Strengthening of a structural fill by claquage

Renforcement d'un remblai structural par claquage

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SYNOPSIS Some heavy buildings of a large facility have been built on structural fills. Recording of movements of the foundations has shown the need for some method of reducing long term settlements. From different earth-strengthening procedures, the "claquage" method was chosen for a large scale field test to be performed before applying it to the solution of the problem. Cement-bentonite was used as grout in the fill, creating a network of seams crossing each other to increase the strength of the fill and reduce its compressibility.

Properties of the soil before and after the treatment, results of several laboratory and "in situ" tests, data from a large plate load test with a diameter of 5 m and a detailed survey of the distribution of grout lenses within the fill are reported in this paper. This method of earth strengthening which is presently being applied to the current structural fills at the facility.

INTRODUCTION

The design of structural fills as direct foundations for close buildings when their bases are situated at different levels, is a common practice. Other alternatives usually involve higher costs.

In spite of constructing these backfills with a good and intense compaction and in spite of obtaining high densities, it may occur that, for some type of backfilling materials, subsequent loads on the fill induce creep phenomena or long term settlements that can cause future problems.

This paper describes a method of strengthening earthfills which creates a network of grout lenses, crossing each other in different directions. Injection is performed by means of claquage. This method not only increases the density of the fill material, acting as a preload, but also gives a framework to supplement the soil structure and assist in transmitting the loads to the fill foundation.

To investigate the effectiveness of such a procedure, a large model test of a structure founded on a structural fill has been constructed. It consists of a rigid circular mat 5 m in diameter loaded to the same pressure as that of the actual buildings. Testing of this method includes, in addition to the model test, a detailed survey of the claquage lenses obtained through exploration pits excavated after the injections as well as several "in situ" tests (plate load tests, cross-hole tests, "in situ" densities, etc.) and laboratory tests which, compared with similar tests run before the treatment, allowed for a quantification of the effectiveness of this method.

THE STRUCTURAL FILL

Natural Ground Conditions

The place where the buildings associated with the structural fill were built was originally levelled. It was formed by a thick deposit of heavily preconsolidated soils. Foundations of some buildings required excavations up to depths of approximately 10 to 15 metres whereas others were to be founded close to the grade level. Excavated soils were used as backfilling materials and some buildings have their foundation resting directly on these structural backfills which in addition is of variable thickness.

The natural soil deposit was formed during the Miocene as a product of erosion of granite and gneiss and the subsequent transport and deposition in horizontal layers, as they are found today. The depth of the bedrock (a conglomerate) is about 100 to 110 m as known by seismic refraction surveying and by some deep boreholes drilled through the soil deposit.

The grain size distribution of these soils changes from one level to another as the conditions of climate were variable in the geologic times of their formation. The clay content (particles with a size of less than two microns) is quite homogeneous, irrespective of the other properties, and varies between 4 and 17%, except in the case of very few samples. These soils appear as a mix of more or less fine grains of silica sands and a random percentage of clays. The dominant clay is montmorillonite and there is a significant proportion of mica, with a minor content of kaolinite and chlorite.

The soil deposit was heavily consolidated partly by the load of the eroded overburden and partly by dessication. The water level, although not clearly marked, is some 5 to 10 m below the ground surface. "In situ" dry densities were close to 1950 kg/m^3 near the surface (up to depths of 10 m) and some increase with depth (about 50 kg/m^3 for each 10 m increment in depth) was noticed from laboratory testing. The natural water content of the uppermost soils, of more interest for this study, varied from 7.5 to 19% (average value 13%) depending on the nature of the soil at each level.

The compressibility of the top layers of this deposit was studied through field tests ("in situ" plate load tests on the horizontal and vertical directions) and through laboratory test (consolidation and triaxial tests). On the basis of these testing and taking into account the experience of the behaviour of structures founded on these soils, which are very common in the central part of Spain, it was estimated that, for vertical loads, the soil would respond with a modulus value of approximately 1.5 x 10^5 kPa for loads applied at depths of approximately 5 m and with a modulus 5000 kPa larger for each metre of additional depth.

Construction of the Fill

The use of the product of excavations in this deposit as backfilling material was thoroughly studied by careful "in situ" and laboratory testing as well as the construction of fill areas for testing. The excavated soils were first piled in an attempt to exclude those whose fine content appeared larger. Afterwards, this soil pile was studied by random sampling inside pits and trenches opened within the pile.

