ASCE Specialty Conference on Vertical and Horizontal Deformations of Foundations and Embankments at Texas A&M University, College Station, Texas, June 16, 17, and 18, 1994

COMPACTION GROUTING STOPS SETTLEMENT OF AN OPERATING WATER TREATMENT PLANT

Percy M. Wimberly, III,[1] Member ASCE, F. Barry Newman,[2] Member, ASCE, Kenneth B. Andromalos,[3] Member, ASCE, and Chris R. Ryan,[4] Member, ASCE

ABSTRACT: In early 1978, about a month into full-time operation, the new filter plant building for the water treatment plant for the City of Glenwood Springs, Colorado, experienced distress due to settlement. Various subsurface investigations and remedial repairs occurred until early 1985, when the intermittent settlement significantly increased. The City sought proposals to investigate and design remedial repairs to stabilize the building. In early 1987, Geo-Con, Inc., a specialty geotechnical contractor, was awarded the contract with GAI Consultants, Inc., being the part of the Geo-Con team charged with investigating and designing the remedial repairs which Geo-Con would install. Earlier subsurface investigations had differed on the depths of settling soil, with estimates from 6 to 27.4 m being given. A 1987 subsurface investigation indicated about 4 to 15.5 m of soil were involved. A compaction grouting program was designed and carried out in 1987-88 involving 65 grout holes outside the building and 48 holes inside, of which about 70 percent were vertical, with the remainder being angled due to physical constraints. Grout takes were relatively high for much of the hole lengths in soil, resulting in significant soil densification and construction of rather large grout columns bearing on rock. After completion, the City

[1] Consulting Engineer, 436 Waubonsee Trail, Batavia, IL 60510; formerly Staff Consultant, GAI Consultants, Inc.

[2] Geotech. Grp. Mgr., GAI Consultants, 570 Beatty Road, Monroeville, PA 15146.

[3] Manager of Construction and Remediation, McLaren/Hart Environmental Engineering Corp., Penn Center West III, Suite 106, Pittsburgh, PA 15276.

4 President, Geo-Solutions, Inc., 1250 Fifth Avenue, New Kensington, PA 15068

installed a sensitive settlement monitoring system, which has confirmed that the settlement has ceased. This paper discusses the building's settlement background, the 1987-88 subsurface investigation and compaction grouting program, and the results of the program.

INTRODUCTION

The Red Mountain Water Treatment Facility's filter plant building is a rectangular, 2-story, precast and cast-in-place concrete structure, about 11.9 m wide by 33.1 m long. The building houses 4 filter beds, a pipe gallery, an office area, and a chemical storage area. The filter beds and pipe gallery are located in the northern (plant north) two-thirds of the building with the pipe gallery separating the 2 pairs of filters. The filter bed and pipe gallery are cast-in-place reinforced concrete. The southern end of the building contains the chemical storage areas and the second floor office. The building walls are comprised of vertical precast concrete panels, while the roof is comprised of horizontal precast concrete panels spanning east-west, which supports a built-up roof. The general plan of the building is shown on Fig.1.

A short start-up test was followed by the plant starting full 24-hour operations on January 3, 1978. On February 10, 1978, settlement was noted when cracks were discovered in the filter plant building walls. Plant operations were stopped on February 13, 1978, and a subsurface investigation was performed using auger borings inside and outside the building. Leakage estimated to be as much as 8.3 million liters occurred during this initial operating period. In April or May 1978, remedial work to increase soil bearing capacities under much of the east and south wall footings was performed by intrusion gravel packing and grouting. Voids were reportedly detected and grouted below Filter Beds 1 and 2. Grout was injected into the upper $2\pm$ m of the foundation soils, with relatively high takes occurring in some areas. After the remedial work, no apparent drop of water level in the filter beds was noted overnight. On June 15, 1978, full 24-hour operation resumed.

The plant operation was again stopped on August 28, 1978, when significant leakage in Filter Bed 3 was noted. A geotechnical investigation was carried out by a new consultant and a range of repair options was suggested in the fall of 1978. This investigation indicated that about 6 to 12 m of soil might be settling. In February 1979, leak tests on the filter beds indicated that as much as 7.2 million liters of water may have leaked during the second operational period of the plant. No action was taken on installing the repair options from the fall 1978 investigation, and the building continued to settle.

In early 1983, another geotechnical investigation and discussion of repair options by still a different consultant indicated that as much as 27+ m of soil might be involved in the settlement. A period of relatively little settlement occurred in 1983-84. However, in the spring of 1985, significant settlement occurred at the southeast corner of the building. Due to the resumption of settlement, the City of Glenwood Springs, Colorado, solicited proposals from geotechnical specialty

