Case Study of Ground Modification to Control Settlement in Uncontrolled Fill

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Abstract

The rapid growth in urban development has limited the availability of virgin construction sites. Complex subsurface conditions were encountered at a site for a 131,000 sq. ft. retail and parking development in Cincinnati, Ohio. The site was located in an industrial area where uncontrolled fill was previously placed to fill a drainage swale. A substantial thickness of existing fill (15 to 25 ft.), with significant organic content, was present.

To establish planned finished floor grades, 30 ft. of new engineered fill was proposed. Considering the high groundwater table and the need to provide drainage paths to promote consolidation of the foundation soils, a Geopier® soil reinforcement system (first application of its kind in Cincinnati) was selected to increase the modulus of the existing fill and to reduce the magnitude and time-rate of settlement.

The ground modification system was designed to increase the composite stiffness of the soil mass and to serve as vertical drainage galleries to reduce time rate of settlement. A settlement monitoring program consisting of vibrating wire settlement cells and settlement plates was undertaken in the improved and unimproved fill areas to validate design methods and confirm performance. This paper is of particular significance as it presents both the theoretical approaches and results of field measurements for a relatively new ground improvement technology that may be cost effectively used to improve marginal building sites in the Ohio River Valley.

Project Description

The proposed project involved construction of a single-story "big box" retail store in a former industrial area in Cincinnati, Ohio. The proposed retail store covered an area of 131,000 square feet. The estimated footing loads were 65 kips for interior columns, 50 kips for exterior columns and 1.5 to 2.0 kips per foot for the masonry walls. In consideration of the proposed building use, the floor slab was to be relatively flat with little tolerable differential settlement. In addition to the proposed building, asphalt-parking areas were planned north and east of the building.

The grades, prior to construction, differed significantly between the northern and southern ends of the building. Figure 1 illustrates the proposed building layout and the grades in the year 2000, prior to construction. The approximate southern half of the proposed building footprint was relatively level with grades sloping gently to the north from about elevation 605 ft. to 600 ft. From about the midpoint of the building footprint northward, grades dropped significantly from elevation 600 ft. to approximately elevation 568 ft. The proposed building had a finished floor elevation of 604 ft. Thus, the southern end of the building required 4 ft. of fill to 1 ft. of cut to establish grade. The northern end of the building required approximately 30 to 36 ft. of new engineered fill to establish grade.

1. Chief Geotechnical Engineer - H. C. Nutting Company
2. Project Geotechnical Engineer - H. C. Nutting Company
3. Project Geotechnical Engineer - H. C. Nutting Company

Historical Topographic Data

Review of the 1912 Topographic Survey of the project area indicated the 2000 grades were near those of 1912 in the southern portion of the proposed building and the parking area east of the building. Figure 2 illustrates the approximate 1912 topography in the vicinity of the building. The topography in the north, east-central and western portions of the building changed significantly between 1912 and 2000. As shown on Figure 2, two drainage tributaries of Duck Creek formerly traversed the east and west sides of the building footprint. The swales extended in a northerly direction and outlet to a level floodplain. Duck Creek is located immediately north of the project area.

The low area in the northern portion of the building footprint was once part of the flood plain of Duck Creek. This area had been named "Lake Success" by the owner of the site. The lowest portion of the eastern swale extended to approximately elevation 570 ft. The western swale extended to approximately elevation 565 ft. The center of the Lake Success, area north of the building, had an approximate elevation of 555 ft. in 1912.

Review of the topographic survey suggested that drainage swales had been filled between 1912 and 2000. Approximately 30 to 35 ft of fill was placed in the eastern and western portions of the building area between 1912 and 2000.

Site Conditions and Geology

The Lake Success area in the northern portion of the proposed building and the parking area to the north had become a harbor to waste materials from the foundry operations. Several end-dumped piles of sand, clay and gravel were present throughout the Lake Success area.

Several storm water lines outlet into the Lake Success area. The area was not properly graded and water had ponded in the northern portion of the Lake Success area. The water appeared to be static as a large area of cattails had grown in the vicinity of the ponded water.

A 54-inch diameter combined sewer was present in the western building area and extended to the Lake Success area. The sewer followed the former natural drainage swale along the western portion of the building and extended northward through Lake Success. The brick sewer was to be replaced with concrete pipe.

The project area lies within the physiographic region known as the "Norwood Trough". The geology of this region is one of the most complex in the Greater Cincinnati Area. The Norwood Trough was a short segment of a buried valley of the ancestral Ohio River. During glacial times, the Ohio River flowed in a northerly direction. Flow of the river in the project area eroded a deep valley. The valley floor is approximately 200 ft. below the ground surface. Glacial advance into the area dammed the north flowing river and filled the eroded valley with glacial outwash, glacial lakebed clay, and glacial till. The level plain left by the glaciers had been subsequently eroded. Duck Creek to the north of the project area was formed by the erosion of the level glacial plains.

Subsurface Exploration

H. C. Nutting had conducted several geotechnical studies in the project area for the owner of the industrial facility. Historical test boring data dating back to 1977 was utilized in conjunction with new test borings and test pits to characterize the subsurface conditions across the proposed development area. The subsurface study consisted of an initial broad preliminary study followed by a detailed study of the specific building and parking area. A total of 17 test borings were drilled in 2000 to characterize the subsurface conditions in the building and north parking area. The 1912 topographic plan map was utilized to locate the test borings in areas were fill was known to have been placed.

A total of nine test pits were performed, in addition to the test borings, as part of the final geotechnical study to further determine the fill composition and to evaluate perched groundwater conditions with regards to an open cut excavation. The test pits were located in the southern portion of the Lake Success area (northern portion of the building). The northern portion of Lake Success could not be studied before construction due to the wet surface soils and ponded water.

Subsurface Conditions

The obtained subsurface information correlated well with the historic topographic maps and known site usage, and geology of the area. As suspected, in the Lake Success area and the former drainage swales, deep uncontrolled fill was present above the natural overburden soil. In some areas, the uncontrolled fill soil was highly organic. The fill in the former east and west drainage swales and Lake Success area ranged from 20 to 35 ft. below the grades prior to construction.

Away from the Lake Success area and the former drainage swales, the general subsurface profile consisted of topsoil, glacial till and lakebed clay from top to bottom.

The heterogeneous fill was generally comprised of two distinct material types. The fill material was associated with the foundry operations and consisted of foundry sand and cinders with varying amounts of slag, fine gravel, clay, sand and organics. Large casting molds (12" diameter) and casting mold fragments were observed in the foundry sand during the test pit excavations. Additionally, layers to lenses of ash were observed with the foundry sand material. Railroad ties and logs up to 10" diameter and 10 ft. long were observed in the test pit excavations. This material was waste material from the former industrial operations. The foundry fill rated as very loose to loose.

In addition to the foundry sand fill, cohesive, wet and highly organic fill material was encountered in the Lake Success area. The highly organic fill soils were generally concentrated in the Lake Success area. The organic content ranged from 9% to 47%. The moisture content was very high, ranging from 28% to 220%. The natural soils underlying the fill consisted of glacial till and lakebed deposits with moderate compressibility characteristics.

