General report/Discussion session 16: Foundation of transmission towers

Rapport de spécialistes/Séance de discussion 16: Fondations des tours de transmission

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INTRODUCTION

During the last decades, the progressive development of high voltage lines, built along hundreds and hundreds of kilometres, has been in continuous growth to satisfy the increasing demand for electric energy.

Due to the fact that the powerful hydroelectric and nuclear power plants are situated far away from the hub, it is necessary to lay cables to carry the electric fluid to high voltage -220, 500 or even 1,000 Kv-, which need, in their turn, to be supported by special structures. The regional distribution of low voltage -33, 132 Kv- requires, in its turn, other proper structures, the characteristics of which are often conditioned to prior locations.

Anyway, the foundation design must be carried out, more often than not, without a full knowledge of the subsoil conditions, as a consequence of the limitations in the purpose and extension of the available geotechnical investigations.

The geotechnical engineering disposes, at present, of large information concerning the engineering characteristics and properties of the soils and rocks obtained during more than 50 years of site and laboratory investigation.

At the same time, the design procedures and mathematical resources nowadays available make it possible to perform an approach to the problem in order to predict, accurately, the structural behaviour, and, for the geotechnical engineer, in particular, to estimate, fairly well, the soil-structure interaction.

Nevertheless, to successfully solve, from an engineering point of view (that is, low cost and adequate technique), the problems arising from the foundations of long electric transmission lines, require considerable efforts. However, these kinds of problems are not a novelty, since they are also present in and are common to every lineal construction as roads, railways, channels, pipelines and so on. The design criteria are, consequently very much in need of the proper engineering judgement as well as of the cautious handling of what has been denominated "calculated risk".

It is essential, in this case, for the engineer who is in charge of the investigation both in the field and in the laboratory, and who will carry out or supervise the analysis and design of foundation structures, to bear a deep knowledge and a vast geotechnical experience. The financial aspects included in the item "Foundations" play a very important role within the total cost of a transmission line. Under favourable conditions, the incidence upon the total cost is not less than 7 to 10%. However, it is frequent that in particularly difficult areas this percentage may reach 15%, 20% or even more. In fact, in some singular locations, the cost of the foundations has almost been the same as that of the towers. If the uncertainties resulting from a poor knowledge of the geotechnical characteristics are considered, the conditionings that arise upon the exact fullfilment of the schedule and programs of work previously stated to successfully carry out the constructions should also be understood. (43, 48, 57).

TYPES OF LOADS AND SUPPORTING STRUCTURAL ELEMENTS (25, 56)

The supporting structural elements vary according to the type of line. Those exclusively used to support cables are called "Suspensions" and they are greater in number in a transmission system. They must stand the weight of the cables and spare parts, the action of the wind, ice and eventual earthquakes.

Along the line, structural elements, capable to resist -besides the above mentioned- the efforts derived from the construction and assembling, temperature, eventual breakage of cables and/or direction changes, are distributed sequentally in less number. They are called "strain towers" and they transmit the major loads to the foundations.

The dead-end structural elements are situated at the exit or entrance of out-door-substations. These elements, together with angle towers, are subject to important permanent horizontal loads besides the accidental loads before mentioned. Special supporting towers are required for deep valley crossings in mountainous regions or big flatness rivers, which may demand major foundation structures.

Poles pulled down in blocks of buried reinforced concrete are used for 33 and 132 Kv voltages; generally, the embedded length is nearly 10% of the total length of the pole. Up to 33 Kv and simple tern, it is common to find wood poles. For greater voltages and double tern, reinforced concrete poles are used, compressed or centrifugated, which are prestressed in accordance with the structural needs; also metalic poles are used, which join the foundation through proper clamps.

Individual poles are used as suspensions; for common "strained" or angular structures, two

poles in ridge are used; for important angle changes or dead-end elements, three poles in tripod are employed. Generally, the foundation is a unique mass. Occasionally, for angle changes, individual metalic poles or four-legged lattice towers are used.
For 220, 500 Kv and higher voltages, lattice metalic towers are used, being these two the most important:

* Self-supported lattice towers. The foundations may be built with isolated footings or blocks, one for each leg, or individual piles, or groups of piles with a pilecap per leg. Each leg transmits to the foundation compression and uplift forces, and horizontal loads.
* Guyed towers, made of two metalic oblique columns, which lay on a footing or pile cap, transmitting mainly compression forces. The horizontal loads are held by four guys joint to footings, plates, blocks or metalic or reinforced concrete deep anchorages.

As it is easy to understand, according to the relation between the magnitude of the loads and the resistance and deformation characteristics of the soils or of the existing rocks in each specific location, fairly deep foundations shall be required, made up by footings, isolated bases, unified bases, individual piles, groups of piles, cylinders, anchors or groups of anchors combined with concrete caps.

GEOTECHNICAL INVESTIGATIONS (20, 22, 27, 30, 33, 39, 59, 66)

The full reach of the subsoil investigation is restricted to the great number of spots where the supporting structures shall be implanted. Of course, every investigation method available may be used, but some of them are out of question due to their cost or the time demanding their carrying out. In fact, the low and medium voltage lines are disposed along right of ways quite well defined by the requirements of rural or urban interconection.

Consequently, when the field investigation work is required, the possible location of the supporting elements is already approximately stated. Therefore, the eventual subsoil characteristics will not modify, save exceptionally, the place where the poles or towers shall be implanted. In these cases, it is the custom to carry out geotechnical investigations every 5 or 10 supportings, and densify the determinations in case important differences between nearby locations occur. The work of a geotechnical engineer with full knowledge of the zone characteristics is highly beneficial in order to minimize modifications to previous investigation programs, or to quickly and exactly know what to do in areas with peculiar characteristics.