The grain size distribution of samples taken from different depths at some fourteen locations within the pile of excavated soils lay within the narrow range indicated in Fig. 1.

The water content was quite close to that of the natural deposit, i.e. from 9 to 14%. Only the external crust of the pile had a lower water content, close to 8%.

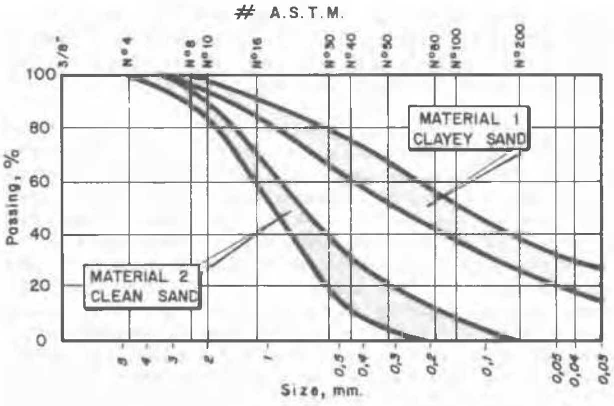

Fig. 1 - Grain-size Distribution of Borrow Materials

The first trial to extend and compact this material failed to achieve the required densities (95% of MP) due to the excessive water content (an average of some 3.5% above optimum). To solve this problem, a method was indicated by mixing these soils with clean sands taken from a far borrow pit, since weather conditions at this location did not warrant the natural dry-out of the clayey sands from the excavations.

The grain size distribution of the clean sands is also illustrated in Fig. 1. They were formed by rounded siliceous particles and their natural water content had an average value of 5.7%. The proportions of the mix were established as 70% natural clayey sands from the excavated soils and 30% clean sands. Results from MP control testing of the mix gave maximum dry densities ranging from 2,070 to 2,130 kg/m^3 at optimum water contents of 7.3 to 8.8%.

Under these circumstances, the construction of the structural fill proceeded according to the specification of achieving at least 95% MP with water contents deviating less than 3% from the optimum. Results of control testing during the construction of the structural fills gave an average value of the dry density of 97.3% of maximum MP and the average water content was 0.4% above optimum.

The evaluation of settlements for the heavy buildings to be founded on this structural fill was performed well in advance when the design of the foundations not included the removal and replacement of natural soils. Under these circumstances, the estimated settlements were of approximately a few centimetres and were acceptable within the limits of future operation. The fact of remolding the natural soils by excavation, mixing, backfilling and compaction changed the compressibility of the soils. On further testing, the estimated modulus of the natural soils, mentioned as being approximately 1.5 x 10^5 kPa, decreased to values of approximately 1.3 x 10^4 kPa (about ten times lower).

Testing after Construction

The excessive settlement of the buildings and the increase thereof over a long period, motivated a very detailed exploration to check the structural fill characteristics. This exploration was very extensive and only a summary of the main findings is given in Table 1.

This comparison shows that the unexpected behaviour of the buildings was not due to a significant deviation from the construction specification.

Consolidation tests, run with block samples taken on exploration pits at different depths (the thickness of the fill varied from 0 to 12 metres), gave similar results as that run prior to construction. A typical result is included in Fig. 2. These tests (a total of 37 were run at this phase of the study) were interpreted on the basis of the Schmertman method to obtain the estimated "in situ" com-

TABLE I - AVERAGE VALUES OF SOME CHARACTERISTICS OF THE STRUCTURAL FILL

	Construction Control	Check after 5 years
Fines (200 ASTM), %	25	35
Liquid limit, %	28	28
Plasticity index, %	10	8.4
Specific gravity grains, kN/m^3	26.1	26.3
Dry density (laboratory), kg/m^3		1930
Water content (laboratory), %		8.4
Dry density ("in situ"), kg/m^3	2020	2050
Water content ("in situ"), %	8.5	7.8

pression curve. The preconsolidation pressure, induced by the compaction force, was estimated as close to 300 kPa. The compression index below and beyond this pressure was C_s = 0.013 and C_c = 0.11, although a considerable scatter was seen in this data (maximum and minimum values of C_s were 0.027 and 0.007 and for C_c were 0.23 and 0.07, respectively).