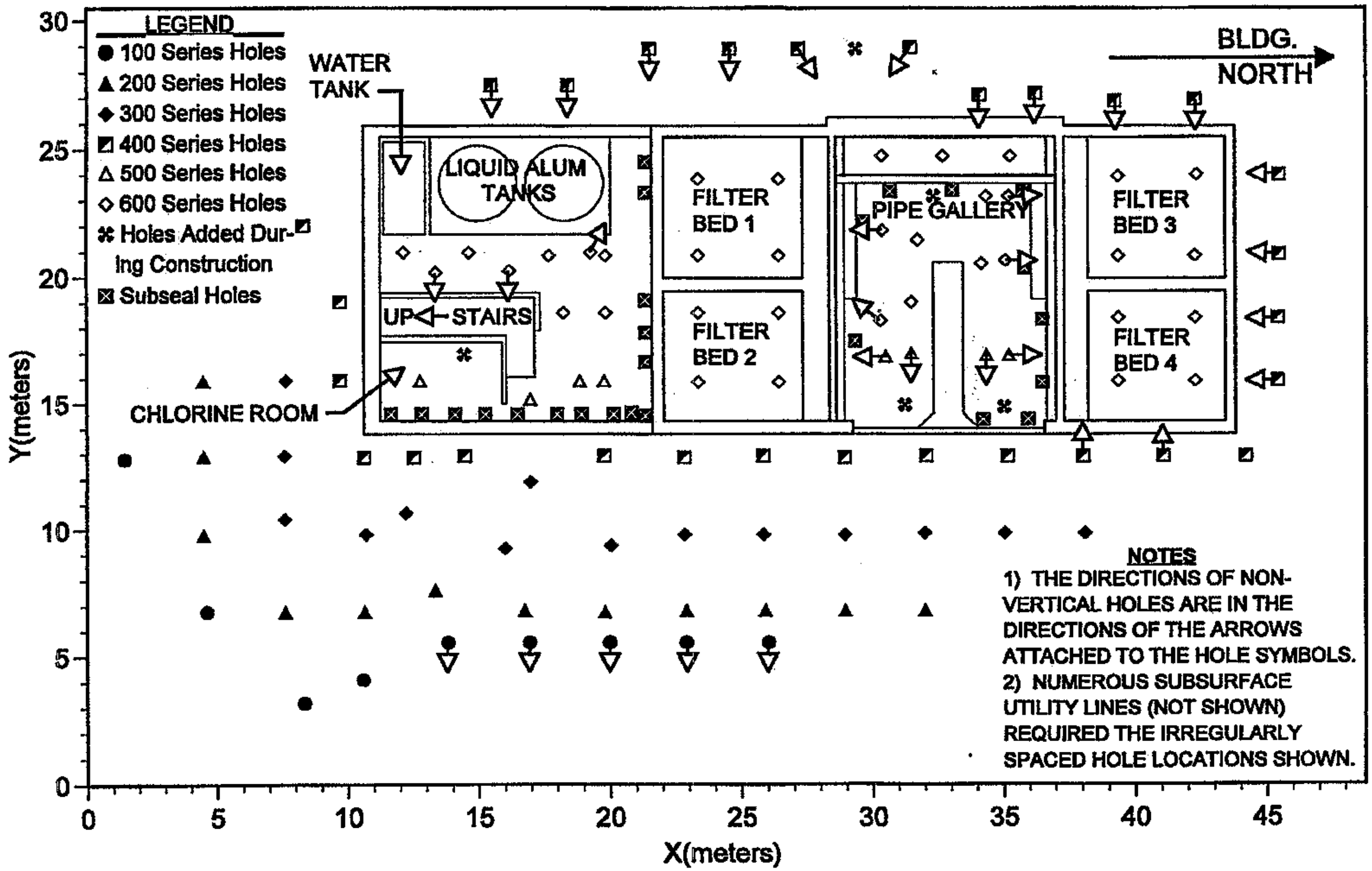

FIG. 1. Plan of Compaction Grouting Holes and Subseal Holes

contractors to evaluate the building settlement problem and provide repair recommendations, with the goal ultimately to have the selected contractor design and install a system to stop the settlement of the building. This paper outlines the design and execution of the selected solution.

SITE CHARACTERISTICS FROM AVAILABLE INFORMATION AND MARCH 1987 FIELD RECONNAISSANCE

The filter plant building was constructed on the northwest slope of a steep-sided, northeast-trending narrow valley about 1.2 km west of the central business area of Glenwood Springs. The ground floor of the building is constructed at about U.S.G.S. Datum elevation 1862.3 m on a flat area located above what was the lower extension of the narrow valley. The terrain west of the building rises sharply to elevations of 2400+ m.

It is believed that some soil, and perhaps rock, were cut from the mountain slope to the west, north, and south of the building to prepare the flat area. Actual disposition of the cut material is not known. However, much of it was used to construct dikes for Backwash Ponds 1 and 2, which are east of the southern half of the building and northeast of the building, respectively. Some cut material may have been utilized for fill in creating the flat area. Based upon the general terrain, it is believed that little, if any, fill was placed in the area below the building. Shallow fill depths, less than 3 m, were reported in test pits excavated near the building during the 1983 subsurface investigation. In addition, cross-sections based upon the fall 1978 subsurface investigation show fill only outside the building area and above the foundation area. Therefore, most of the soil beneath the building is believed to be residual soil, colluvial soil, slope wash soil, and/or debris fan soil. Soil visible near the building consists of coarse gravel, cobble and boulder regions with finer sandy, silty, and clayey regions, which agrees with the above assessment of the soil's sources. No visible signs of a landslide were noted in the area around the filter plant building.

Bedrock beneath the site is believed to consist mainly of siltstones, sandstones, limestones, shales, and claystones of the Paradox Formation. Rock outcrops occurred in nearby access road cuts north and west of the filter plant building. The bedrock exposed upslope (west) of the building dips generally east at about 39 degrees. In the roadcut upslope and slightly southwest, the bedrock bedding planes were highly contorted with irregular dip angles to the north and south. There is an apparent downdrag character to the bedding, which possibly represents a local fault. Bedrock in the road cut north of the building appeared to exhibit an undetermined dip to the east with a strike of about 9 degrees to the southwest.

Settlement profiles of the east and west walls of the building as of mid-1986, prior to remedial work, are shown in Fig. 2. The differential settlements were observable in the form of displacements and cracks in the building.