Groundwater was observed during drilling of the test borings at 2.5 to 45 ft. below existing grades. Upon completion of drilling and removal of the drilling augers from the borehole, groundwater was noted at various elevations. The variation in the groundwater level can be attributed to the presence of granular lenses with the glacial till and perched groundwater within the uncontrolled fill. The at-completion of drilling groundwater levels ranged from 10 to 28.5 ft. below existing grades.

During the test pit exploration, groundwater was encountered in 4 of the 9 test pits at depths that ranged from 2.5 to 10.0 ft. below existing grades. The groundwater seepage into the test pit excavation was significant in several of the test pits causing the sidewalls to collapse.

Geotechnical Analyses / Foundation Evaluation

The building area had a very complex subsurface profile. The southeastern portion of the building area exposed natural glacial till soils at the anticipated bearing elevation. The east and west sides of the proposed building area were characterized by deep buried drainage swales. The grades in the northern portion of the building area (Lake Success) were 20 to 30 ft. below the proposed finished floor level.

A conventional undercut and replacement of existing fill was selected in the east and west drainage swale areas of the building pad. Based on the test borings and 1912 topography, a 20 to 30 ft. undercut was performed. The undercut material was typically foundry sand, which was reused as an engineered fill.

Several foundation/ground modification options were considered in the Lake Success area and presented to the design team during project meetings, these options included:

1. Deep Dynamic Compaction: The soil profile was predominately cohesive, though some granular zones were present in the Lake Success area. Also, elevated perched groundwater levels were observed. Additionally, the vibration from the dynamic compaction would likely impact adjacent industrial structures sensitive to vibrations. In consideration of all of the above factors, dynamic compaction was not considered as an attractive option.

2. Conventional Deep Foundation: The natural soils underlying the heterogeneous fill were medium stiff to stiff alluvial and lakebed deposits. Per preliminary analyses, very long piles/piers (80 ft or longer) would be needed. Also, significant negative skin friction would develop due to the large magnitude of soil settlement after foundation installation; this would require expensive measures to reduce downdrag or an extensive time delay to allow settlement to occur before foundation construction could begin. Due to project schedule and cost considerations, this option was not considered attractive.

3. Complete Undercut and Replacement: Partial undercut and replacement of the fill was not recommended due to long-term settlement considerations. A complete undercut of up to 25 ft. of fill was considered. The primary disadvantages included high perched groundwater conditions, the majority of the undercut material composition was not suitable for reuse as engineered fill and had to be wasted off-site, and time involved with undercut and replacement. Also, the base of the undercut would expose water softened soils requiring stabilization. Significant construction difficulties and uncertainties were anticipated with this option.

4. Pile Supported Geogrid Reinforced Mattress: Due to the height of new fill (up to 30 ft.) and presence of relatively weak alluvial and lakebed deposits below the existing fill, the auger-cast piles were estimated to be very long and costly.

5. Surcharge: Preliminary analyses indicated that a minimum of 12 weeks would be needed for 90% of primary consolidation settlement to occur. Also, secondary settlements of 1" to 1.5" were estimated due to degradation of the organic

portions of the fill. Due to the volume of fill needed for the surcharge and the time required after surcharge placement, it was determined that this option would impact project completion and was not preferred.

Following review of all available data, various options, and owners aggressive completion schedule, it was our opinion that the most practical means of establishing a suitable base for the new engineered fill and limit long-term settlement was to improve the existing heterogeneous fill in-place with the use of rammed aggregate piers (Geopier®) or stone columns. Geopier® elements were selected as the choice of ground modification and a detailed design was performed by Geopier Foundation Company using performance criteria and soil data established by H. C. Nutting Company.

Overview of Geopier® Intermediate Foundation

Geopier intermediate foundations are both a specialized foundation support system and a vertical soil reinforcement system. The Geopier installation process is shown on Figure 3. Rammed aggregate piers (Geopier) are constructed by first removing a volume of compressible material by drilling a hole. A thin lift of open graded stone is then placed at the bottom of the cavity. The soil at the bottom of the cavity is prestressed and prestrained by ramming the stone with a specially designed tamper using an impact energy source that causes a "ramming" action. Vibration energy is not as effective in densifying the Geopier element. A very stiff element is then constructed within the cavity using well-graded aggregate placed in thin lifts and highly densified by ramming with the same tamper used for bottom prestressing rather than densification. The adjacent matrix soils are improved by lateral prestressing. The energy source applies impact ramming action, rather than vibratory energy, to a 45-degree beveled tamping apparatus that maximizes lateral prestressing of the matrix soil. The rated energy of the present Geopier installation apparatus ranges from 250,000 ft-lb. to 1.7 million ft-lb. per minute, while the ramming frequency generally varies from 300 to 600 cycles per minute. The buildup of lateral stress in the surrounding matrix soils develops an over-consolidated soil surrounding each rammed aggregate pier, resulting in a stiffened aggregate pier/matrix soil mass. In addition, the prestressing and prestraining of soils adjacent to the sides of the aggregate pier results in an undulated aggregate pier/matrix soil interface that provides excellent engagement of the aggregate pier with the surrounding soil. The lateral stress buildup approaches the passive limit of the soil, thereby providing maximum shear strength along the aggregate pier shaft. The combination of an increased stiffness of the subsurface soils and a decreased drainage path for water provides a stable base for support of the new engineered fill and aids in accelerating the fill induced settlement of the foundation soils.

Geopier Element Design

The Geopier elements were designed by the Geopier Foundation Company. The 30-inch diameter Geopier elements were located on 9-foot centers extending a minimum of 20 ft. beyond building lines. The layout of the Geopier elements are shown on Figure 4. The settlement of the reinforced zone is influenced by the spacing of the Geopier elements. Spacing of 8 to 12 ft. were evaluated and based on cost and engineering performance a 9 ft. spacing was selected. The Geopier subgrade reinforcement covers an area of approximately 54,000 square feet and includes 667 Geopier elements. A total settlement of 5.8" was predicted by design calculations. The settlement was predicted to occur rather quickly and footing and slab-on-grade construction could begin within one week after fill completion. The Geopier element can be approximated as a stiff spring being at least 10 times stiffer than the surrounding soil matrix. The behavior of a Geopier element in compression is not elastic, however, its deflection under load occurs rapidly, typically becoming less than 0.01"/hour within 30 minutes of loading except for relatively high loads or in very soft clay soils. The effective friction angle for the No. 57 stone (as measured on other projects by Geopier Foundation Company) was 48.8 degrees. The Geopier elements had an average length of 15 to 20 ft., extending through the fill and a minimum of 3 ft. into natural soils. Some of the Geopier elements had to be extended deeper where greater depths of fill were present. Since the Geopier elements were also intended to serve as drainage paths to reduce the time for settlement, an open-graded stone (No.57 stone) was recommended by H. C. Nutting Company. The average compacted lift was 12". A 2 ft. layer of sand (free-draining) was placed over the geopier elements to serve as a reinforcing layer and also aid in removing excess water being wicked by the Geopier elements. A schematic representation of the ground improvement scheme is shown on Figure 5.