Cross-hole tests were also run on this structural fill after construction. The most significant values of these results are:

- V_s = 483 m/s (standard deviation 50 m/s)

- V_p = 917 m/s (standard deviation 104 m/s)

THE METHOD OF EARTH STRENGTHENING

Two main buildings at the facility have suffered settlements well over those acceptable for regular operation mainly because of the associated titling towards the zones of the larger thickness of the structural fills. A method to repair these buildings should at least stop future settlements and, if possible, relevel part of the inclination of the building.

Some methods of underpinning were thought of but none was acceptable since they interfered with the operation of the facility which was starting up at that time. Among different methods of ground improvement available (Mitchel and Katti, 1981) squeezing grout into the soil (claquage) was selected for the fill tests since it appeared to be the simplest method and has a good background of experience in Spain (Escario and Rodríguez, 1983; Escario, 1983).

Compaction grouting (Brown and Warner, 1972; ASCE, 1977) was kept as the second choice in case the claquage proved to be inappropriate after field testing.

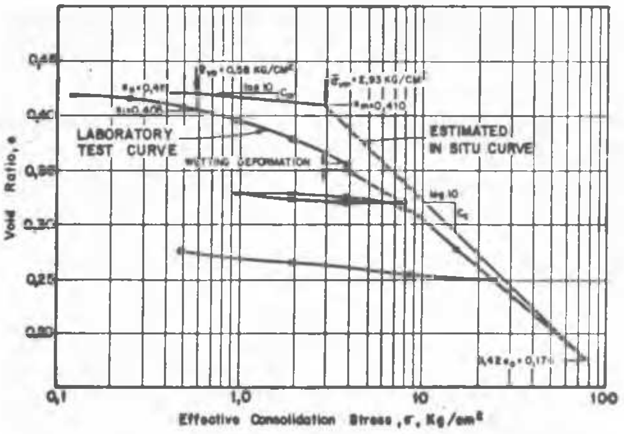

Fig.2 - Tipical Result of Consolidation Test

The injection by claquage (Cambefort, 1977) consists of pressing a suitable mix of cement and bentonite into the ground through a grout pipe with regularly spaced sleeves used one at a time. Breaking the soil with initial high pressures opens a seam through which the grout may propagate under moderate pressure. Control of the grout pressure and the viscosity of the grout mix allows the heave of the foundations to be controlled. The initial pressures necessary to start the claquage process, the opening of the plane of the fracture, the quantities of grout mix to be injected, etc., are parameters whose theoretical determination is not possible now. Only field tests and past experience were considered reasonable bases to design the procedure.

The extrapolation of the observed settlement of the buildings indicated that a future change in volume of the fill of more than 0.5% should be expected if no action were taken. The purpose of the injection should be, at least, to bring about this change in volume and, if possible, to inject more so that the verticality of the buildings may be partly recovered. On the basis of these data, and taking past experience into account, the objective of the grout operation was fixed to inject about 5% in volume of a mixture with the following characteristics:

- Cement	> 400 kg per m^3 of mix
- Bentonite	≃ 6% of cement by weight
- Viscosity (marsh cone)	35 to 70 s
- Rigidity	> 0.4 kPa at 4 h
- Free water	< 3%
- Strength	> 10^3 kPa at 28 days

First, a vertical wall should be injected to contain the foundation filler grout, with a limiting pressure of 100 kPa for each metre of depth.

GROUT TESTING

The preliminary work was performed in an area close to the foundation of the buildings where the thickness of the fill was about 8 m. The ground was covered by 10 concrete mats with a surface area of 2 x 2 m each, and a thickness of 0.40 m, in order to prevent, to some degree, the grout from escaping to the surface. A total of 19 grout pipes were installed and the grouting was carried out in four different phases.

Once the grouting was finished, the grouted soil was excavated over an area of 7 x 3 m to a depth of 5 m and the distribution of grout seams within the fill, their thickness, the grout paths, etc., could be observed and mapped

The main conclusions derived from this preliminary field test were the following:

- Grouting through points at depths of less than 3 m causes the mix to leak to the surface at very low grout pressures. Confinement of the ground surface would be required if grout is needed in shallow areas.
- The thickness of grout seams appears to be very variable, but 5 to 10 mm lenses are frequent. Some planes of the fracture appear parallel to others creating a sandwich type of grout and soil with a thickness of several centimetres.
- A good part of the grout mix is distributed along inclined and subvertical fracture planes although there are many horizontal grout seams of large thickness, particularly close to the ground surface.