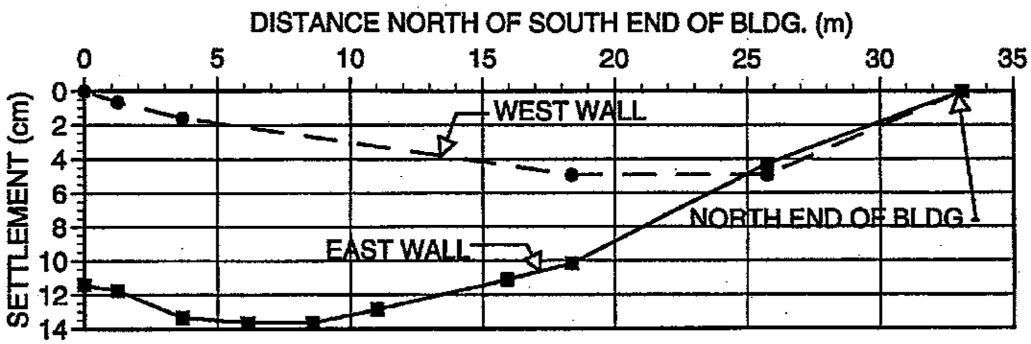

FIG. 2. Settlement of East and West Walls in Mid-1986

GEOTECHNICAL INVESTIGATION AND SUBSURFACE CONDITIONS

Due to differences in soil depths presumed to be involved in the settling presented by earlier consultants, an additional geotechnical investigation was performed by GAI to better define the problem. Five borings were drilled in early March 1987 at the locations shown on Fig. 3. Borings B-1 through B-5 were drilled to depths of 32.0, 36.6, 36.6, 14.6, and 13.7 m, respectively. Continuous standard penetration testing was done in soil, followed by cleaning out of the holes between samples using 15.9 cm O.D. hollow-stem augers, except where boulders were encountered and the augers were used to penetrate the boulders before resuming sampling. Two 6.35 cm O.D. Shelby tube samples were attempted in an auger hole near Boring B-2 between depths of 3.35 to 3.66 m and 7.32 to 7.68 m. Both tubes suffered rock induced deformation, poor recovery, and high sample disturbance. Pocket penetrometer readings of undrained shear strength were obtained for some of the cohesive soil samples. In general, continuous "NQ" rock core samples were obtained when soil sampling met refusal. Drilling mud was used during coring to maintain an open hole and remove cuttings. Special techniques, such as the use of short core runs, a split inner core barrel to aid in core removal, etc., were used to obtain and preserve the rather delicate rock types encountered during coring. Coring began in Borings B-1 through B-5 at depths of 5.6, 14.0, 13.7, 5.3, and 4.2 m, respectively. Soil depths in Borings B-1 through B-5 were estimated to be 4.9, 13.7, 15.6, 4.6, and 3.9 m, respectively. These depths to rock agree roughly with those of the earlier fall 1978 study. The top of rock contours estimated in the area of the filter plant building and shown on Fig. 3 are generally based on the 5 borings and the visible rock outcrops near the building of the 1987 investigation. Top of angles, clay seams, etc. were recorded for the recovered core. Perforated PVC pipe of 5.1 cm I.D. was inserted to or below the top of rock in Borings B-1, B-3, B-4, and B-5 to permit ground-water level monitoring following drilling. In general, rock information from earlier borings near the building was given less consideration in developing the top of rock contours shown on Fig. 3, since in cases the earlier data conflicted with that of the 1987 investigation. Core recovery, RQD, fracture some flushing of the holes using a garden hose and water was required to remove caved plugs and permit the insertion of the PVC pipe to the desired depths. Boring B-2 was reamed to about 15.2 cm I.D., and unperforated 8.6 cm O.D. aluminum slope indicator tubing was installed to the maximum feasible depth of 24.1 m to permit measurements of both settlement and lateral movement. The 24.1 m depth was limited by problems with the hole caving in both the soil and rock zones. Initial settlement data was recorded at the time of installation. Filter plant personnel measured subsequent settlements of the tubing. Inclinometer readings were taken in the slope indicator tubing on April 3 and 24, 1987. Falling head permeability tests were made as the drilling mud level dropped in the open cored holes in rock in Borings B-1 and B-4, yielding an estimated overall rock permeability value of 10^{-6} cm/sec (a value judged to be perhaps an order of magnitude or so low due to the effects of the drilling mud).