When these types of lines are built along zones not having too many details of it, it is greately useful to previously study air-photographs, which enable the geotechnical photo-interpretation of the sites in question. It should be insisted that the right-of-way is often determined, not for the geotechnical conditions, but for the topographic characteristics, access conditions and legal circumstances.

In high and very high voltage lines, in some cases for hundreds and hundreds kilometres of distance, the alignement shall demand, as far as possible, long straight sections. Generally, and before any geotechnical investigation, topographic works are carried out and consequently rather complete photogrametric relief programs exist; this enables the previous geotechnical evaluation for photointerpretation.

As the length of the spans is 400 to 500 metres, to carry out tests "in situ" every two or four towers mean distances of 1,000 to 2,000 metres between the spots. When very homogeneous conditions exist, the necessary exploration spots may restrict to the location of the strain towers and/or direction changes, plus the study of one or two half-way spots.

The purpose of these investigations is to define the mechanic characterization of the soils or of the rocks, to determine the potential chemical attach to concrete or to steel structures, and to evaluate eventual hydric erosion possibilities to the foundations. To state a methodology for the previous geotechnical studies, it should be taken into account the probable geotechnical conditions, the program requirements and the access conditions to the investigation sites.

Eventually, in soils, open pits are carried out, as well as Plate Loading Tests (PLT), Screw Plate Loading Tests (SPLT), Cone Penetration Tests (CPT), Cone Penetration Tests with Pore Pressure Measurements (CPTU), Dynamic Cone Penetration Tests (DCPT), Standard Penetration Tests (SPT), Self Boring Pressurementer (SBP), Menard, Marchetti's Flat Dylatometer Tests (DMT).

Anyway, the purpose is to define a subsoil profile to evaluate later on the foundation soil structure interaction. As it is understood, the skill and all the experience of the geotechnical engineer should be aimed to state a specific study methodology, specially considering the convenience of using those mechanical elements easy to transport and simple to use. At the same time, it is necessary to emphazise the convenience of being able to employ the best reliable correlations available between the results obtained in the field and the values of the engineering parametres which shall be used in the calculations.

It is also important to use field procedures which enable to obtain representative samples of the soils in question for the geotechnical specialists' direct investigation and laboratory tests. Bearing in mind the aforesaid, the authors consider that, for the time being, there is no better methodology than the one including the use of improved and normalized samplers which make it possible, at the same time, to recover a representative sample and to carry out tests of static or dynamic penetration in the field.
The following operations have been successful for soils:

* The use of interchangeable thin shoe samplers with internal liners, both metalic or plastic, which enable the housing of samples in their interior preserving their natural water content.
* The recover of samples every ½ or 1 meter -according to the requirements of the soil profile or acting loads on the structures- carrying out, at the same time a standard penetration test. The result of this test is correlated with the N value of the SPT.
* The determination of ground water to eventually install a piezometer or phreatimeter to enable the study of the variation of piezometric levels.

These operations enable a good description of the stratigraphic profile. It also makes possible, in the laboratory, to correctly determine: a) the natural water content; b) the piezometer levels; c)Atterberg's limits, the granulometric characteristics and, therefore, the soil classification; d) whether the soils are under the free water level, the unitary weights and volumetric ratios.

But for exceptional cases, using these improved samplers with the proper diameter and special sharp shoes of chrome vanadium, sufficiently thin and long, the samples may be employed to carry out shear or triaxial tests, in order to estimate, approximately, the parameters C and Ø, or to state determinations which make it possible to have an idea about the swelling or shrinking potential of the soils in question.

The direct observation of the sample make it possible to see the fisuration degree of highly preconsolidated soils by dessication. The use of bentonitic mud permit drilling without special casing.

When boring in sands under the phreatic level, the penetration test is very useful to apply the correlations available over the eventual liquefaction potential under earthquake effect and, at the same time, to obtain a representative sample for its granulometric analysis.

Of course, if it is necessary, any of the complementary studies considered important may be done. But these shall be not too much, taking into account the usual geotechnical requirements, and in case they are necessary, they can be programmed. Even for loess' case, if the sampler recover samples of at least 3", has long thin sharp shoes, and proper grease is used, the remaining structure in the center of the core shall not be totally destroyed so as to prevent the detection of the potential collapse under saturation in the oedometer or the triaxial chamber. Special care should be taken when evaluating unstable cemented soils with high natural porosity, due to the fact that the sampler, when altering the intergranular bond resistance, does not permit to accurately appreciate the inherent mechanical characteristic of the natural formation; in this case, it should be necessary to excavate open pits and to take out undisturbed samples.

Using this methodology, which is very cheap, a stratigraphic resistant profile can be established, which in almost every case enables to properly solve the questions involved. The field equipment is suitable to bore hard soils or dense sands or fine gravels, and to reach all the depths required in this kind of investigations. It should also be stated that it is easy to adapt it for the use of spoons with a foot valve to be able to efficiently study clean gravels.

When rocks or highly cemented formations are encountered, small rotative drill machines are successfully used. These drills have samplers Ax or Bx which enable to state the litological characteristics and the eventual rock mass structures.

In all the cases, the soil samples and the ground water recovered, enable to carry out all chemical tests, over aqueous extract, or of the soils necessary to be able to establish the potential agressivity to concrete or steel.

Another is the case of the eventual possibility of hydric erosion in the tower placement; in this case the engineering judgement to evaluate "in situ" the possible cases of local erosion is essential. Speaking from a regional point of view, where big floods may not only modify the soil properties in the upper part of the stratigraphic profile but also modify the planialtimetry, it is necessary to gather round hydraulic experienced engineers in these types of events. In this sense, the combined action of big floods and the setting of relief structures near the location of a tower, may produce a catastrophic combination for the stability of the supporting structure. Even the eolic erosion should be considered in very loose fine soils, to state depths sufficiently safe for the foundation.