BUILDING FOUNDATION MODEL

To model the conditions of the fill below the buildings to be subjected to treatment, a concrete mat was constructed, as shown in Fig. 3. To simulate the building foundation pressures, the concrete mat was loaded by means of seven anchor cables fixed at a sufficient depth in natural ground. Each of these cables was designed for a total capacity of 2,500 KN. The mat was poured at a depth of 2 m (the

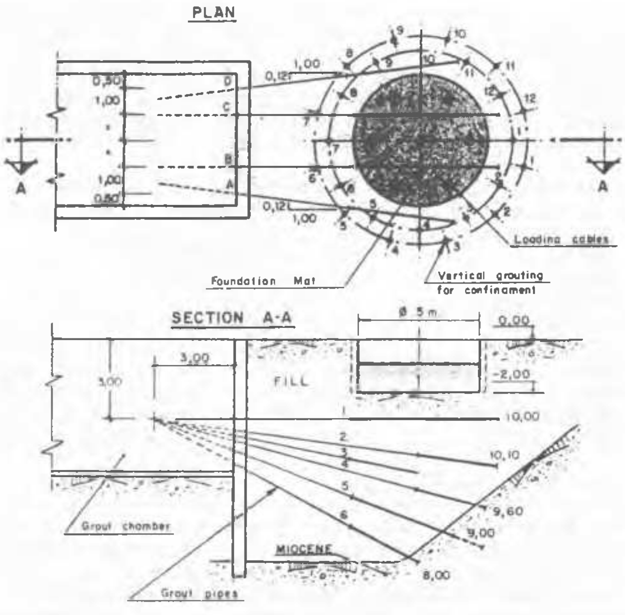

Fig. 3 - Lay-out of Field Model Test

same as the actual buildings) and close to the buildings, resting on the same structural fill.

Prior to any grouting operations, a load was applied to the mat and settlements were recorded by high precision levelling using the same references (anchored at a depth of 40 m in the ground) being used to monitor building movements. Control of movements was followed at several points on the mat surface. The two extreme values of the settlement records (maximum and minimum settlements) are shown as a function of time and load, in Fig. 4.

The first phase of grout was injected to create a vertical curtain to laterally confine the soils below the foundation. For that purpose, a double crown of vertical tubes was installed around the perimeter of the circular model mat. A total volume of 78.07 m^3 of grout was inserted into this vertical curtain wall. During this phase of the grouting operations, the mat was permanently loaded at a constant pressure of 300 kPa by minor adjustment on the anchor cables (whose load changed slightly with these movements).

After that, grout holes were driven from the grout chamber beneath the mat and the pipes with grout sleeves were installed. Injection below the mat was carried out in two phases, separated by a waiting period of ten days. A total volume of 49.3 m^3 was grouted.

Movements of the mat were carefully recorded and are shown in Fig. 5. A small settlement of about 1 mm was observed in the initial stages when grouting the perimeter tubes, but soon the movements were directed upwards. A total heave of 6 mm took place while grouting the vertical grout tubes and an additional 33 mm heave was recorded while grouting beneath the mat. No settlements were noticed due to drilling operations to perforate horizontal holes for installation of the grout tube.

Once the grouting was finished, a waiting period was established to allow the grout mix to harden, after which the mat was subjected to a load test by increasing the load up to 700 kPa and performing unloadin-greloading cycles. Movements during this phase of the model test are shown in Fig. 6. Maximum settlement for this part of the load was less than 3 mm.

INSPECTION AFTER TESTING

To study the effects of the grout on the fill, a test pit of 2 x 2 x 2 m was excavated after removal of the model concrete mat. Excavation was carried out by horizontal levels, each with a depth of 25 cm, so that all grout seams appearing at the bottom of the test pit could be carefully mapped. Drawings were made at each of the nine stages of the excavations, as well as of the vertical walls. The length, thickness and orientation of each grout seam was measured and a computer data bank was created to handle this information. A report on the statistical distribution of grout planes inside the fill is being prepared by the authors. As an example, a sketch of the grout lenses in one of the vertical walls of the test pit is included in Fig. 7.