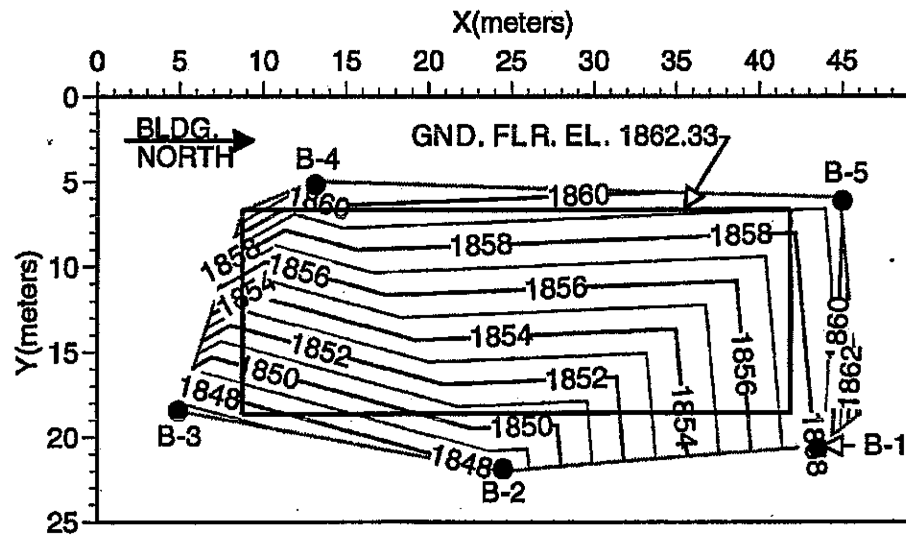

FIG. 3. Plan of Borings and Estimated Top of Rock Elevations in Meters

The soils could be generalized as clayey silt with sandy silt and silty sand zones, with few to many rock fragments. Compactness of granular soils ranged from very loose to very dense. Consistency of cohesive soils ranged from very soft to hard. In general, granular soils were mostly loose to medium dense. Uncorrected standard penetration resistance in the deeper soils of Borings B-2 and B-3 averaged 15, neglecting values greater than 50, which were considered indicative of boulders in the soil. Cohesive soils were mostly soft to stiff. Pocket penetrometer readings in the deeper cohesive soils of Borings B-2 and B-3 ranged from 35.9 to 430.9 kPa, but averaged about 191.5 kPa. Rock varied considerably in type, degree of weathering, existence of and the degree of decomposition of soil seams in the rock, angularity and cementation of fractures, etc. Three prominent rock types encountered in the borings were: (1) Conglomerate comprised of light gray to gray, medium hard to hard siltstone fragments in a poorly cemented matrix of brown to black, very soft to soft (rock classification) or soft to hard (soil classification) sandy or silty clay seams containing a few to extensive calcite stringers along old high and low angle fractures; (2) Gray to brown to black, soft to medium hard, shaly, calcareous siltstone to calcareous, silty shale with weathered brown to black, very soft to soft (rock classification) or soft to hard (soil classification) poorly cemented sandy or silty clay seams containing a few to extensive old calcite cemented high and low angle fractures; and (3) Very soft, fine grained sandstone, which had a white exterior surface when looking at the rock core, but which had a bright orange interior when the core was freshly broken. Except in Boring B-4, this sandstone was so poorly cemented that it crumbled to a highly decomposed, orange sand upon even moderate handling of the core. Ground-water levels in the borings were well below the top of rock.

Once the character of the soil and rock was observed, it was understandable that unless unusual care was taken in soil sampling and rock coring, zones of one could have been mistaken for the other, as was apparently the case in the earlier investigations.

It was noted by plotting top of rock profiles along the east and west walls of the filter plant building that the shapes of the plotted profiles were similar to those of the building settlement profiles along the east and west walls shown on Fig. 2. This suggested that the full soil depth above the top of rock was settling.

Settlement data for the slope indicator tubing in Boring B-2 indicated that no settlement of the soil and rock above the base of the slope indicator tubing occurred during the short monitoring period from March 9 to April 29, 1987. The inclinometer data for the short period from April 3 to 24, 1987, showed no significant lateral movement of the soil and rock above the base of the slope indicator tubing. This lateral movement result supported the earlier reconnaissance judgement that a landslide was not affecting the building.

Laboratory and Field Testing and Estimated Soil Properties

Laboratory and field tests on soils were performed during the various investigations. Testing was for pH, moisture content, unit weight, plasticity indices, grain size, specific gravity, chlorides, sulfate, sulfate sulfur, organic sulfur, pyritic sulfur, and total sulfur. The chemical testing for pH, chlorides, sulfate, sulfate sulfur, organic sulfur, pyritic sulfur, and total sulfur showed no results considered significant in relation to the building's settlement. Table 1 summarizes the estimated physical properties for the site soils.

Settlement Analyses

Since no relatively undisturbed Shelby tube samples could be obtained for testing, it was necessary to utilize literature correlations with standard penetration resistance in estimating parameters for use in settlement analyses. Using the estimated soil properties, settlements were estimated for 7 locations around the building and compared to measured total settlements at 6 locations around the building through mid-1986. In estimating settlements, the following approach was used for each point analyzed. First, assuming soil below the point was deposited in 1.52 m thick layers, elastic settlement of the soil and one-dimensional consolidation settlement of the partially saturated soil were estimated. It was assumed that all of the elastic settlement and one-half to all of the consolidation settlement

TABLE 1. Summary of Estimated Soil Physical Properties

A. From Testing of 1987, 1983 and Fall 1978 Investigations:

	Natural Water Content (%)	Natural Dry Density (kN/m³)	Specific Gravity	Liquid Limit (%)	Plastic Limit (%)	Grain Size Analysis Percent Passing U.S. Standard Sieve No. 4.76 mm	425μm	75μm
Range	3-15.3	12.95-19.64	2.52-2.56	22.3-25	19-22.8	23-82	19-56	15-45
No. of Tests	38	32	2	5	5	16	16	16
Average	9.1	17.44	2.54	23.7	20.4	~55(1)	~36(1)	~27(1)

B. Estimated Average Values from Calculations with Above Average Data:

Moist Density (kN/m³)	In-Situ Void Ratio	Degree of Saturation (%)	Water Content for 100% Saturation (%)	Plasticity Index (%)	Liquidity Index
18.85	0.43	53.8	16.9	3.3	-3.4

C. Estimated Average Values from Literature Correlations with Standard Penetration Resistance, Moisture Content, Etc.:

	Effective Shear Strength		Undrained Shear Strength				
Compression Index	Friction Angle (degrees)	Cohesion (kPa)	Friction Angle (degrees)	Cohesion (kPa)	Elastic Modulus (MPa)	Poisson's Ratio	Overall Permeability (cm/sec)
0.050(2)	35	0	0	95.8	9.576	0.3	10^{-4} to 10^{-6}

Notes: (1) On average, the results indicate that overall site soils are about 45 percent gravel and larger size particles, about 28 percent sand size particles, and about 27 percent silt and clay size particles, with about 8 percent clay size particles based on the 4 hydrometer analyses of the 1987 investigation.