When the foundations are built upon bases or blocks, they are in general not deeper than 3 meters; consequently, the soils that will provide the lateral or uplift capacity, shall be subject to all the changes that the weather may produce upon them. In the laboratory, the eventual modification of the shear parameters should be checked in each case, when an increase in the saturation degree is produced, and even eventually the modifications that this may cause upon the soil deformation characteristics. These estimations are not only necessary for the design of the bases or blocks, but they are also very useful to forsee the stability of the excavations when carrying out the work, the possible rising effects of the phreatic level, the difficulties or additional requirements that may be necessary for the refilling and good compaction of the soils previously excavated and relocated, and so on. This kind of events, and the programatic and purse consequences, generally raise important differences between the constructors and the surveying engineers, and bring about disagreements between the design engineers, owners and contractors.

The geotechnical data arising from the studies carried out following the above described methodology, enable to estimate the calculation parameters, either through the direct results from the tests or using the correlations available. It is necessary to keep in mind that the most common forces that these structures require are generally caused by the wind, or are the result of working hypothesis of cable breakage, or events while setting; they are, in consequence, short-term conditions. The use of shear parameters in undrained conditions shall consequently be required.

It should be taken into account, nevertheless, that for strain towers, or with direction change, or for dead-end frames, the loads act for a long period of time, i.e. long-term conditions, and the verifications should include the use of the values corresponding to the parameters for drained conditions; for the verifying of guyed structures, a part of the design effort is composed by the permanent applications, since the guys should not be loose but tight.

TIPIFICATION (2,21,50,51)

In long lines, occuring along zones with different geotechnical characteristics, to which different topographic and altimetric conditions may be added, if the foundation design (only from a technical point of view) should be optimized, a vast quantity of different types of foundations can be defined, although they may not be convenient from a global programatic point of view. It is necessary, then, to try to classify, in some way or

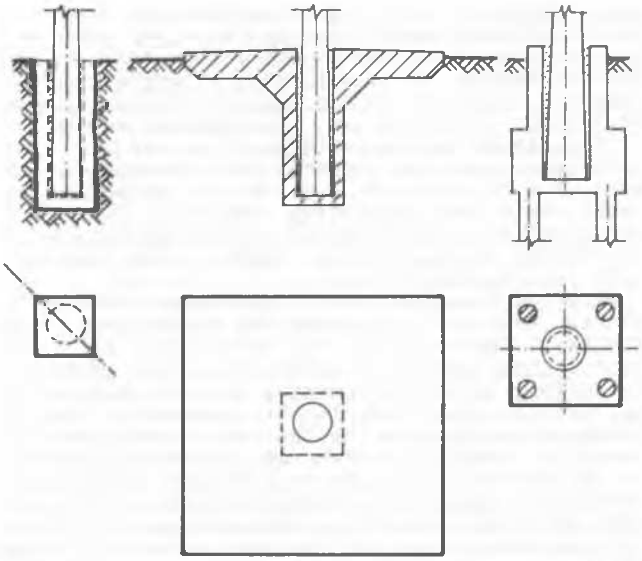

FIG 1 - Pole foundation

other, the types of foundations to be used; this classification may lead to distribute along the line a maximum number of different characteristic foundations. The acting loads and the subsoil properties in each location shall define the type of foundation to be built; each type, therefore, shall cover some combination of these design facts.

Poles

15 to 30 meters high. Lateral loads: 10 to 30 kN. Overturning moments: 200 to 500 kNm. The foundations are generally formed by concrete masses or bases with upper or bottom enlargement; according to certain local conditions, these bases may be reinforced with steel bars. When the subsoil conditions are very much unfavourable, one can turn to the following indirect foundations:

a) A large diameter pile or a reinforced concrete cylinder;
b) A pile made up by a steel metalic casing, sunk, open or closed in the bottom, partial or fully filled with concrete;
c) A group of conventional piles with a special pile cap, where the pole is;
d) A group of non-conventional piles, built with the modern techniques of soil or jet grouting (micropiles, pali radici, metalic pipes with postinjection, cCCP, etc.)

The use of metalic poles in high voltage lines, with heights of 30 to 50 meters and moments of 5000 kNm, leads to the economic use of large diameter or reinforced concrete piles.

Self-supported lattice towers

In each of the four legs in suspension towers, compression or uplift forces of 300 to 500 kN and lateral forces of 60 to 80 kN generally take part; in strain towers, the efforts may considerably be greater. Isolated bases of simple and reinforced shaft concrete, blocks, unified bases and mats are used as foundation. When soils are of very low resistance, conventional piles, large diameter piles, piers

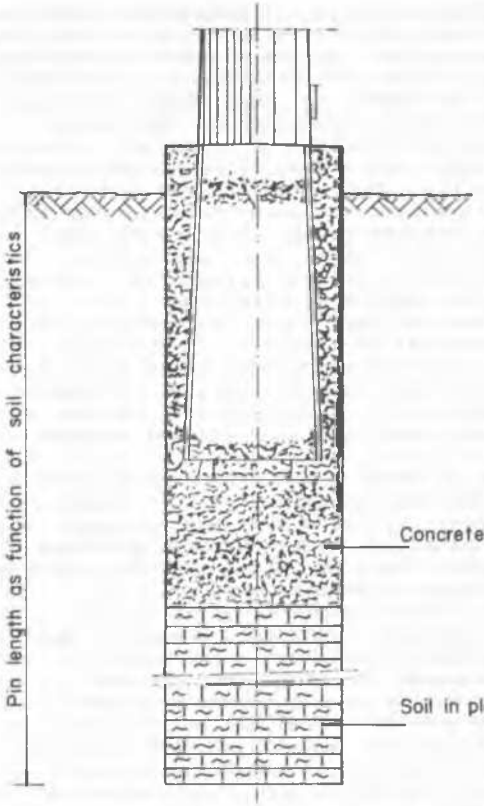

FIG 2-Pole foundation. Driven pier with direct sealing of the support. (Ref. 21)