Geopier Element Load Test

Modulus load tests on Geopier elements are typically required to provide essential confirmatory information on element stiffness and behavior of Geopier element under load in the reinforced zone. The Geopier element stiffness modulus is determined by applying downward pressure on top of a Geopier element in a series of load increments, which are determined from design calculations. Loads are applied using a hydraulic jack and load frame as shown on Figure 6. The measured deflection for each load increment is then plotted against the stress for that increment. The modulus used for design is equal to the design stress divided by the corresponding deflection at that stress. The load test procedure is based on portions of ASTM D 1143 and ASTM D 1194. The modulus test is not performed to determine bearing capacity, but rather to determine stiffness (modulus) of the pier at design stress to be used for settlement estimates. Thus, it is not necessary to extend the test to 150% to 200% of the design stress. The project modulus test data (Stress vs. Deflection curve) is shown on Figure 6. The test results indicated a Geopier modulus of 223 pci at a top of Geopier stress of 27 ksf (132 kips).

In the proposed north parking area within Lake Success, it was decided to do a limited surcharge program and not use Geopier elements due to cost considerations. A minimum 6 ft. surcharge above the new fill was proposed. The primary purpose of the surcharge was not to simulate anticipated loads but to accelerate the time rate of settlement. Per our settlement estimates, up to 12" of settlement was calculated and up to 6 to 9 weeks for 90% of the settlement to occur. An additional 1.0 to 1.5" of secondary compression was estimated due to presence of organics. However, due to presence of up to 30 ft. of new engineering fill which would arch over the foundation soil settlements, significant impact to paving was not anticipated. Some continued pavement maintenance and limited cracking was a distinct possibility.

Settlement Monitoring Program

The objective of the settlement monitoring program was to monitor the settlement of the foundation soils during engineered fill placement, to monitor (to a limited extent) the settlement/internal compression of the new engineered fill, and to evaluate the effectiveness of the geopiers in reducing the fill induced settlement of foundation soils. Settlement monitoring was performed using a combination of conventional settlement plates and vibrating wire settlement systems. A total of five conventional settlement plates (P-1 to P-5 on Figure 7) and three vibrating wire settlement systems (VP-1, -2, and –3 on Figure 7 were used. Two vibrating wire settlement systems (VP-1 and VP-2) and two settlement plates (P-3 and P-5) were located within the northern portion of the building footprint where the foundation soils were improved using geopier elements. Three settlement plates (P-1, P-2, and P-4) and one vibrating wire settlement system (VP-3) were located within the north parking lot where a surcharge program was used.

The conventional settlement plates consisted of a 2 ft. x 2 ft. square, ¾" thick plywood base with concentric 3" outer and 1-½" inner PVC riser pipes. The vibrating wire settlement systems were Geokon Model 4650 manufactured and calibrated by Geokon, Inc., Lebanon, New Hampshire, USA.

The riser pipes of conventional settlement plates often hinder engineered fill placement operations and consequently become "targets" of earthworking equipment. Vibrating wire settlement systems were selected because they offer the advantage of remote monitoring of settlement without impeding fill placement operations. In addition, vibrating wire instruments use a frequency, rather than a voltage, as the output signal from the transducer. Frequencies can be transmitted over long cable lengths without appreciable degradation caused by variations in cable resistance which could be caused by water penetration, temperature fluctuations, contact resistance, cable splicing, or leakage to the ground. The rugged construction of vibrating wire transducers results in their excellent zero stability.

The settlement plates and vibrating wire settlement systems were installed during June, 2001. The elevation at which the settlement plates/systems were installed was determined using standard level surveying procedures and is summarized in the table below.

Instrument ID	Plate Elevation (ft.)
VP-1	580.71
VP-2	577.87
VP-3	575.11
P-1	573.03
P-2	573.02
P-3	579.44
P-4	580.7
P-5	583.13

The field installation of the vibrating wire settlement systems and reading of initial base line readings was done in accordance with the manufacturer's recommendations. The instrument cables were directly buried in shallow trenches and led toward a timber post-supported reservoir/monitoring station located beyond the fill area. The data acquisition was performed at discrete time intervals during fill placement using a Geokon vibrating wire readout (Model GK-403). Data reduction was performed in accordance with the manufacturer's recommendations.

The fill placement operations were completed in August, 2001. The settlement plates were monitored up to October 1, 2001 and the vibrating wire systems were monitored until October 9, 2001. The results of the monitoring program are summarized in the following tables and Figures 8 and 9.

Instrument ID	Plate Elevation (ft.)	Maximum Fill Height (+/-) ft.	Total Settlement (in.)
VP-1	580.71	22	5.98
VP-2	577.87	26	4.98
VP-3	575.11	31	10.56
P-1	573.03	27	2.99
P-2[a]	573.02	20	6.48
P-3	579.44	24	3.12
P-4[b]	580.7	23	9.44
P-5	583.13	20	6.99

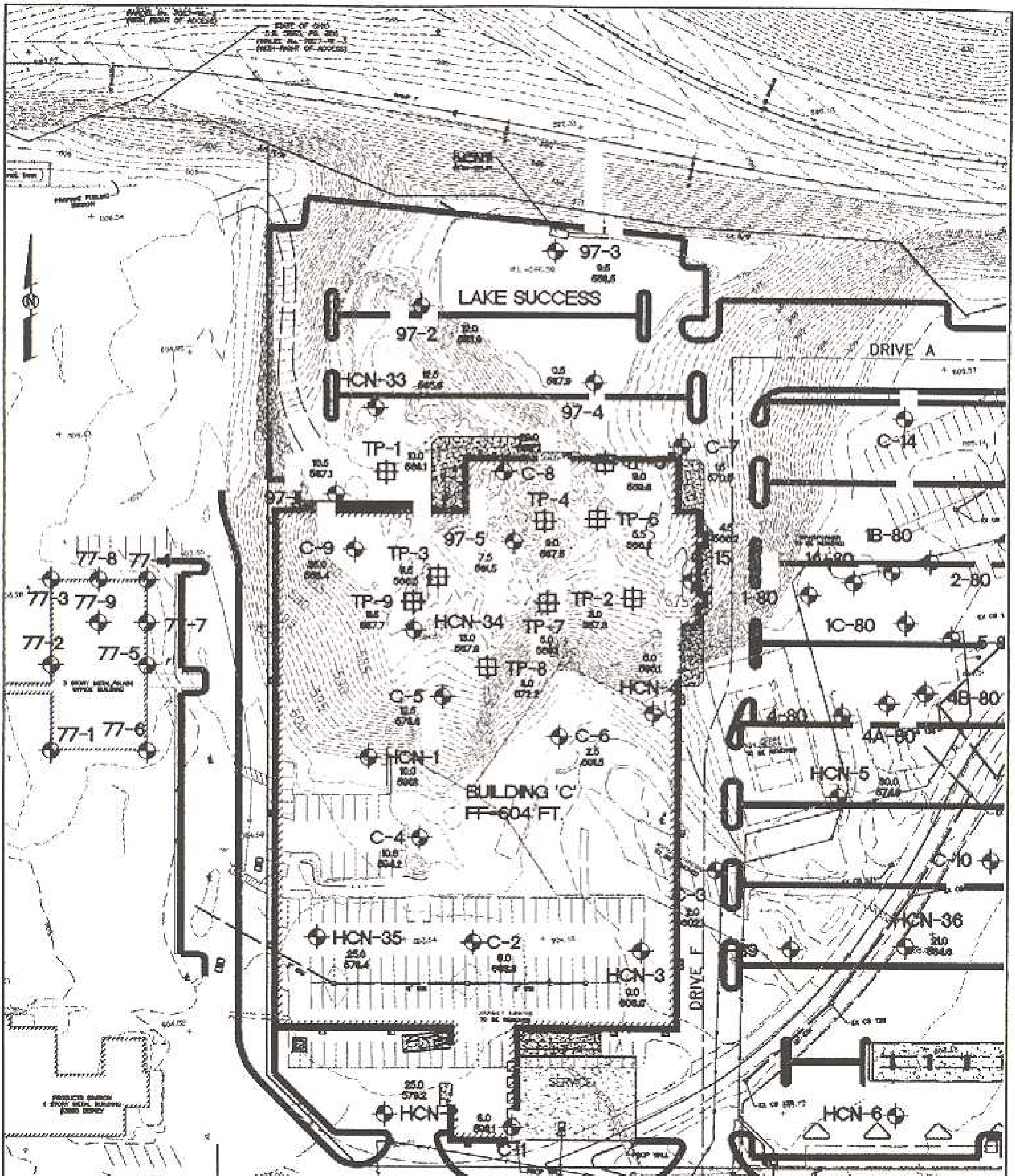
a P-2 broken after 7/31/01
b P-4 broken on 8/8/01, reinstalled