(2) Average of 8 different correlations.

occurred prior to building construction. Second, additional post-construction settlement was estimated for the gravelly pockets believed to exist in the soils below the building, assuming that finer particles migrate downward into open voids between gravel particles and that the particles forming the pockets degrade over time due to wetting, etc. Settlements due to this cause are not included in conventional elastic and one-dimensional consolidation settlement estimates, in which the parameters are related mostly to fine material characteristics. Since no well accepted approach exists to assess the settlement of rocky soils susceptible to particle degradation upon wetting, the admittedly crude approach outlined hereafter was applied. The estimated 25 percent thickness of gravelly pockets visible in soil cut heights near the building was applied to the soil depth at the point being analyzed, and the settlement of the gravelly pockets was estimated to be 2 percent of the gravelly pocket thickness at the point. The assumption was made that 30 percent of the gravelly pocket settlement should have occurred over the 9+ years since building construction. These percentages were estimated by extrapolating information in Sherard et al. (1963) on the post-construction settlement of rockfills, tempered by observations from another project discussed in Wimberly et al. (1993), where a building built on an initially rocky 15 m deep fill was settling. It was realized that it was possible that much more than 30 percent of the potential gravelly pocket settlement had occurred, since there had been significant leakage from the plant facilities into the soils below. Third, elastic and one-dimensional consolidation settlements of the partially saturated soil were estimated due to the estimated loads on strip footings alone and on Filter Beds 1 and 2 alone. The consolidation settlement in each case was assumed to include the elastic settlement plus the later time dependent settlement. The post-construction portion of these settlements was taken to be the time dependent settlement and 50 percent of the elastic settlement. Fourth, estimated post-construction total settlements were estimated for the strip footing area alone, for the Filter Beds 1 and 2 areas alone, and for areas where the loaded strip footing and Filter Beds 1 and 2 areas interact. Table 2 summarizes the final computed settlements and measured settlements for the 6 locations where comparisons were made. In general, settlements from the computations exceeded those observed through mid-1986. This supported the idea that significant future settlements were possible, even though the highly approximate nature of the computed values was understood by all involved.

REMEDIAL REPAIR OPTIONS

Brief reviews of most of the available methods to stabilize settling sites were provided to the City, along with a brief discussion of the feasibility of each method for use in stabilizing the settling filter plant building. It was recommended that any system implemented to reduce potential future settlements provide relatively uniformly stiffness conditions below the entire building, in order to not aggravate the existing effects of differential settlements on the structure.

After considering available options, it was recommended that compaction grouting of all soil above the top of rock be carried out in the area below the building and below the planes extending at one horizontal to two vertical downward from the outside edges of the ground floor. In addition, the following 3 drainage options were recommended: (1) Construction of diversion ditches upslope of the filter plant building to divert surface runoff around the building area; (2) Grading of the area around the building to facilitate surface runoff and covering of the area around the building to reduce infiltration by a suitably engineered low permeability soil cap or paving; and (3) Periodically leak testing the filter beds, piping, etc. at the site and performing needed maintenance to minimize leakage. It was also recommended that if voids were encountered below the ground floor in the filter plant building they should be filled with flowable grout.

The City reviewed the recommendations, elected to have the compaction grouting done, and decided to perform the 3 drainage recommendations themselves or with local contractors once the compaction grouting was completed.

TABLE 2. Estimated Post-Construction Total Settlements versus Measured Total Settlements as of Mid-1986

Location	Estimated Settlement With Interaction of Strip Footing and Filter Beds 1 and 2 Loaded Areas (cm)	Measured Settlement (cm)
Southwest Corner of Bldg.	0	0
West Side of Filter Bed 1	10.2(1) to 11.5(2)	4.3
East Side of Filter Bed 1	18.4(1) to 38.5(2)	11.9
Southeast Corner of Bldg.	11.2(1) to 31.3(2)	11.4
Southern 15± m of East Wall	11.2(1) to 31.3(2) at South End 18.4(1) to 38.5(2) at North End	12.7
Center of Filter Beds 1 and 2	14.4(1) to 22.3(2)	8.1

Notes: (1) With no post-building consolidation settlement of soil under its own weight.
(2) With maximum estimated post-building consolidation settlement of soil under its own weight.

Compaction Grouting, A Short Perspective

Briefly, compaction grouting, unlike permeation grouting, involves the controlled injection of stiff, mortar-like grout into previously drilled holes in the soil. The basic concepts were discussed by Graf (1969). Mitchell (1970) compared the approach to other grouting methods. Brown and Warner (1973) discussed how to perform the process and its applicability, as well as the then current technology. Criteria for planning and performing compaction grouting were given by Warner and Brown (1974). Warner (1982) discussed the first 30 years of compaction grouting, and contains excellent summary discussions regarding the planning and execution of compaction grouting programs. Details on the compaction grouting approach used to stabilize the settling water treatment plant building follow, along with an evaluation of its success.