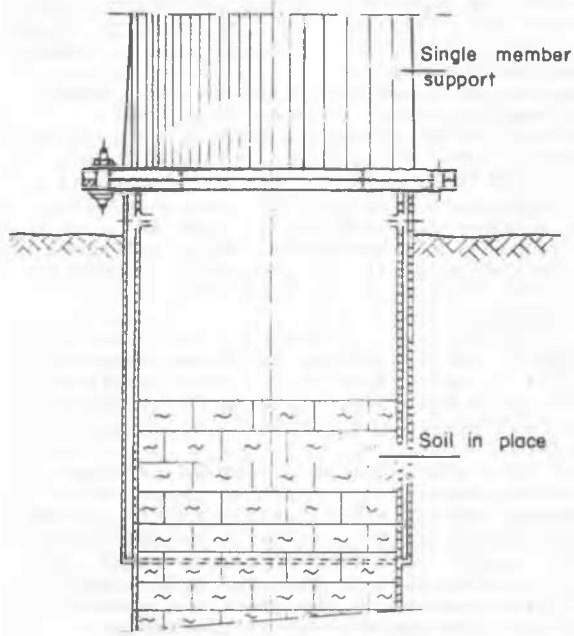

FIG 3-Pole foundation Driven pier with cap. (Ref. 21) or caissons are used. These piles are built isolated -sometimes it is necessary to brace them-, or they are placed in groups under each of the legs. When river crossings occur, in some cases the foundations of the towers may be large and expensive, either for the extent of the spans or as a consequence of the hypothesis of local erosion which is necessary to take into account as regards each of the foundations.

Guyed Towers

The oblique columns laid on central bases or blocks transmitting forces of 400 to 600 kN. The guys transmit to the foundations uplift forces of about 300 kN. They are connected with concrete or metalic anchors, buried plates or steel grillage.

DESIGN AND CALCULATION OF FOUNDATION STRUCTURES (5, 17, 18, 38, 40, 41, 53)

A. Shallow Foundations (17, 18, 19, 28, 29, 35, 52, 53, 58, 60)

Poles

Lateral and vertical forces are relatively small; overturning moments are the determining applications. It is important to limit the pole rotations not only from the stability point of view but also form the aesthetic one; the rotation approximately 1° is usual as maximum value. Sulzberger's over 60 years' analysis method, is still widely used. This method takes into account the soil reaction values (both lateral and vertical), to calculate the geometry of the mass necessary to be able to balance the acting moments within the allowable rotation order.

Usually when calculating, a spring model is used; the constant of the spring depends on the coefficients of the lateral and vertical soil reaction (k_h and k_v). When FEM is applied, the medium is modelled, and the stressed and deformations are calculated; besides the fact that to obtain a satisfactory mechanical model is troublesome, another important question is to properly appreciate the changes that during the structure living life the distortion and resistant parameters may be altered. The variation may be quite notorious in the upper 2 to 3 meters.

Pole block foundation

When the foundation conditions are abruptly modified, improving the soil resistance, at a relativelly shallow depth, the masses before described (but with an enlarged base) may be convenient to be used; the main reactions are applied on the bottom. For these cases, the general formula of bearing capacity are applied, such as Brinch Hansen's, with safety factor to limit rotations. On the other hand, the presence of a superior suitable crust where thick deposits of soft clay normally consolidated exist, enables the use of enlarged inverted bases, which take advantage of the better conditions existing in the upper part of the soil profile.

In these cases, even though one must resort to the use of relativerly big plates, quite good advantages are obtained if compared to the alternative to use piles. The calculation

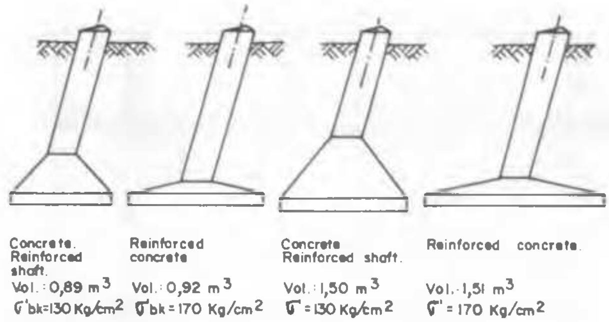

FIG 4 – Shallow foundations

method generally employs the spring method and the k subgrade reaction coefficients.It depends on the thickness of the superior plate, or the inclusion or not of rigidity nerves, whether reforcing steel bars be used or not. Obviously, if the partial replacement of natural soils by others of better quality which may substitute them, or by natural compacted soils is stated in the design program, these circumstances should be taken into account when calculating.

Numerous calculation methods commonly used exist both for the blocks and the foundation bases, even though the resultings for identical elements may often be quite different. (53).

Foundation blocks for self supported lattice towers under compression

The lattice tower leg supports and those for the central masts of the guyed structures are subject to compression forces and to the action of the lateral forces. These bases are calculated employing the general formula of bearing capacity, which include the lateral forces and moments, the peculiar ground surface configuration and the eventual base and ground inclination. The differential settlements accepted for lattice towers are similar to those used for likely metalic structures; on the other hand, they may be bigger for guyed towers.

