Monitoring of Surcharge-Induced Settlement at the MARTA Chamblee Station

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A portion of the proposed location of the Metropolitan Atlanta Rapid Transit Authority (MARTA) Chamblee Station was underlain by uncontrolled fill and soft sandy alluvial soils atop residual Piedmont soils. Due to high groundwater conditions and adjacent railroad tracks, it was not considered viable to excavate and replace these soils. Since this area was to support a Reinforced Earth (TM) wall and the station platform, deep foundations were considered unacceptable due to concerns over differential settlements between the wall, trackway, and platform. The original design called for dewatering, placement of fill, and a surcharge load to preconsolidate the soils. The area was to be monitored with settlement platforms and observation wells that were to be extended as fill was placed. The authors jointly devised a plan to utilize vibrating wire settlement transducers and piezometers to monitor pore pressure dissipation and consolidation. These instruments were used in conjunction with consolidation tests to evaluate the rate of consolidation. Predicted magnitudes and rates of settlement are presented and compared to field measurements.

The Metropolitan Atlanta Rapid Transit Authority (MARTA) has completed 29 rapid rail stations and 32 miles of heavy rail dual trackway. The transit system currently consists of four branches emanating from a central hub in the center of Atlanta. The branches currently extend to the northeast, south, east, and west. The MARTA northeast rail line passes through the suburban city of Chamblee, Georgia. For several miles the rapid rail line parallels the Norfolk Southern Railway, which generally follows a northeast-southwest trending topographic ridge. In the vicinity of the Chamblee Station, the northeast MARTA rail line lies immediately southeast of the railroad.

SURFACE AND SUBSURFACE CONDITIONS AT CHAMBLEE STATION

Chamblee is located within the Piedmont physiographic province. The Piedmont is known for its residual soils weathered from underlying metamorphic and igneous rocks consisting of gneisses, schists, and granites. The residual soils consist mainly of sandy silts and silty sands, with clays near the surface. The Chamblee Station platform area is located in a previously existing low area that drained toward the northwest. The ground surface elevation before construction was approximately 1,021 ft.

A subsurface investigation revealed fill overlying most of the site. This fill was apparently placed for the railroad in about 1870, before the availability of heavy compaction equipment. Thus, the fill was not compacted to a significant degree and typically consisted of very loose to loose sand or soft silt, with standard penetration resistances ranging from 3 to 10.

Alluvial soils were discovered underlying the fill. These soils ranged in thickness up to 9 ft and were highly variable in composition. The alluvium consisted of clayey and silty sands, silts and clays, with occasional inclusions of fine organic matter. The deepest fill and alluvial soils were encountered down to approximately elevation 998 ft.

Loose residual soils were found directly beneath the fill and alluvium. A consolidated undrained triaxial test performed on a residual soil sample from Boring NCH-287 indicated an angle of shearing resistance of 33 degrees and a cohesion of 250 lb/ft^2. Atterberg limit testing indicated that the sample was nonplastic. Table 1 presents a summary of laboratory data for the soils tested at the site.

Partially weathered rock marked the transition from the residual soils to solid rock. The underlying rock was generally biotite gneiss. The groundwater level measured during the subsurface investigation was near elevation 1,015 ft at the site.

DESIGN CONSIDERATIONS FOR PLATFORM FOUNDATION

The station platform was 600 ft long, with the "north" end of the station actually pointing toward the northeast (see Figure 1). The entire station platform was originally intended to be part of an aerial structure supported on deep foundations. However, estimates indicated that at-grade track on embankment was less costly. The at-grade trackway on embankment required a retaining wall to separate the elevated platform from the east parking lot, access road, and east busway. This wall was designed as a Reinforced Earth (TM) wall to extend from the abutment at the south end of the platform to the north concourse. The height of the wall ranged from about 20 ft to 30 ft.