COMPACTION GROUTING PROGRAM

The program was carried out between early July 1987 and mid-March 1988. It consisted of injecting cement grout, having a slump of 2.54 cm or less (usually between 1.25 and 2.54 cm), into the soil to be stabilized via holes drilled to below the top of rock outside and inside the building. The grout was a mixture of cement, sand, fly ash, water, and a minor amount of bentonite and had a minimum unconfined compressive strength of 8.27 MPa after 7 days. The grout injection was limited by: (1) A maximum take of 0.28 m^3 per 0.30 m increment of hole length being grouted; (2) A maximum grouting pressure of 6.90 MPa measured at the top of the hole; and (3) Surface movement as detected by laser levels, or by audible or visible signs. The grout mixer was capable of complete mixing of the stiff grout and had a metered supply system for the ingredients to enable close control of grout consistency. The grout pump was capable of nearly uniform flow rates from 0.003 to 0.15 m^3 per minute. The pump hopper had a force feed mechanism to reduce cavitation of the very stiff grout during pumping. Grout take was measured as the volume displaced by the force feeding piston times the number of piston strokes. Slight cavitation did occur in pumping the grout such that the actual take was less than (within 10$\pm$ percent of) the measured take on the above basis. As such, takes discussed herein are uncorrected for this effect. The grout was injected through minimum 5.1 cm I.D. pressure hose having non-restrictive flow couplings from the pump to the hole and through 5.1 cm I.D. flush joint steel casing placed in the holes to the top of each 0.30 m increment of hole to be grouted.

Holes were drilled using the ODEX over-ream bit method with air as the drilling fluid to remove cuttings. A specially fabricated drill was utilized in the tight access, low overhead clearance locations inside the building. The holes were spaced on a nominal 3.05 m square grid over the area to be stabilized, a spacing in the mid-range of the 2.44 to 3.66 m square grids generally used for initial hole spacings in compacting grouting programs. Hole spacings were adjusted where necessary due to lack of access or buried utility lines. Angled holes were used for relocated holes to try to grout the desired soil depths in locations which would achieve as close to uniform support over the grouted area as feasible. Maximum angles from the vertical were 20 degrees for outside holes and 10 degrees for inside holes. The final hole locations are shown on Fig. 1. The outside holes in Fig. 1 were initially drilled and grouted in the following order to provide support and lateral confinement to the soil below the building: 100 Series (9 holes, 4 vertical and 5 angled to avoid drilling on the steep Backwash Pond 1 slope), 200 Series (12 holes vertical), 300 Series (14 holes vertical), 400 Series (13 holes vertical and 16 angled to avoid buried utilities). The inside holes were then drilled and grouted in the following general order: 500 Series (4 holes vertical and 4 angled) and 600 Series (30 holes vertical and 6 angled). Due to hole relocations and wide spacings in certain areas, 4 vertical inside holes were added during construction. One outside vertical hole was added on the west side of the building, where the 3 nearby holes had relatively high grout takes to the bottom of the holes.

Due to the nature of the soil and rock at the site, it was difficult to detect when the top of rock was reached in drilling the grout holes. Therefore, many of the holes were overdrilled to be sure they were below the top of rock. In rock, as expected, takes were typically low and grouting was usually controlled by high pressures. Some rock-like zones in soil also occurred. In evaluating the grout takes, it was necessary to initially estimate a top of rock from the grouting record of take and pressure per each 0.30 m increment of hole grouted. The evaluation was then carried out only for the depth grouted above the estimated top of rock (nominally that in soil). The 100 Series and 200 Series holes were compaction grouted using the "bottom-up" approach in which the casing was initially located at the drilled hole bottom. It was raised in 0.30 m increments as grout was injected. In only the 100 Series holes, the upper take limit was set as 0.14 m^3 per 0.30 m increment of hole length, as these holes were the first done and were done while the final specifications were still in preparation. The fact that 94 percent of the soil depth grouted in these holes was controlled by the upper take limit resulted in the take limit being doubled for subsequent holes. The 300 through 600 Series and the 5 added holes were grouted using a partial "top-down" and partial "bottom-up" approach. In this approach, an oversized hole was drilled and cased to 5 feet below the surface. Outside the building the casing was held in place by granular backfill around it, while inside the building it was usually cemented in place. Three successively deeper 1.52 m stages of "top-down" compaction grouting were generally carried out between 1.52 m and 6.1 m below the surface to achieve near-surface densification and vertical confinement to permit the normally higher pressures to be used below. In each stage, drilling occurred to the stage base, where the above "bottom-up" method was used in grouting that stage. The stage was then drilled through, after a delay of at least 16 hours, before subsequent work below the stage. After these 3 upper stages were completed, the hole was drilled to its full depth and grouted using the above "bottom-up" approach.

Table 3 summarizes the grout take data in soil. The data in Table 3 indicate that the soil was generally looser or softer in the region outside of the building than it was below the building. This may be partly due to one or more of: (1) The settlement which had occurred under the building; (2) Better initial near-surface compaction in the area of the building than elsewhere; and (3) The earlier grouting and remedial work below the building. Fig. 4 shows a plot of the percent of the soil length grouted for outside, inside, and total holes that were controlled by the upper grout take limit versus the pressure interval occurring during injection when the upper take limit was reached. In general, most of the upper limit takes occurred under relative low grouting pressures, below about 30 percent of the upper pressure limit.