Foundation blocks for self supported lattice towers under uplift (14,. 15, 16, 64, 24, 32, 38, 40, 41, 44, 45, 49)

The uplift forces generally occur with a very little inclination, so, from a practical point of view, as regards calculation, a horizontal base and vertical uplift load may be taken into account. The reaction is determined by the base own weight, by the weight of the acting soil upon the plate base, enclosed in the so called "failure figure", and by the adding-up of the shear resistance developed on the boundary surface when calculating. If a plate of width B and length L, placed at a depth D, under an uplift force T is considered, a mass of soil may be imagined as a paralellepipedo moving with the soil; along its lateral surface, the soil can be considered in a plastic equilibrium state. If W_o is the own plate weight, we may write:

$$P_{rot}: \gamma . D . N'_q + cN'_c = \frac{T - W_o}{B . L} = \gamma . D [1 + (\frac{B}{L} + 1) k tg\phi \frac{D}{B}] + 2 (\frac{B}{L} + 1) . \frac{D}{B} . c$$

The true failure figure is not a paralellepi-

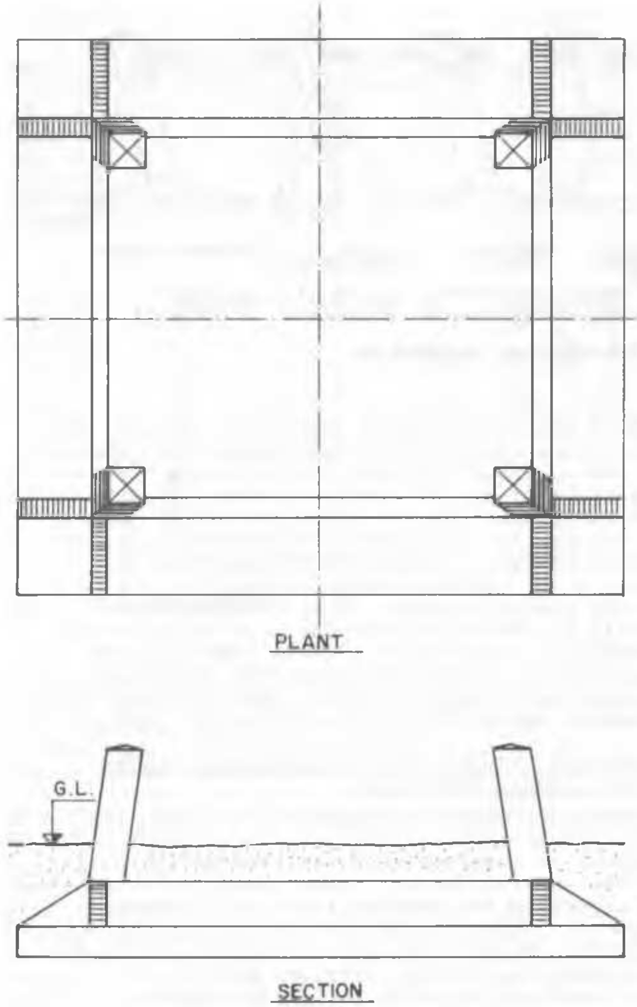

FIG 5- Mat foundation

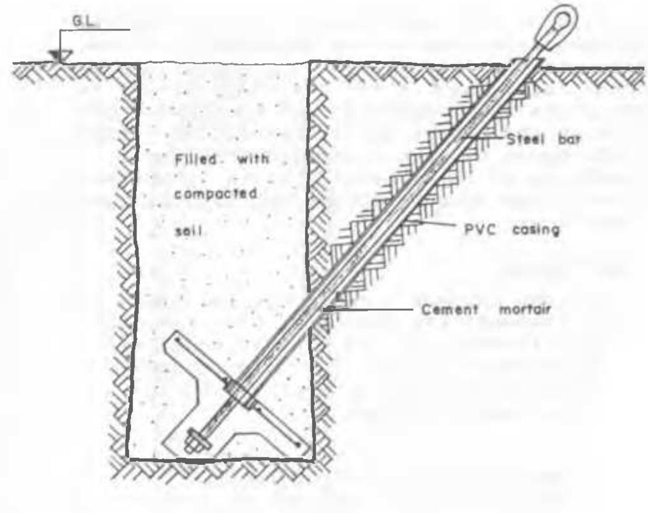

FIG 6 - Guy foundation Inclined plate.

pedo, whereby a proper shape factor should be introduced to take into account the depth effect. Nevertheless, for a relatively D/B low ratio, as is usual in this kind of foundations, for instance 2, and if we consider a square base, we would have:

For a purely cohesive soil:
Prot= γ D+8c (Nq=1;N'c=8)

If the analysis had been done in an ideal plastic material, without taking into account the suction effect, the following would have been obtained:
N'q=1 and N'c-1,15(7/4π+1)$\cong$7

For a frictional soil, with $\emptyset$=35°c'=0,K=1 we have N'q$\cong$4

If a shape factor had been considered for this D/B ratio, the value would have been not too much larger than 4. Consequently, for this kind of foundations of relatively reduced depth of foundation, the analysis that a failure figure assimilated to a paralellepipedo simply employs, enables to obtain reasonably reliable results, always on the safe side. Whenever the pull-out force is oblique, several theoretical and experimental results are available to evaluate the anchor element resistance. On the other hand, for relatively large depths, the soil volumes involved in the limit balance do not reach the ground surface; in this case, the uplift capacity factor values correspond to deep foundations. The D/B ratio has been determined on model-tests; an expression such as $(D/B)_L$=[exp.($\frac{1}{2}\pi+\alpha$)+tg$\emptyset$+senα)]/2cosα, with a ponderated value of α=(π/4+3$\emptyset$)4 enables to obtain results quite coherent with the experimental values.