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TABLE 1 SUMMARY OF LABORATORY DATA

BORING NUMBER	SAMPLE ELEVATION (FEET)	SAMPLE CLASSIF.	MOISTURE CONTENT (%)	PERCENT FINER NO. 200 SIEVE	COMPR. INDEX	INIT. VOID RATIO
NCH-287	1008	RESIDUAL- SILTY SAND	35	39	N/A	0.95
NCH-290	1015	ALLUVIUM- SILTY SAND	16	48	N/A	N/A
NCH-291	1001	FILL- SANDY SILT	33	50	N/A	N/A
NCH-292	1018	FILL-SILTY CLAYEY SAND	28	39	0.19	0.90
NCH-292	1008	FILL-SILTY CLAYEY SAND	31	34	0.20	0.91
NCH-292	1002	FILL-SILTY CLAYEY SAND	22	38	0.11	0.65
NCH-294	1002	RESIDUAL- SILTY SAND	52	45	N/A	N/A

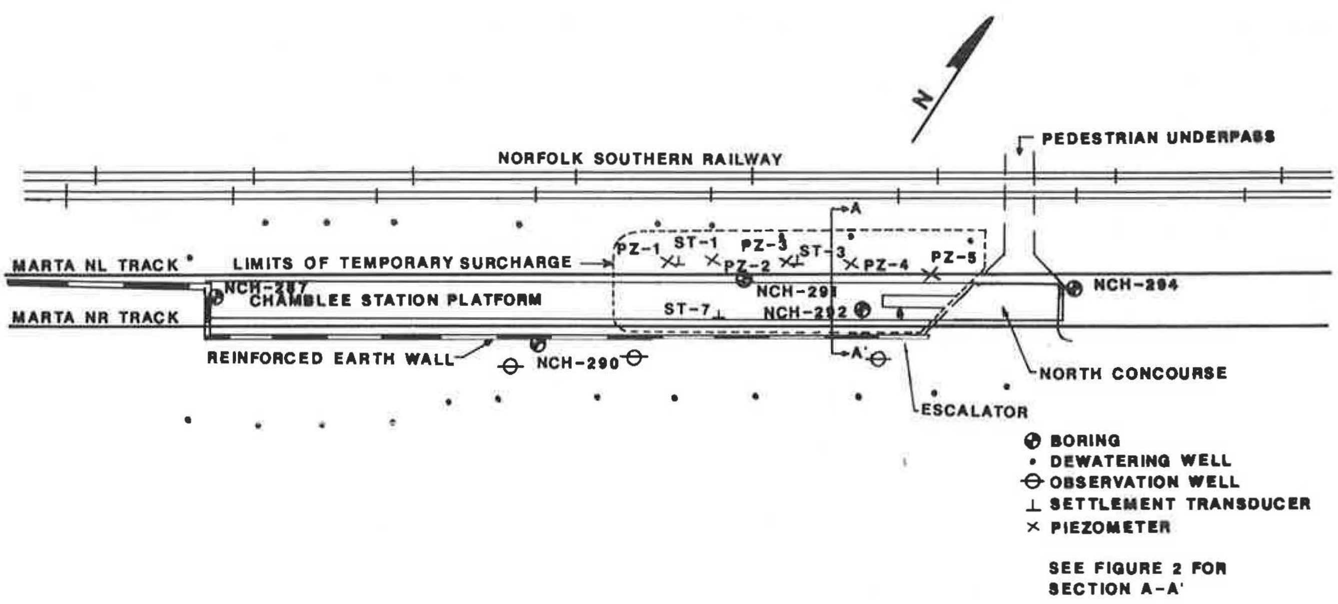

FIGURE 1 Plan of surcharge area.

The presence of the soft fill and alluvium under the proposed platform caused concern about differential settlement between the platform and trackway. Structural requirements dictated that any differential settlement be held to ¼ in. A ballasted trackway would have allowed readjustments of the track for differential settlement. However, a direct fixation trackway on a concrete slab was selected to prevent shifting of the track, which can occur with ballasted trackway. A direct fixation trackway is difficult to repair if excessive settlement occurs. Therefore, for a direct fixation trackway to be a viable solution, careful geotechnical analysis and design were required to limit differential settlement.

A temporary surcharge was chosen as the least expensive and most effective solution for preparing the subgrade to support the embankment and trackway. A temporary surcharge was initially considered for the entire length of the platform. However, within the southern section of the platform it was possible to excavate almost all of the 5 ft thickness of existing loose fill without penetrating the railroad influence line (Figure 2), below which excavation supports would have been required. Figure 1 shows the area encompassed by the temporary surcharge. Figures 3 and 4 show placement and the final configuration of the surcharge.