Observations and Conclusions

Figure 8 is a graphical presentation of the vibrating wire settlement system monitoring data. Up to a fill height of about 20 ft., the settlement of the foundation soils was observed to be comparable in the Geopier reinforced and non-Geopier reinforced areas. After that, the magnitude of settlement was observed to be greater in the unreinforced area. The Geopier elements appear to provide the necessary reinforcement to reduce the settlement magnitude.

A comparison of the slope of the settlement curves after site grading was completed (constant fill height) indicated the slope in the unreinforced area was generally steeper than the reinforced area. Sixty days after completion of fill the Geopier reinforced areas settled an additional 0.75" while the unreinforced parking area settled 1.75" and additional movement continued to occur. This finding suggests that the majority of (approximately 85%) settlement in the reinforced area is relatively more immediate in nature. The open-graded aggregate in the Geopier elements appear to significantly accelerate the time rate of settlement by enhancing drainage.

The settlement plate data (shown graphically on Figure 9) also indicated that the settlement magnitude, for comparable fill heights, was generally less in the Geopier reinforced area than the unreinforced area.



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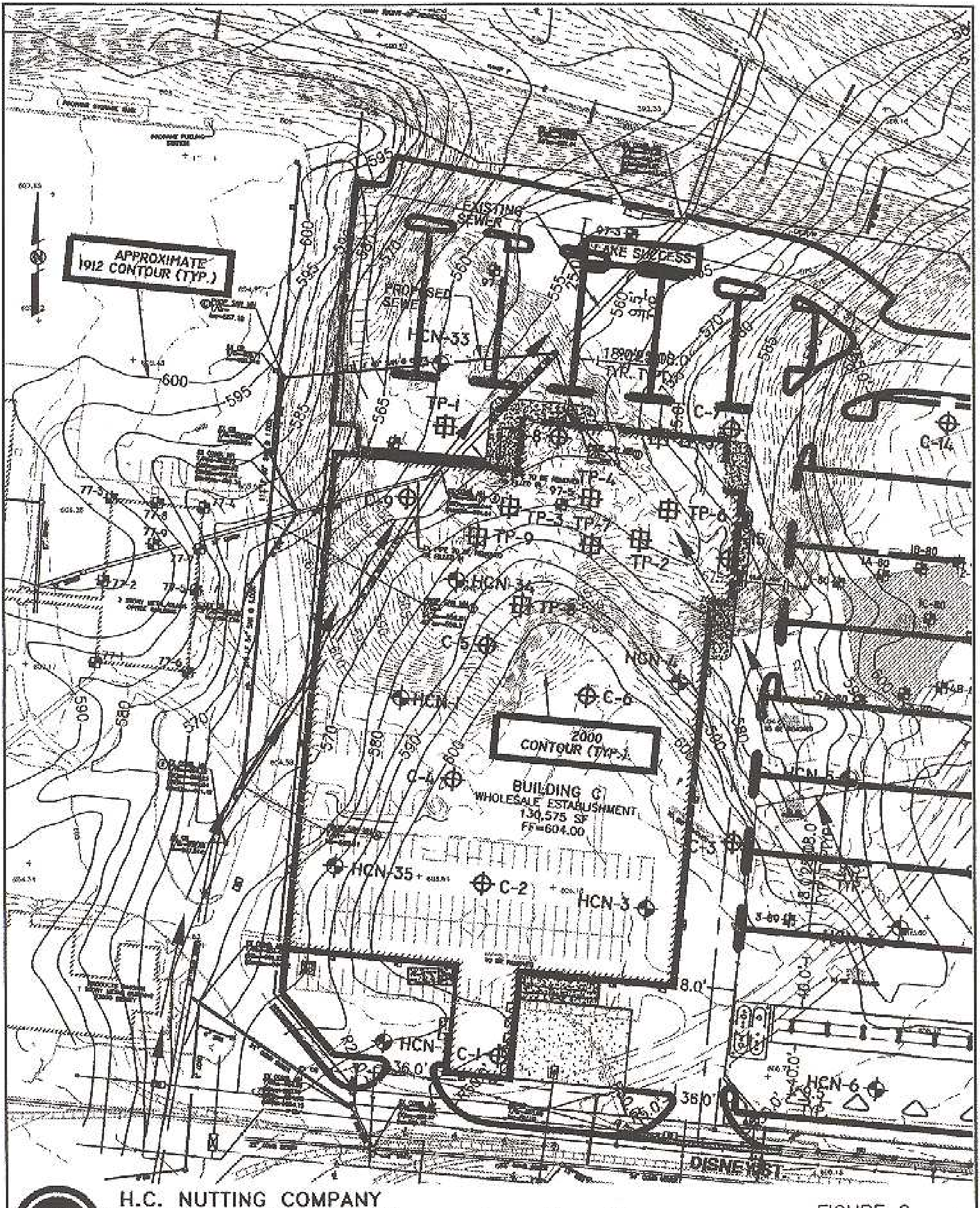
0 50 100
APPROXIMATE SCALE IN FEET