The experimental analysis, the conventional stability calculation and the modern applications of electronic calculation used in the resolution of finite elements, in elastic and plastic media, is making it possible to reasonably move towards an investigation on the pull force for several stated boundary conditions and different values of γ,c,$\emptyset$,v and for parameters derived from the stress-strain-strength-time relations. The analysis may include the action of transient loads -behaviour in undrained conditions- and of permanent loads -behaviour in drained conditions (31, 54, 55). As the actions in real structures are repetitive, the effects derived from the application of cyclic loads may be considerable (31).

Furthermore, the analysis should take into account the possible effects that the excavations necessary for the base building may produce. In this sense, the presence of free underground water over the foundation level may substiantially modify the hypothesis originally stated in the calculations. Even the absence of phreatic table, the missleading during the refilling soil compaction or the modification of the original levels -bringing along the subsequent modification to the local draining regime of the surface water- may cause very important modifications between the foreseen behaviour and the actual one. This situation may be worse the greater the porportion of the resisting forces derived from the soil shear strength with respect to the soil own weight is.

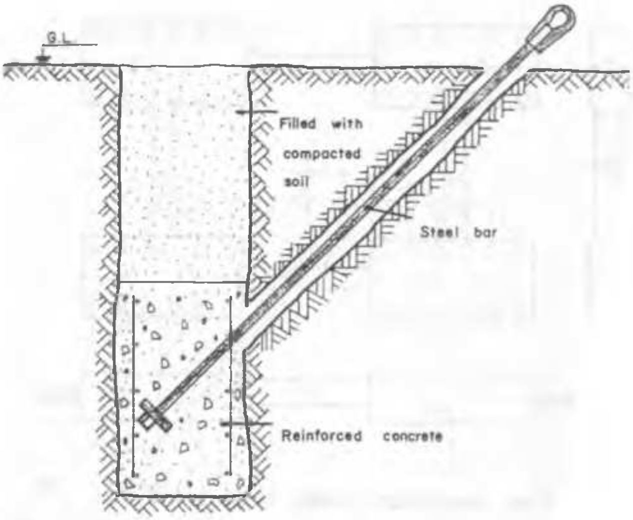

FIG 7-Anchored foundation . Short pile

Anchor plates for guyed towers

The plates connected to the pull out cables are perpendicular to them; hence, the failure figure movilized by the plate is different from the one devebeloped when the plate is horizontal or vertical. The pull-out force angle, with respect to the horizontal plane, is generally 55-60°. If a rough calculation is done considering the horizontal plate, the results will be on the safe side. As it has already been said, the depth effect is notorious. Similar to the case above, for (D/B) 2 ratio, a prismatic failure figure enables a reasonable evaluation of the pull out reactions available.

It should also be emphasized that excavation, setting of plates, soil relocation and compaction actions may carry along considerable changes as regards the behaviour originally predicted.

B. Deep foundations (6, 8, 9, 10, 11, 54, 46, 47, 53, 58, 61, 62, 65)

Pole foundations

Whenever the soil conditions are such that due to the very low resistance in relation with the magnitude of the loads it is impossible to build inverted bases; piles, either isolated or in groups, are used.

For isolated poles, only one large diameter pile may be used. It may be of reinforced concrete and built "in situ" with bentonitic mud; in this case, the upper part should be carried out at a second stage, so that a hole can be dug to house the socket of the pole when it's reinforced concrete, or to house the union welded flange when the pole is metalic (21). As it has already been stated, metalic piles are successfully used, which may be driven into the ground either open or closed. Sometimes, a cement postgrouting is added.

The dimensioning is carried out using the calculation methods usually used to design piles submitted to lateral forces. When conventional (small and medium diameter) piles are employed, three are used at least, and a transference pile cap is built. If they are strain poles, or correspond to angle poles or dead ends, it is common that due to the fact that ridges or tripodes are used, the resulting pile cap might lay on a group of piles. However, it is possible, in this case, to design only one pile for each pole, and to add properly brace to the ground level by means of precast elements similar to those frequently employed to brace the poles one another. The piles may be battered according to the better design solutions; the excavated piles or drilled piers are generally vertical. Whenever groups of piles are used, they may be small piles, and laid with the modern technologies as used in injected micropiles, "pali radici", jet grouting, and so on (12,13),which may add in their interior metalic casings, conventional reinforcement or steel bars that may transfer the compression and uplift loads.

Foundations for self-supported lattice towers

For the unfavourable conditions stated before, deep foundations may be required under the legs of these towers. In this case, the uplift forces may be very considerable and determine the program. Conventional braced piles, either isolated or in groups, large-diameter piles or unconventional anchor piles are also used under this condition. To support long spans, which may be 1,000 meters) very high towers may be required (100 meters), and large diameter or conventional piles may be used. Sometimes, the need to take into account great erosions that may occur, or before great variations of the river levels, the use of piles combined with large frames is advisable. Many times bracing between the foundations of each leg are used; in earthquake regions, the use of piles or braced pile caps are mandatory.

Foundations for guyed towers(4, 37)

The central base is built with one or two piles; for guys, conventional piles, micropiles, or structural elements built following the jet-grouting are employed. In this latter case, a lot of grouting may be saved making the anchors with multiple bulbes. Obviously, the transference of the pull-out force through the bar towards the periphery and anchor cap should be ensured to achieve the interaction with the surrounding soil foreseen in the geotechnical analysis. Sometimes, when the soil is hard, it is cheaper to use conventional anchor bars, either pasive or pretensioned. Another alternative may be the use of helicoidal metalic piles, either injected or not, used as anchors.