FIGURE 3 Placement of surcharge.

DETAILS OF CHAMBLEE STATION TRACKWAY SUPPORT

The MARTA northbound track was located immediately behind the Reinforced Earth wall, bearing on the zone of internal reinforcing and select backfill. The Chamblee Station contained a center platform, supported mainly by two low walls bearing on compacted embankment. The northern end of the platform was supported on a structure bridging the north concourse. Each track slab was composed of concrete 9 in. thick and 11 ft wide. A canopy was supported by the platform which was founded on spread foundations bearing on underlying compacted structure embankment (see Figure 2).

Near the south abutment of the north concourse (which was supported by piles), the surcharge could not be placed to full height. Removal and replacement of the existing compressible fill and alluvium was required to minimize anticipated differential settlement in this transition area from embankment support to pile support.

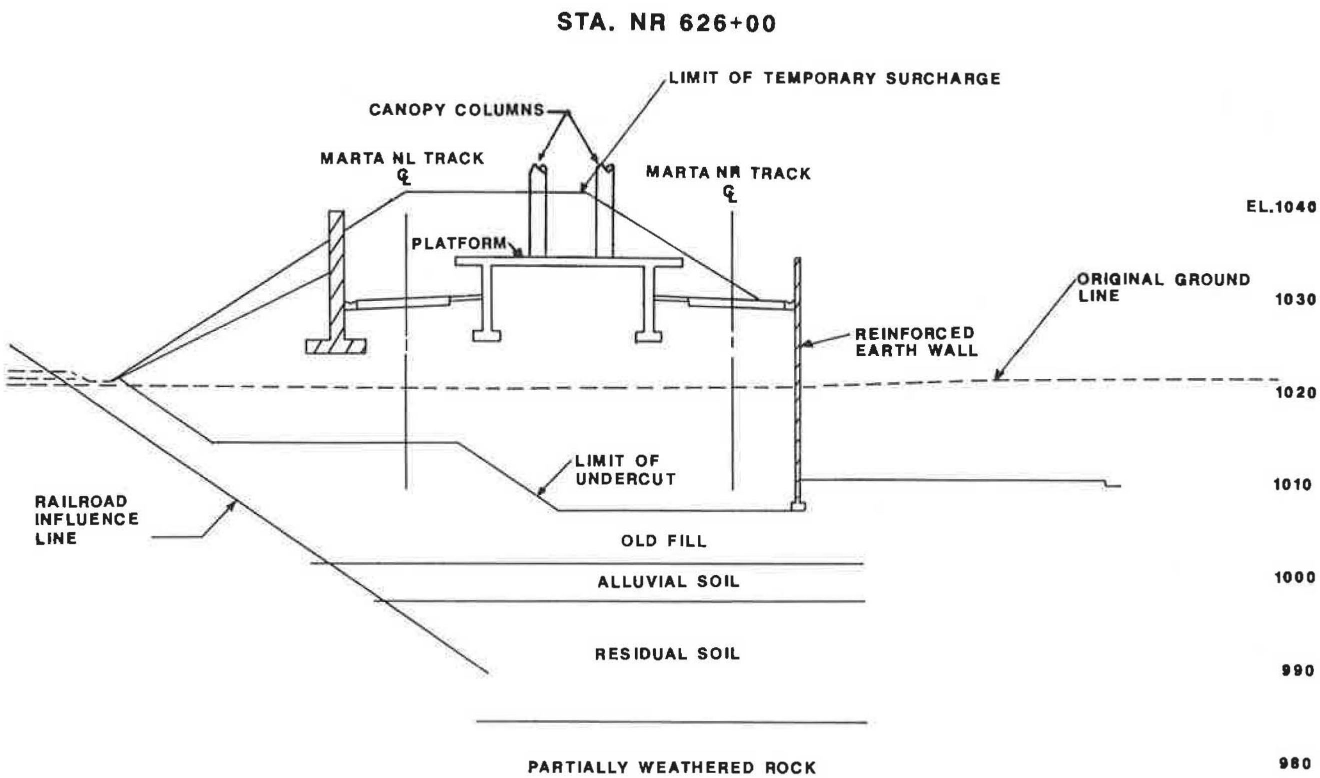

FIGURE 2 Cross-section of station area (section A-A′).