FIGURE 1

TEST BORING AND TEST PIT LOCATIONS



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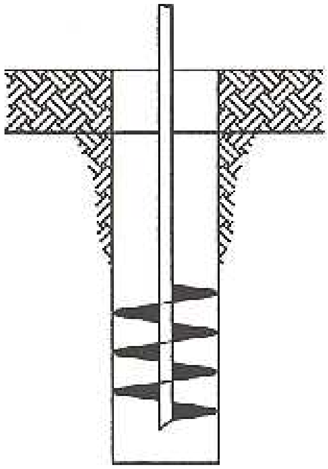
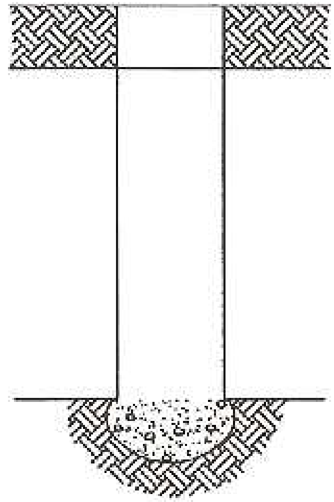
0 50 100
APPROXIMATE SCALE IN FEET

FIGURE 2

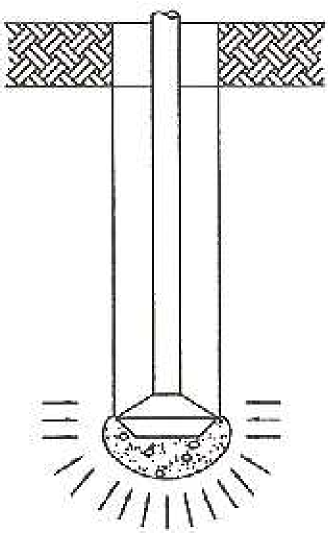
1912 TOPOGRAPHIC SURVEY



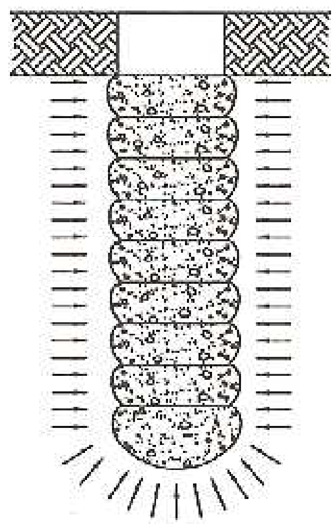
1. Make cavity



2. Place stone at bottom of cavity.



3. Make a bottom bulb. Densify and vertically prestress matrix soils beneath the bottom bulb.



4. Make undulated-sided Geopier shaft with 12-inch (or less) thick lifts. Build up lateral soil pressures in matrix soil during shaft construction. Use well-graded base course stone in Geopier element shaft above groundwater levels.

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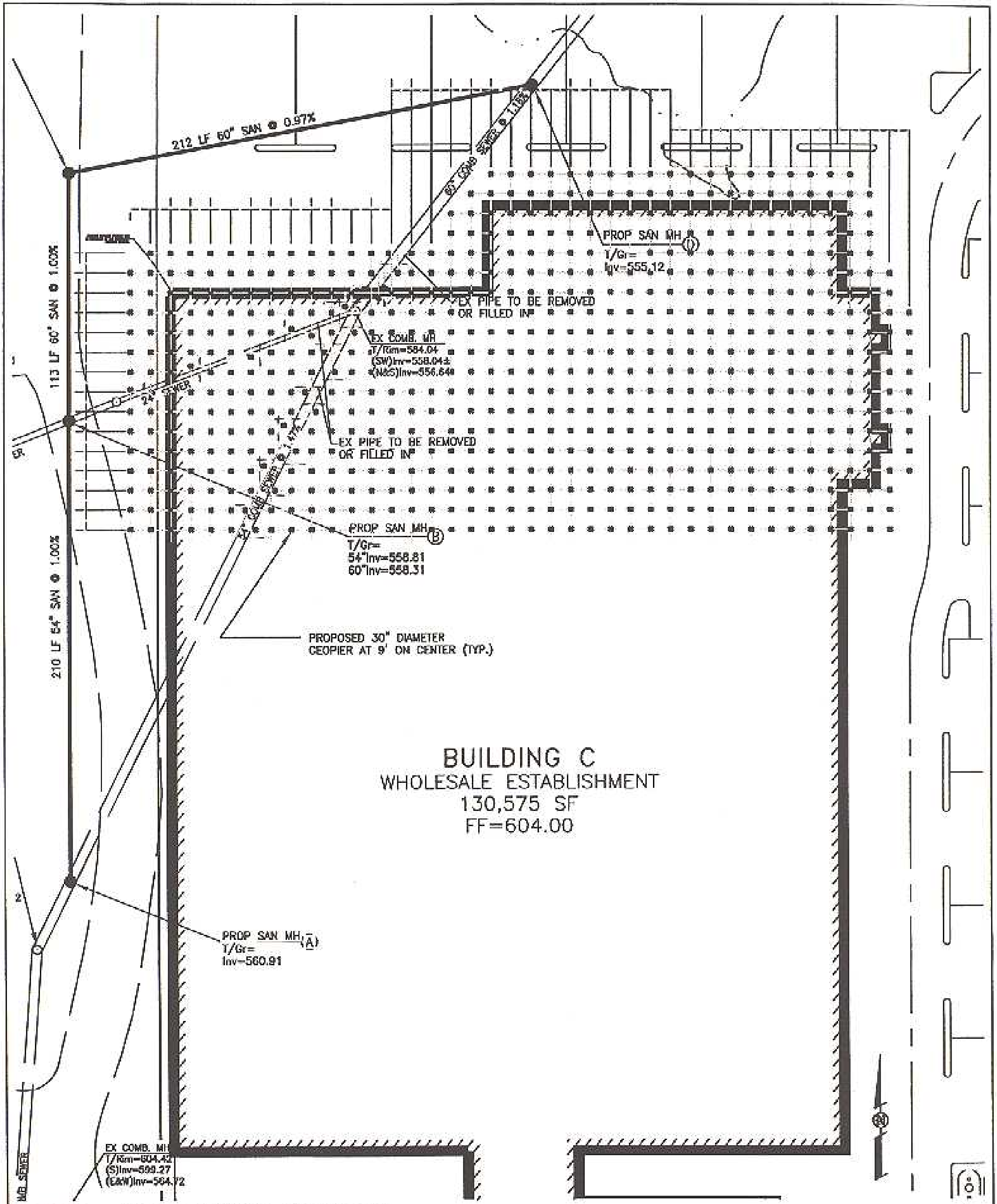
NOT TO SCALE