PERFORMANCE INVESTIGATIONS (36,42,63)

Mathematical models

The development of calculus through computers has enabled the use of elasticity and plasticity theories applied to the study of the soil-structure interaction. The panorama in this field is broad,only restricted by the difficulties to correctly express the mechanical characteristics of:

a) Natural soils;
b) Replaced and compacted soils;
c) Altered soils during the foundation construction.

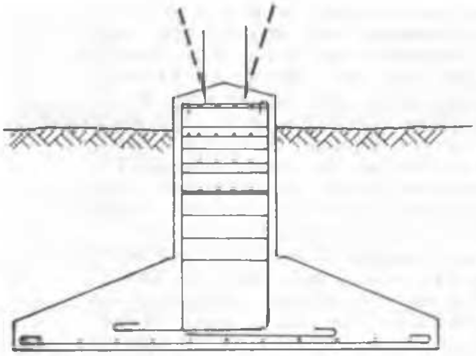

FIG8-Guyed tower. Central base, shallow foundation.

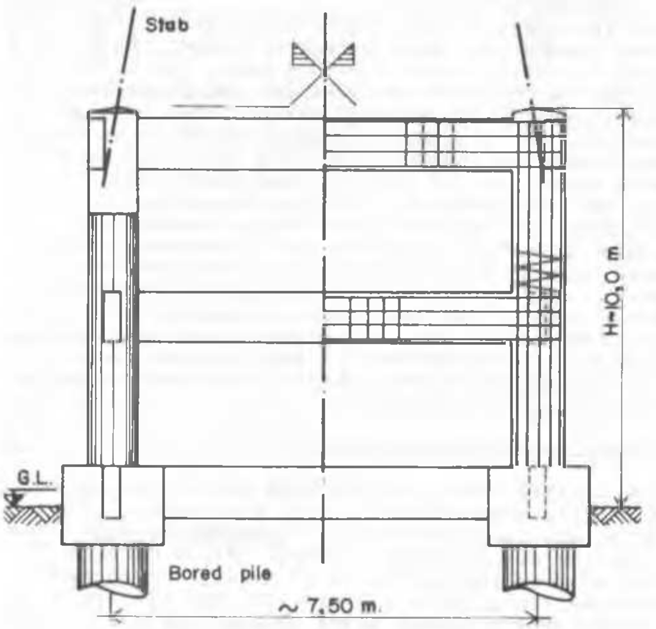

FIG9 - Lattice tower. Deep foundation and structural frame.

<u>Physical models and laboratory tests</u>

Almost all the experience obtained as regards the relative displacement of soils surrounding the foundation, and which determine the so called "failure figures", has been gathered carrying out lab tests in small and medium physical models. No doubt this methodology will continue to be used, since it permits to clarify and even show some features which do not turn out so clear in the mathematical models.

<u>Tests on prototypes</u> (23)

There is an increasing trend to carry out -sometimes massively- field investigation programs over specimens built at a natural scale. This kind of investigation is very expensive, and

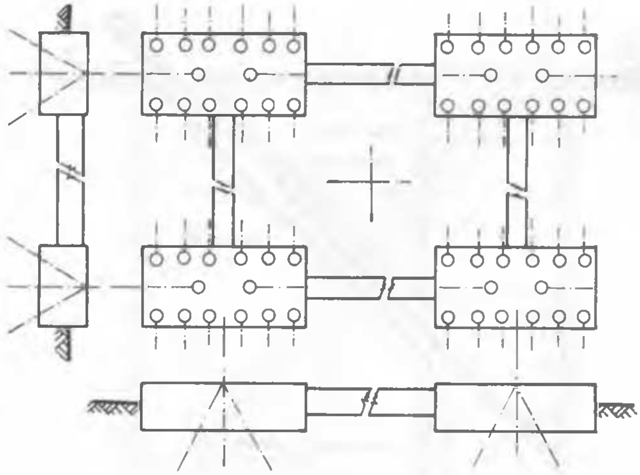

FIG 10 - Pile foundation under large loads

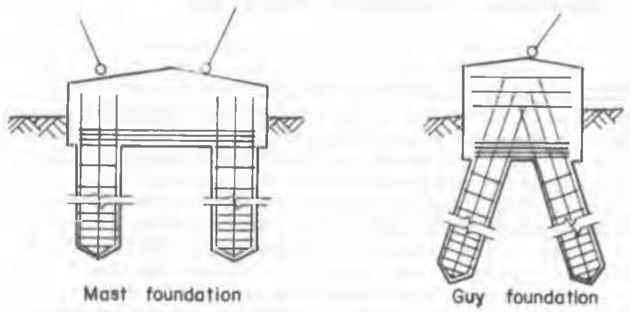

FIG 11 - Guyed tower. Deep foundation.

the corresponding programs should be carefully evaluated before, to obtain the maximum benefit through the interpretation of the results. One of the main difficulties is that they are carried out in one or two predetermined places in order to take decissions afterwards, which affect all the alternatives that may be encountered along a long transmission line. Another limitation is that the investigations dealing with soil mechanics or rocks should be very detailed to achieve the correct interpretation of the performance results. Further-,more,it is not usual to carry out cyclic tests, even though one of the main facts in these structures is the wind. Anyway, when the programs are well prepared, and they include the eventual action of the raising of phreatic level, or floods, they are a very much worthy source of experiences. It is sometimes the only means to prove not only the flexibility of a certain kind of foundation, but also to reveal the most delicate stages in each of the constructive processes. In this sense, field tests on prototypes not only prove the previous hypothesis of soil-structure interaction but also the interaction of the different parts of the resistant structure considered as a whole. Moreover, these tests may show the errors of methods or constructive procedures, the incorrect systematic applications of which may lead to a failure, even when the geotechnical previsions had been satisfactory.

