitable. Isoparametric 8-node rectangular elements were used, with 4-point Gaussian integration.
As regards the mechanical characteristics of the soils, S.P.T. and cross-hole test results were used, defining the values of elastic moduli E at the various depths according to:

$$E = S1 \times N_{SPT} + S2$$

where constants S1 and S2 are functions of the type of soil (Denver, 1982; D'Appollonia, 1970). From a preliminary calculation of the shear strains in the soil, which varied from 0.05 ÷ 0.1% on the surface to 5×10^{-3}% at depth, the values of the soil elastic moduli were selected at 15 ÷ 30 % of those obtained from cross-hole tests at strain levels of less than 10^{-3}% (Battaglio & Jamiolkowski, 1987).
On the basis of these tests, two linear variation laws were chosen for soil modulus E, the first valid from ground level to -12 m, the second from -12 m downwards (see table I). As regards the behavior of the jet-grouted area, some considerations are necessary in order to attribute moduli suitable for representing its behavior (Borsetto et al., 1991).
Fig. 4 shows a sketch of the jet-grouted area, where Ej and Aj are the elastic modulus and jet-grouting area, and Es and As are the same parameters for the soil. At is the total area and E^* the equivalent modulus referring to overall behavior.
The equivalent modulus was obtained by imposing the equilibrium and compatibility of the system shown in fig. 4, loaded at the top.
The values of the elastic moduli and Poisson's coefficients of the various materials are shown in table I. The geometrical sketch is shown in fig. 5.

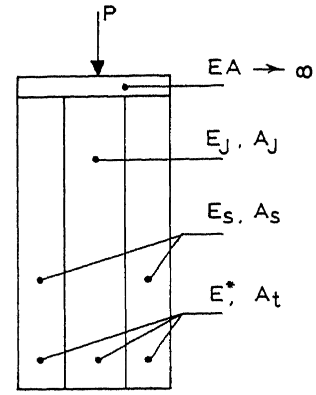


Fig. 4.

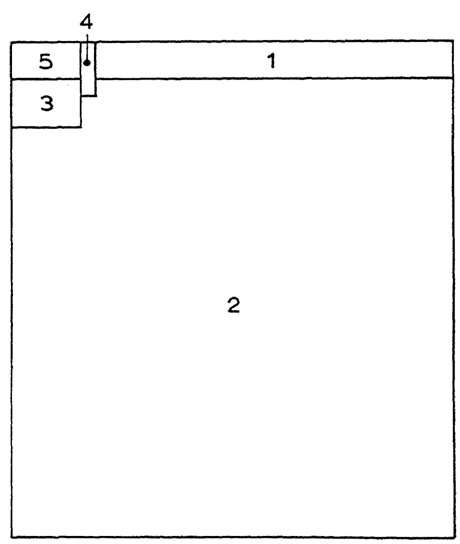


Fig. 5.

TABLE I.

	mat.	E (MPa)	μ	range
shallow soil	1	50+7.14 z	.30	0 ≤ z ≤ 12
deep soil	2	50+10(z-12)	.30	z ≤ 12
foundation j-g*	3	2000	.25	
perimetral j-g*	4	2000	.25	
concrete	5	20000	.20	

* j-g = jet-grouting

Axisymmetric analysis was used to study the behavior of the tower foundation, approximating the square base to a circular equivalent and considering the treated soil as homogeneous. Fig. 6 shows the calculation mesh. Two load conditions were considered (see table II).

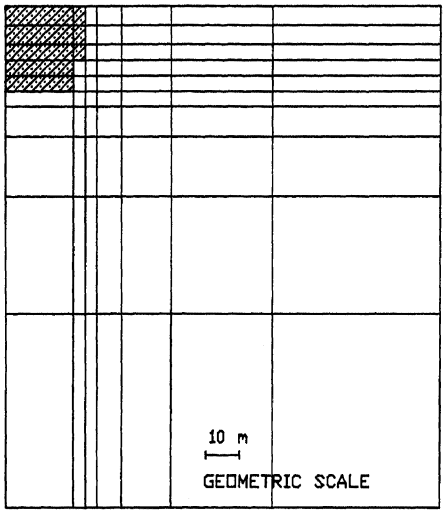


Fig. 6. Adopted Mesh

TABLE II.

load condition	N (MN)	M (MNm)	σmin (MPa)	σmax (MPa)
1	630.27	0	0.486	0.486
2	0	777.0	-0.0999	0.0999