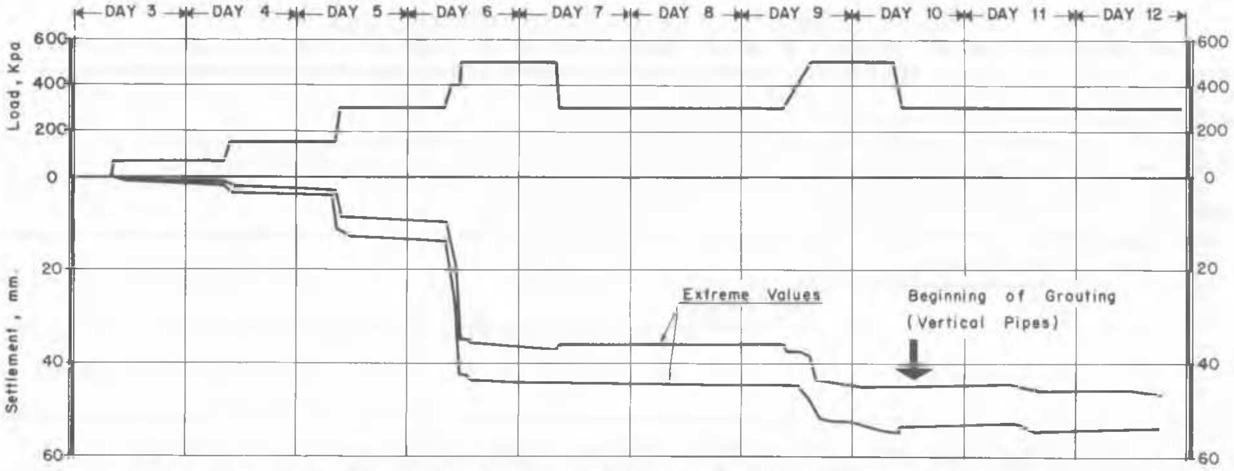

Fig. 4 - Evolution of Settlements. Building Model Load Test. Before Treatment.

The material taken from this pit was analyzed as a sample of 8 m^3. A special grain size distribution test was designed to separate grout material from the fill and, in this manner, it was possible to determine that the weight of grout within the sample was 3.01% of the total weight. The heave of the mat during injection was equal to a change in volume of about 0.5% so that only a small proportion of the grout entered the fill by raising the concrete mat; the main part of the grout had to squeeze the soil and reduce its volume. The dry density of the soil in the large sample was 2097 kg/m^3, so a slight increase was observed over the dry densities prior to grouting. The water content was 11.4%.

Five plate load tests, with a rigid circular plate 750 mm in diameter, were run at different depths at the bottom of the test pit and other exploration trenches within the treated soils. Comparison with a similar test run prior to grouting gave an average modulus which was three to four times greater.

Cross-hole tests were repeated after treatment. The main results were:

- V_s = 628 m/s (standard deviation 46 m/s)

- V_p = 995 m/s (standard deviation 81 m/s)

Similar changes due to grouting were obtained by Serrano and Cuellar (1979) at a different site.

CONCLUSIONS

The work described in this paper is still under way and final conclusions will be obtained when it finishes. However, from the data shown throughout the text of this paper, some preliminary conclusions may be reached.

The method described here is applicable to reducing the settlement of heavy structures founded on structural fills through a rigid mat, even when the building is in operation.