FIGURE 3

GEOPIER INSTALLATION PROCESS



HCN

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FIGURE 4

GEOPIER LAYOUT

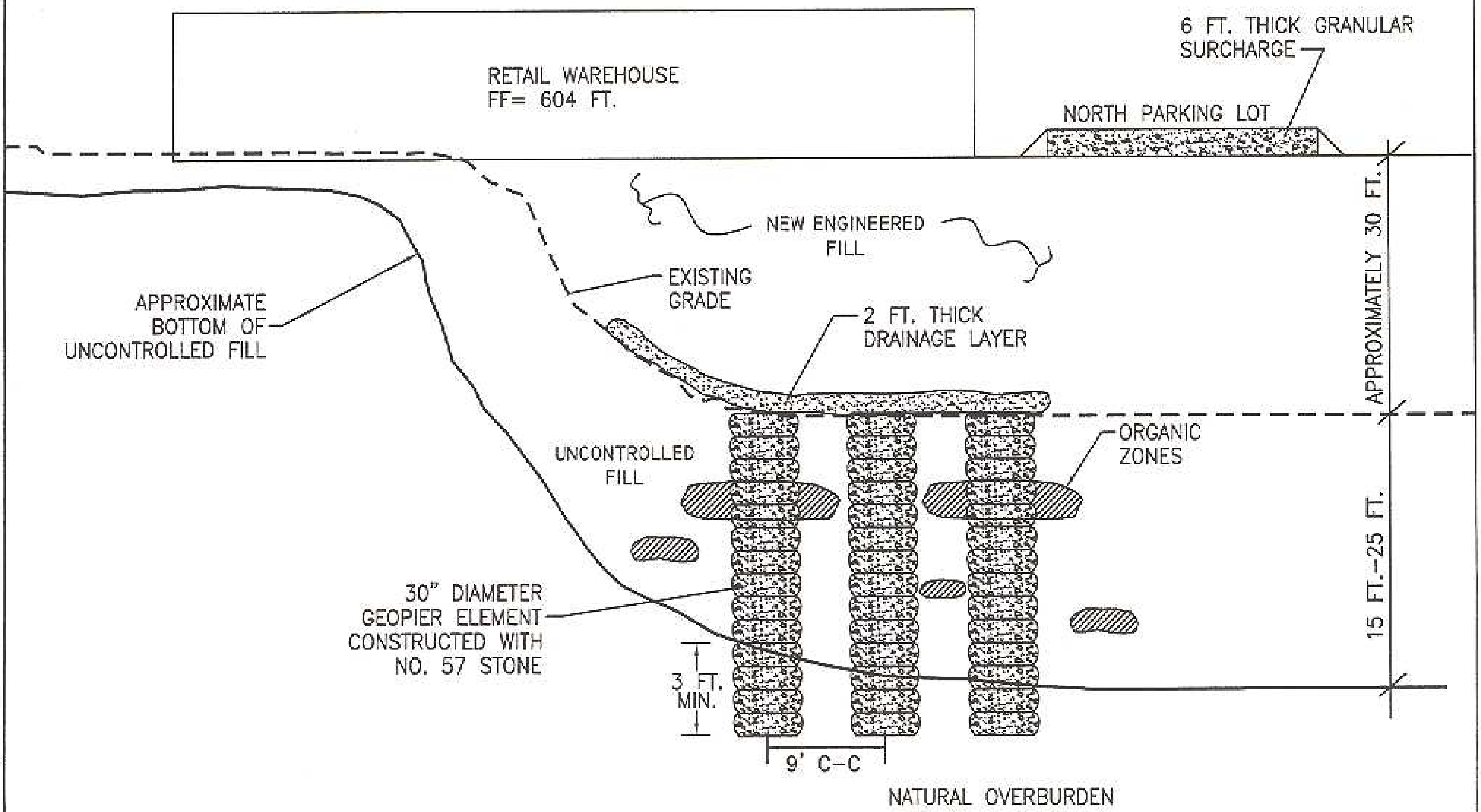

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FIGURE 5

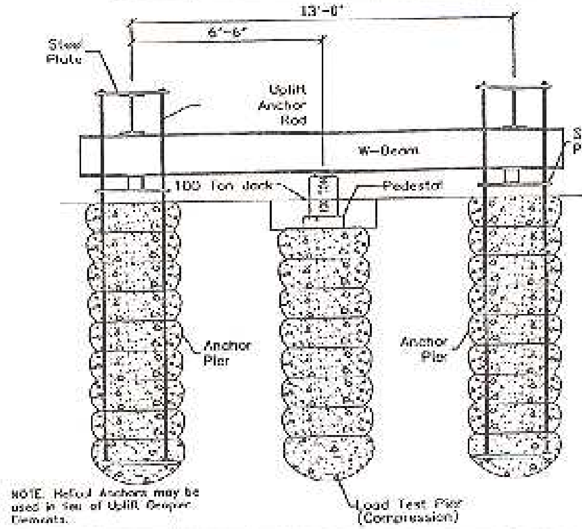
SCHEMATIC REPRESENTATION OF GROUND IMPROVEMENT

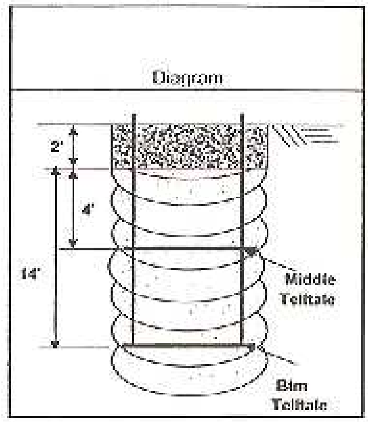
geopier® ***GEOPIER* Foundation Company - MidSouth, LLC**

MODULUS TEST RESULTS

Project:	Building "C" - the Center of Cincinnati	**Design Stress:**	30000 psf
Location	Cincinnati, Ohio	**Geopier Diameter:**	30 inches
Project Number:	PLM-00056	**Geopier Length:**	14 ft.
Date of Installation:	May 15, 2001	**Date of Test:**	May 19, 2001
Location:	21-ft east of Boring C-15	**Nearest Boring:**	H.C. Nutting Boring C-15
PCI Representative:	Walt Jackson	**QA Consultant:**	H.C. Nutting

General Requirements: Maximum deflection not to exceed 3.0 inch at 100% of the design load.