ACCEPTANCE CRITERIA

Ideally, if soil conditions at the site were relatively homogeneous and such that representative undisturbed Shelby tube samples of the soil could have been obtained, it would have been possible to estimate the in-place average unit weight of the settling soil. Modified Proctor tests could then have been performed on soil samples to estimate the maximum dry unit weight for the settling soil. If one assumes that the soil might have settled an acceptable amount if the soil was compacted

TABLE 3. Summary of Grout Takes Per 0.30 m (1-Foot) Increment of Hole Grouted in Soil

For 329 m Length Grouted in Outside Holes (1)	
Percent of Soil Length Grouted	Grout Take
61.4	≥ U.L. (2)
10.8	≥ 0.5 U.L. but < U.L.
8.9	≥ 0.2 U.L. but < 0.5 U.L.
18.9	< 0.2 U.L.
100.0 Total	

For 311 m Length Grouted in Inside Holes	
Percent of Soil Length Grouted	Grout Take
27.6	≥ U.L.
9.5	≥ 0.5 U.L. but < U.L.
9.6	≥ 0.2 U.L. but < 0.5 U.L.
53.3	< 0.2 U.L.
100.0 Total	

Notes: (1) Length of outside holes includes 100 Series Holes for which upper limit on grout take was 0.5 U.L.
(2) U.L. = Upper Limit on grout take = 0.28 m^3 per 0.30 m increment of hole length (equivalent to 0.93 m^3/m of hole and 10 ft^3/ft of hole).

compacted to 90 to 95 percent of the modified Proctor maximum dry unit weight, one could compare the estimated average in-place soil unit weight to the unit weight corresponding to 90 to 95 percent of the modified Proctor maximum dry unit weight. Then, an acceptable increase in the average soil unit weight may be selected, from which the needed volume compression of the soil during injection could be estimated. The injection program could then be planned, monitored, and adjusted, as judged necessary, to try to obtain the desired level of grout take to yield the desired level of soil compression. Unfortunately, the highly variable and rocky nature of the soils at this site prevented the obtaining of representative Shelby tube samples of the settling soil and rendered the above approach impractical.

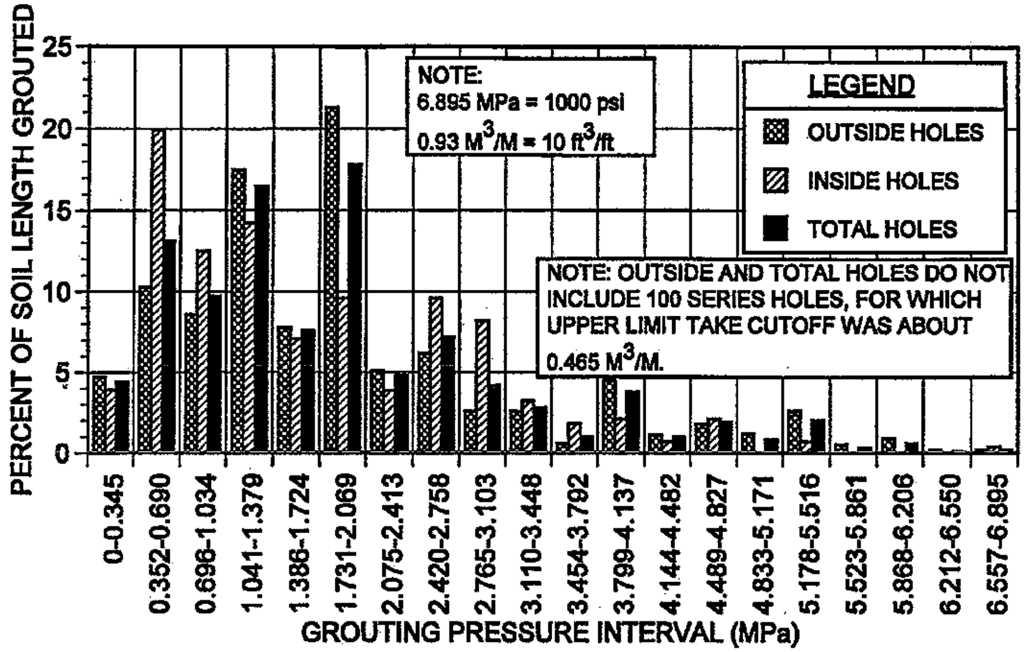

FIG. 4. Percent of Soil Length Grouted When Take ≥ 0.93 m^3/m versus Grouting Pressure Interval

Some judgement of the success of the program may have been achievable by carrying out after grouting standard penetration tests to compare to the data from the earlier field investigations. Such data from below the building were not available from the earlier investigations. Collecting such data before or after the compaction grouting was carried out would have required difficult and expensive inside drilling. In addition, due to the highly variable and rocky nature of much of the soil, it was felt that even outside drilling and comparisons of before and after standard penetration test data, while possible, would be costly and may yield inconclusive results. Therefore, since conditions at this site were far from ideal, an acceptable level of grout take was based largely on judgement, followed by further monitoring to assess if the settlements ceased or slowed sufficiently. If they did, the initial program achieved its goal. If not, a subsequent program, using split-spaced holes, might be required.

A grout take of 0.28 m^3 per 0.30 m increment of hole length for 4 holes at the corners of a 3.05 m square grid implies average theoretical densification of the soil within the grid of about 10 percent; i.e., an increase in the soil's unit weight of about 10 percent, due to compression of the original soil's volume by the injected grout. This level of densification (which also implies soil stiffening) was judged to be a reasonable level of improvement in the site soils and was selected to be the program's upper limit on grout takes, as given above. Such a take also theoretically implies the construction of grout columns at the hole locations having average diameters of about 1.1 m. Due to injection variations from interval to interval, the columns typically have substantial outer surface irregularity. It should be noted, that grout takes and the resulting grout column diameters tend to be larger in more compressible (poorer) soils than in less compressible (better) soils; i.e., the grout tends to go if and where it is needed to affect an improvement in the subsurface conditions. Since the injection of grout can add considerable weight to the settling soil, it is necessary to treat the zone of potentially settling soil down to an essentially incompressible material, such as rock. This is generally the case unless only a relatively thin compressible zone requires treatment, such that the injected grout does not add significant weight to the soil and cause settlement of the soils below the treated zone.