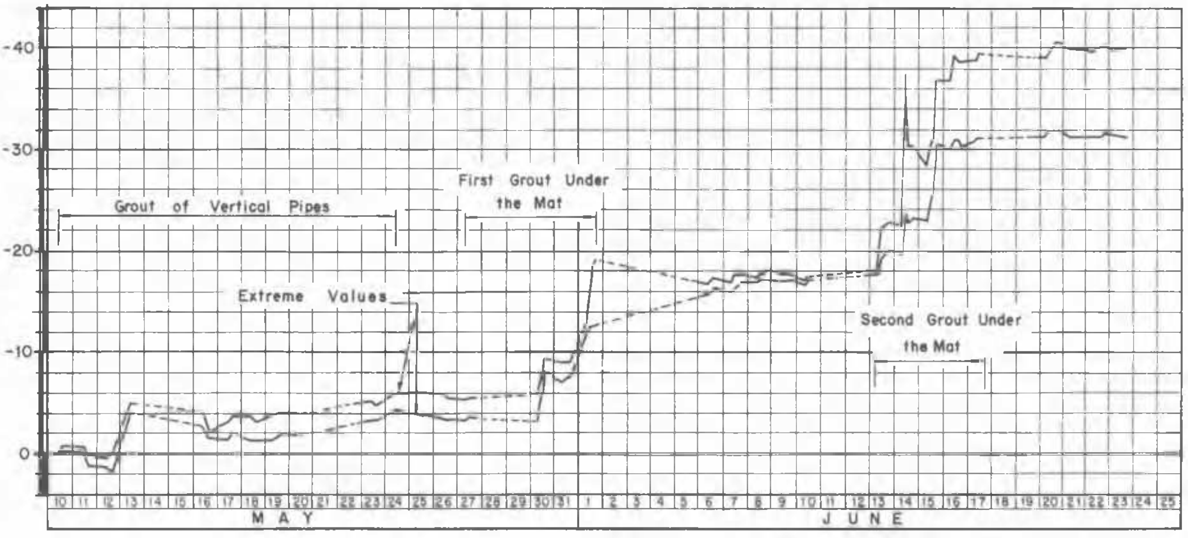

Fig. 5 - Evolution of Heaving. Building Model Grout Test

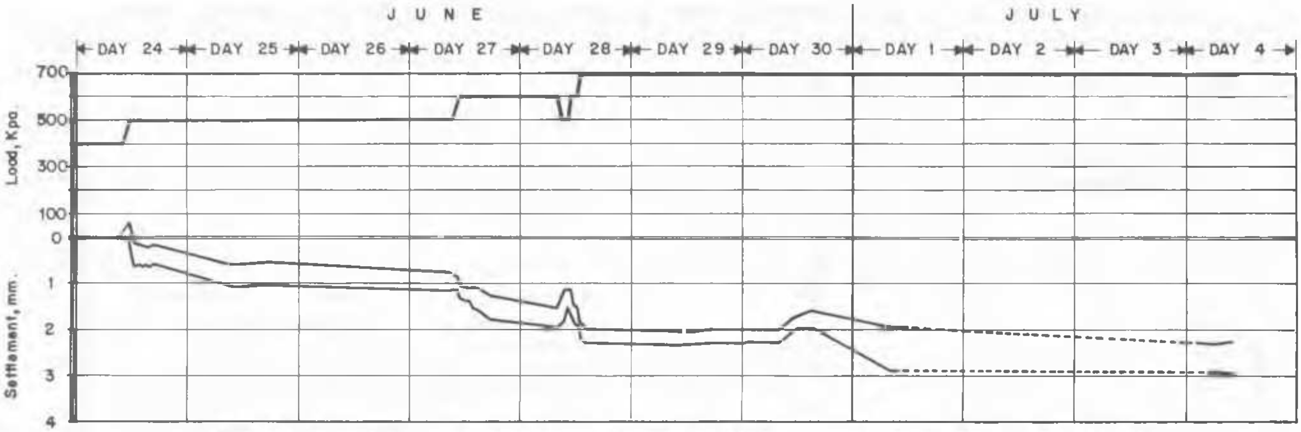

Fig. 6 - Evolution of Settlements. Building Model Load Test. After Treatment

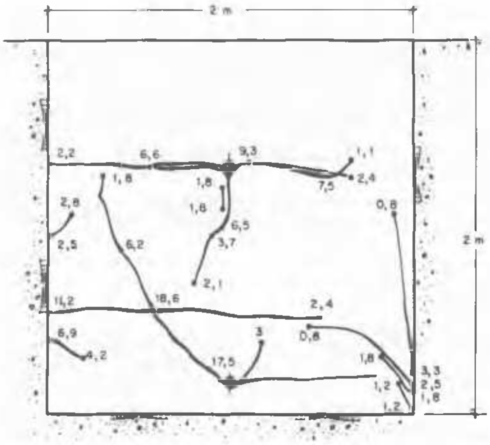

Fig.7 - Distribution of Grout. Vertical Wall of Test Pit. (Numbers indicate grout thickness in mm.)

No settlements are induced by grouting (drilling, washing, injection, etc.) and the heave can be controlled by limiting pressures and grout volumes. The method can be applied in its entirety from outside the buildings without any constraint on the regular operation thereof.

The main effect of the grout is a reduction in the compressibility of the structural fill. This fact is due to an increase in the preconsolidation pressure of the soil and the creation of a network of grout seams, crossing in all directions, which assist in supporting the loads. The reduction in compressibility has been measured as approximately four times for loading new, additional loads.

The method is suitable for soils with a high fine content where the use of other types of grout is not possible. The conclusions of this work are, restricted to soils similar to those where the tests were carried out.

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