Geopier Modulus Load Test
Deflection vs. Geopier Stress

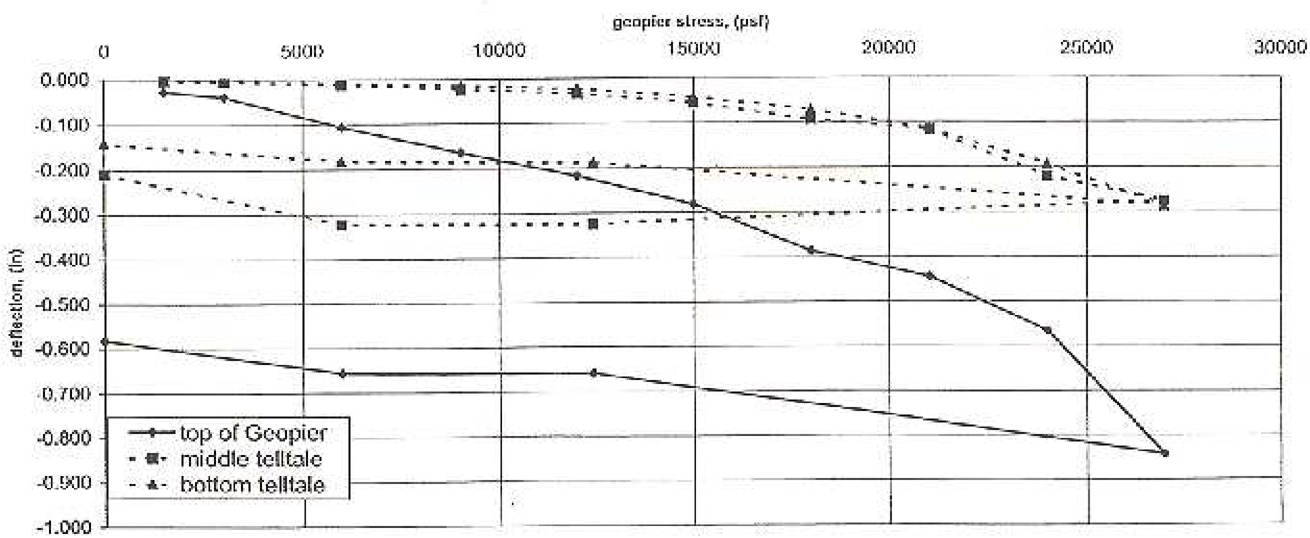


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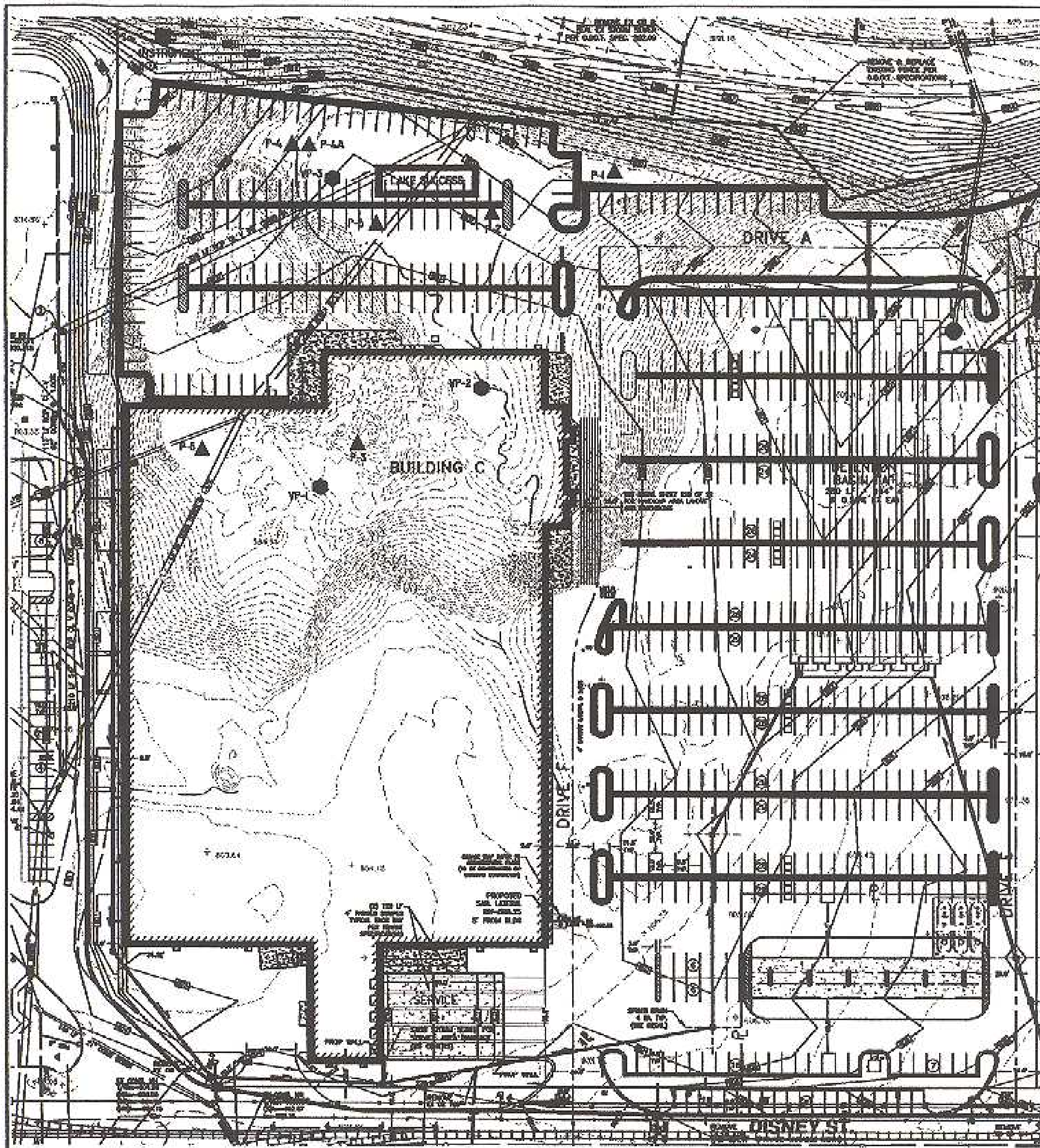
FIGURE 6