Construction of rather substantial grout columns, such as those theoretically constructed at this site (or even substantially smaller ones) from the top of rock to near the surface over the area treated results in stiff grout columns that help resist soil settlement between the columns by shear stresses along the column-soil interface. Both soil densification and the constructed columns improve the soil's resistance to settling. Which of these is most important in improving a site's stability to acceptable levels often can not be accurately assessed. The rather large takes at this site, implying both substantial soil densification and construction of relatively large grout columns, indicated that major improvement was made in the soil's ability to resist further settlement. However, as explained above, once a site is treated only future monitoring (discussed below) tells the story of success, or failure requiring further treatment.

Subseal Holes

In performing the compaction grouting work, voids were detected below the concrete in the building area. Soundings were done and 25 holes were cored through the concrete to permit high slump (30.48$\pm$ cm) grout to be injected to fill (seal) the subsurface voids. These hole locations are shown on Fig. 1 as subseal holes. Little to no take occurred in the Pipe and Valve Gallery area. Along the south side of Filter Beds 1 and 2, voids about 2.5 cm deep were grouted. Along the east wall south of Filter Bed 2, a 7.6 to 10.2 cm deep void was grouted for about the northern half. The void depth tapered to about 2.5 cm near the southeast corner. About 2 m^3 of subseal grout were injected to fill the voids.

Settlements During and After the Compaction Grouting Work

During the compaction grouting work, the building continued to be monitored for settlement, along with the slope indicator tubing in Boring B-2. The southern two-thirds of the east side of the building experienced about 1.3 cm of settlement, as did roughly the upper 9.1 m of soil at Boring B-2. At Boring B-2, the soil in the depth range of 9.1 to 12.2 m experienced about 2.5 cm of upward movement. The upward movement was likely associated with rather large grout takes occurring in the area of Boring B-2 (one hole was only about 3 feet to the southeast).

Settlements observed along the southern two-thirds of the east side of the building may be related to compression of the soil mass under the weight of injected grout, but are more likely related to the 1.2 to 2.1 m deep trench excavated parallel to and well below the footings along the east wall by the City in mid-July 1987 to assist in locating utilities so that the grout holes could avoid them. This trench was left open about a week and was backfilled with excavated soil tamped by the backhoe bucket. Also, after several rains, 2 drain pipes from the roof discharged into the trench area until the pipes were extended to the east to drain to Backwash Pond 1.

The City performed or had performed the recommendations regarding drainage control and had a settlement monitoring system installed after the compaction grouting work was completed. The actual data from this system were not provided to the authors. However, from telephone discussions with City personnel, the system is reportedly sensitive enough to detect very small movements, including the thermal movements of the structure. This system, which has gathered data for about 5 years, has reportedly confirmed that no significant additional settlement of the filter plant building has occurred; i.e., no movements not believed to be associated with thermal changes in the structure have occurred over this 5-year period.

Conclusions

Compaction grouting was effective in stabilizing the settling filter plant building. Close cooperation and communication between the City and the Geo-Con/GAI team facilitated the work at this difficult site being completed within City budgetary constraints and without interruption of the City's water supply. The investigation and implementation of the compaction grouting and subseal hole grouting program cost about \$525,000 when it was done in 1987-88. The original estimate was about \$470,000, which did not fully account for the high grout takes and required subseal holes. This grouting program cost about one-tenth of the cost the City would have incurred if settlement had continued such that construction of a new filter plant building was necessary. The City's cost in implementing the other recommendations is not available to the authors.

Acknowledgements and Comments

The authors thank the City of Glenwood Springs, Colorado, and in particular Mr. Michael Kopp, City Manager, for permission to publish this paper and Mrs. Arlene Wimberly (formerly of GAI) for the preparation of this manuscript. The authors acknowledge the value of the information provided for use by the City from the 1983 and fall 1978 subsurface investigations, as well as from other reports and drawings, and choose not to identify the firms involved in view of differences noted between the various investigations. Overall, the subsurface conditions at this site were highly complex; and the soil and rock were difficult to drill and sample well, and to test effectively. In view of this, the differences are not surprising or particularly unusual in the authors' opinion.

APPENDIX - REFERENCES

Brown, D. R., and Warner, J. (1973). "Compaction grouting." *J. Soil Mech. Found. Div.*, Proc. ASCE, 99(SM8), 589-601.

Graf, E. D. (1969). "Compaction grouting technique." *J. Soil Mech. Found. Div.*, Proc. ASCE, 95(SM5), 1151-1158.

Mitchell, J. K. (1970). "In-place treatment of foundation soils." *J. Soil Mech. Found. Div.*, ASCE, 96(SM1), 73-110.

Sherard, J. L., Woodward, R. J., Gizienski, S. F., and Clevenger, W. A. (1963). *Earth and Earth-Rock Dams*, John Wiley and Sons, New York, New York.

Warner, J. (1982). "Compaction grouting - the first 30 years." *Proc. ASCE Specialty Conf. on Grouting in Geotechnical Engineering*, ASCE, New York, New York, 694-706.

Warner, J., and Brown, D. R. (1974). "Planning and performing compaction grouting." *J. Geotech. Eng. Div.*, Proc. ASCE, 100(GT6), 653-666.

Wimberly, P. M., III, Mazzella, S. G., and Newman, F. B. (1994). "Settlement of a 15-meter deep fill below a building." *Vertical and Horizontal Deformations of Foundations and Embankments*, Geotechnical Special Publication No. 40, ASCE, New York, New York.