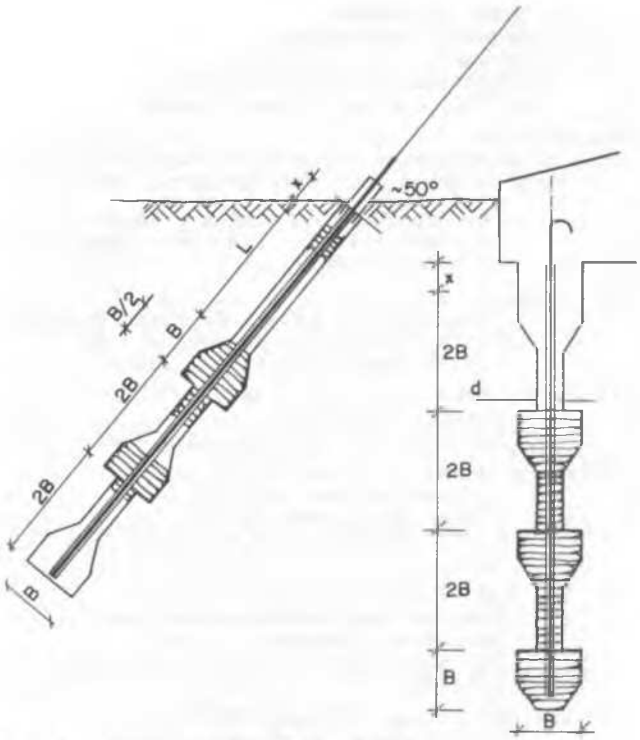

FIG 12-Anchored. foundation. Jet grout technique.

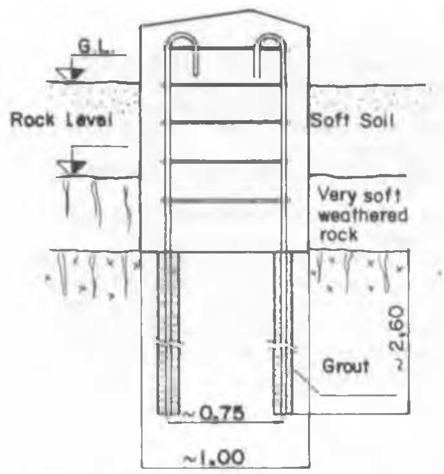

FIG 13- Rock foundation

CONSTRUCTIVE ASPECTS

The constructive proceedings play an important role in the development of calculation and building of foundation structures. The use of bases under compresssion or uplift, or of anchor plates, implies the execution of excavations, the geometry of which should be preserved. This is not always easy, where the water table is near the surface or where its level varies greatly. The geometry falencies, or those resulting from an undercompaction of soils, may affect the global performance. When the resistance proportion considered in the calculus fundamentally derives from the co-operation of the resistance of the soil involved, escessive deformation may appear. In the case of large excavations, where the original soil is replaced and compacted, an unhomogeneity as regards the stratigraphic of the natural soil is produced: this circumstance, and the consequences thereof, should be considered when designing.

The execution of piles and pile caps do not present any other inconvenience not in relation to any other construction. The use of non-conventional piles and anchors demands the careful sequential execution of the constructive stages proved during the carrying-out of tests "in situ" on prototypes. As along a transmission line, regional materials should be used to elaborate concrete and mortars, previous studies and controls during the construction are extremely important to ensure the physic-chemical stability of the materials used. The medium aggresivety of the foundation structure, the attack of the ground water, of the soil salts, of the own aggregates with the cement, the corrosion over metalic elements, etc., has very often caused serious maintenance problems.

SAFETY FACTORS

The safety factors used in Soil Mechanic are applicable for structures under compression. For the pole calculation, where the overturning moments are determining factors and the structural element reacts principally over the lateral surface of the foundation, the shear strength parameters and those regarding distortion should be prudently evaluated. It should always be born in mind that the upper part of the soil profile is being considered, where the weather action may produce very important changes. The soil reaction values commonly employed and the influence of the base size, are generally evaluated with conservative criteria. The special influence that a deffective construction exert over the inclination of poles during service must be emphazised.

When designing bases, plates or anchors under uplift, the resistance generally arises from three factors:

a) The own weight of the structural element;
b) The weight of the soils involved in the pull adhered to the base.
c) The shearing resistance available in the soil.

As regards the own weight of a), a safety coefficient with respect to the failure of 1,2 seems to be enough. As regards the own weight of b), much more care should be taken due to the uncertainty of the real configuration of the acting soil; the safety factor may be between 1,5 and 2. As regards the calculated resistance in c), the safety factor may be between 2 and 3, depending on the information available of the shear strength variation of the natural soil or replaced soils, for extreme conditions regarding weather and floods. These analysis correspond when the maximum structure design loads have been considered. The stability analysis include, of course, the probable methods and techniques applied to the evaluation of loads and soil parameters, but the results must be used with caution.

ROCK FOUNDATIONS

Foundations under compression are generally formed by bases or masses dug in rocks stable enough to interperization. Foundations under pulling forces are generally constructed by:
a) An isolated anchor;
b) A block acting as transference element in anchor groups.

These anchor may be pasive or pretensioned; it is necessary, sometimes, a previously grouting treatment of the rock. A great variety in types of anchors, constructive capabilities and methodologies exist, as well as rules for load tests and generally accepted conditions. The description, analysis and design are out of the scope of this report.

ACKNOWLEDGEMENTS

The authors thank TECHINT, SADE S.A.and DEBA for the croquis used in this report.

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