GEOPIER LOAD TEST SETUP AND RESULTS



NOTE: P-4A LOCATED 4' EAST OF P-4

▲ CONVENTIONAL PVC SETTLEMENT PLATE

● VIBRATING WIRE PLATE

■ DATA AQUISITION BOX

0 50 100

APPROXIMATE SCALE IN FEET

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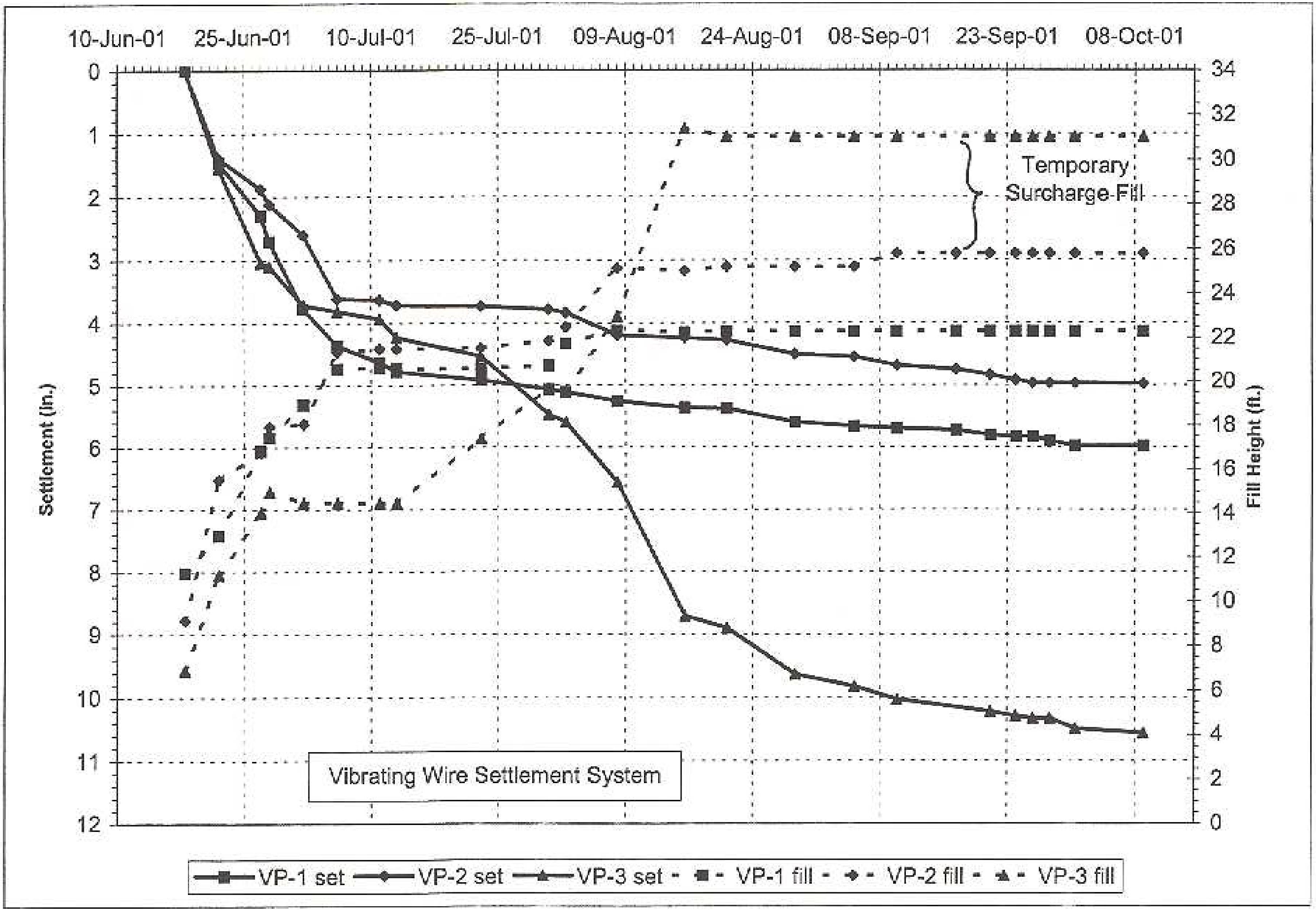
FIGURE 7

SETTLEMENT MONITORING LOCATIONS


10-Jun-01
25-Jun-01
10-Jul-01
25-Jul-01
09-Aug-01
24-Aug-01
08-Sep-01
23-Sep-01
08-Oct-01
0
1
2
3
4
5
6
7
8
9
10
11
12
Settlement (in.)
34
32
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
Fill Height (ft.)
Temporary
Surcharge Fill
Vibrating Wire Settlement System
VP-1 set
VP-2 set
VP-3 set
VP-1 fill
VP-2 fill
VP-3 fill

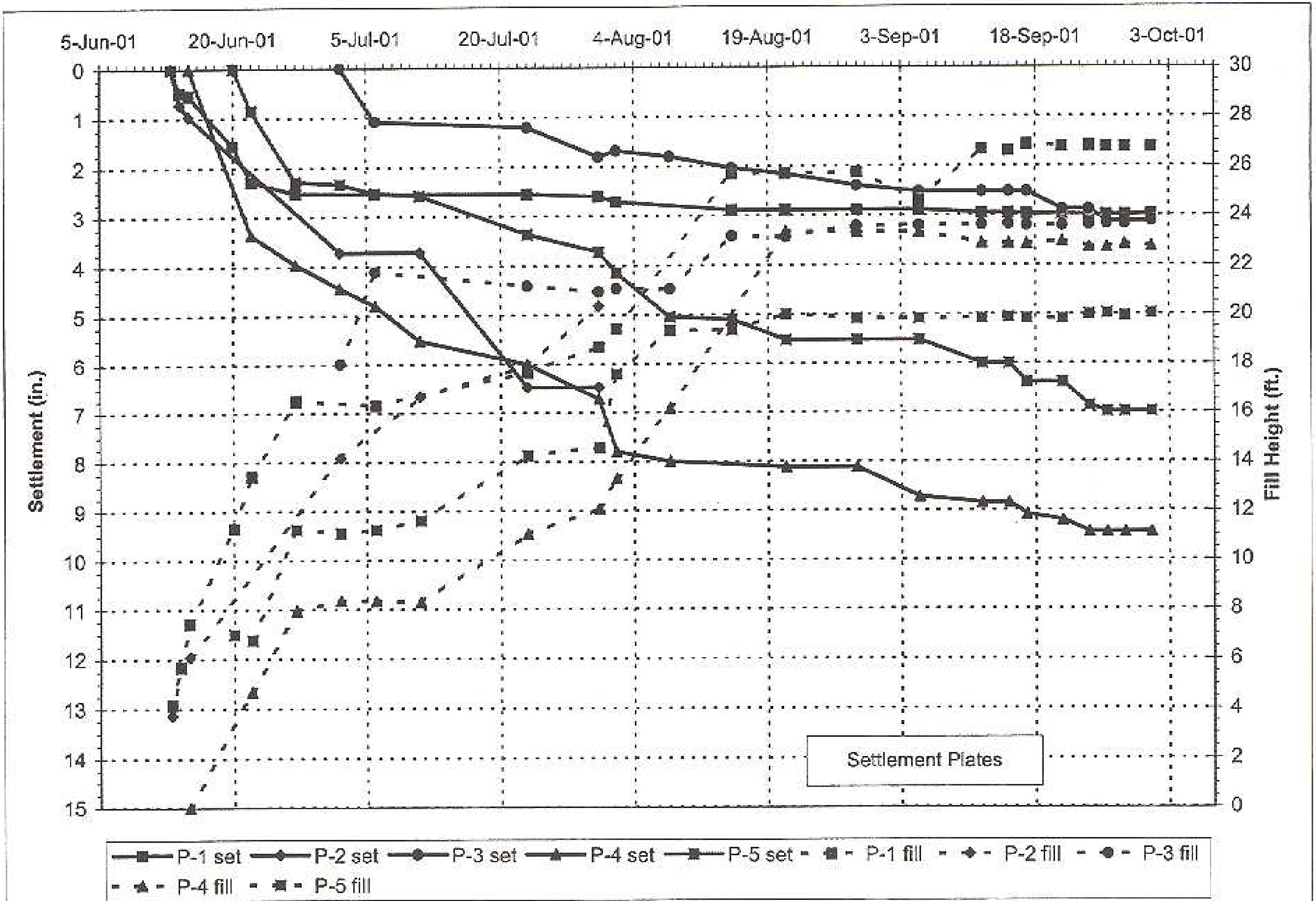

5-Jun-01
20-Jun-01
5-Jul-01
20-Jul-01
4-Aug-01
19-Aug-01
3-Sep-01
18-Sep-01
3-Oct-01
Settlement (in.)
Fill Height (ft.)
Settlement Plates
P-1 set
P-2 set
P-3 set
P-4 set
P-5 set
P-1 fill
P-2 fill
P-3 fill
P-4 fill
P-5 fill

FIGURE 9