Load condition 1 only includes the vertical load component, including foundation self-weight. Load condition 2 corresponds to the maximum moment due to wind.
The results of the two load conditions were then overlapped, as the analysis was carried out using an elastic linear model.
Finite-element analysis of load condition 1 showed maximum uniform settlement of 24 mm over the whole base of the foundation (see figs. 7 and 8).
In order to analyse load condition 2, which was asymmetrical, Winkler's modulus Kw was obtained from finite-element analysis with uniform load:

$$Kw = \frac{q}{\delta} = \frac{0.486}{0.024} = 20.2 \text{ MN/m}^3$$

We then considered a very stiff beam on elastic soil subjected to loads given by the sum of conditions 1 and 2. Obtained settlement values were: maximum 28.6 mm, minimum 19.4 mm, and differential 9.2 mm.
The sides of the foundation are 36 m, so that the inclination is 2.555x10-4 rad; rigid rotation

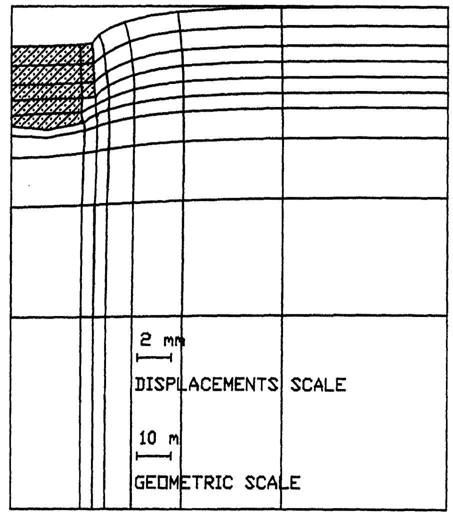


Fig. 7. Deformed Mesh

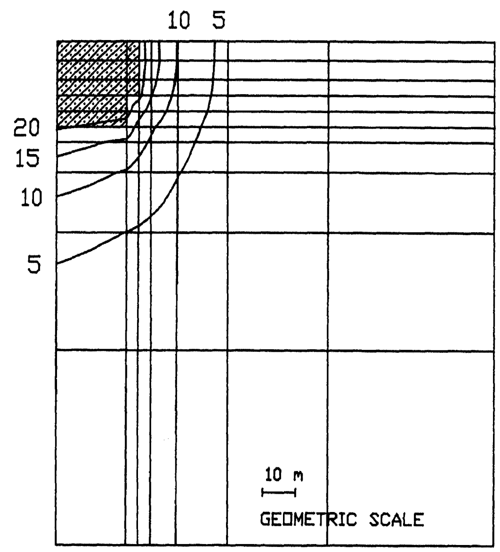


Fig. 8. Vertical Displacements Contours (mm)

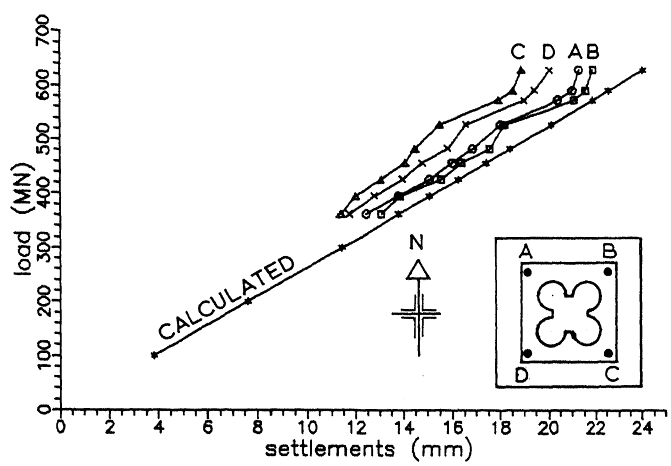


Fig. 9. Measured And Calculated Settlements

causes the top of the tower to move by about 38 mm.
Settlements measured during tower construction are shown in fig. 9, together with settlements predicted during the design phase.
As may be seen, foundation behavior turned out to be good, with total settlements ranging between 18.8 mm and 21.8 mm and differential settlements of 3 mm. There is a good fit between measured and predicted values.

CONCLUSIONS

Soil improvement operations by means of jet-grouting columns allows settlements to be contained within quite small limits, considering the magnitude and size of acting loads, although on soils with good characteristics.
The settlement calculation model used during the design phase and the soil compressibility parameters gave satisfactory predictions of foundation behaviour.
The almost linear trend of settlements measured with increasing loads confirmed that the choice made in this case to adopt a constitutive law of elastic linear type was correct.
Measured differential settlements were very small (≈ 3 mm) and were mainly due to stratigraphic irregularities and stress induced by the foundations of nearby buildings.

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