FIGURE 4 Final configuration of surcharge.

DEWATERING SYSTEM

Specifications required lowering the groundwater to at least 3 ft below the bottom of excavation, and keeping the groundwater lowered at least 3 ft below the excavation bottom at any given time. This specification was established to limit loosening of the soil due to upward seepage pressures.

The general contractor hired a specialty contractor to design and install the dewatering system. The system included about 50 deep wells in two rows. The rows were 85 to 130 ft apart. One row of wells was located on each side of the platform area and extended to the north and south. Wells within each row ranged from 44 to 58 ft apart. The system also included 6 observation wells, which were cut off as the excavation proceeded (see Figures 1 and 5). Estimated single well flows were 3.5 gal/min, and 3 to 4 weeks of pumping were anticipated to dewater the site.

All dewatering wells were drilled 10 to 30 ft into rock, terminating at depths varying from 53 to 94 ft. The wells were completed and pumping began in late February 1986. Initial flows varied from less than 1 gal/min to 18 gal/min. Wells with yields less than 1 gal/min were abandoned. The entire system pumped about 250,000 gal of water per day. Water levels were lowered significantly within 10 days, which was much faster than anticipated. Pumping rates were reduced to maintain the drawndown water level to just below subgrade elevation (989 ft).

ESTIMATES OF SURCHARGE REQUIREMENTS

The Reinforced Earth wall and compacted embankment imposed calculated loads of about 1,000 lb/ft^2 on the subgrade materials. Anticipated final loads on the trackway subgrade, including track slab, train, platform, and canopy, totaled about 560 lb/ft^2. The surcharge height of 8 ft was of limited width and was calculated to impose a pressure of 800 lb/ft^2. Assumed dewatering to a shallow depth was calculated to impose an effective pressure of 470 lb/ft^2. These total pressures were calculated to induce approximately the same settlements in 8 weeks that the final loads would have induced at the end of primary consolidation.

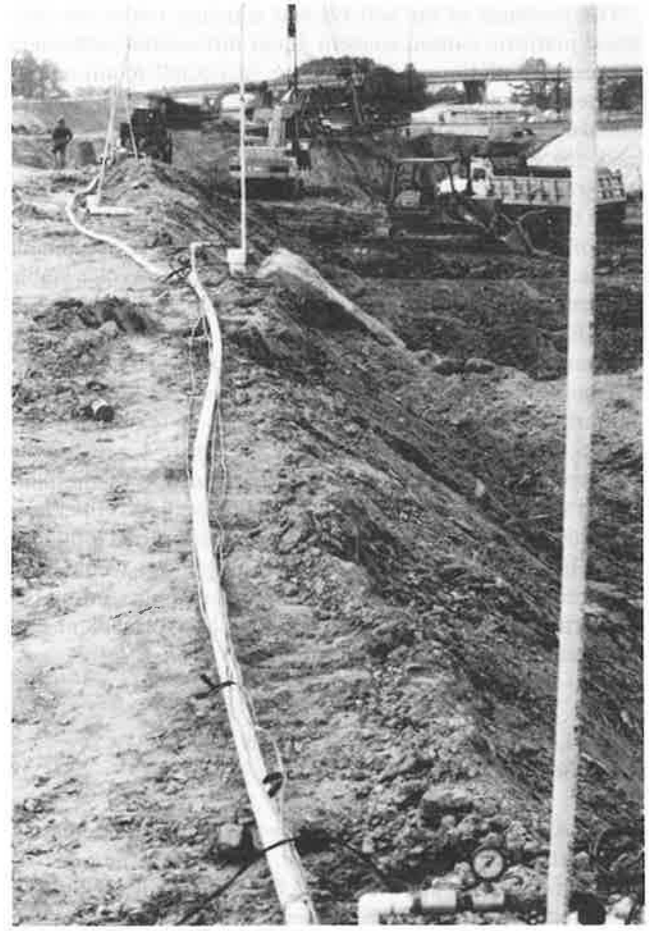

FIGURE 5 Dewatering system.

No vertical drains were planned for hastening the consolidation of the compressible materials. Settlements of the compressible materials left in place were calculated based on consolidation testing data to be between 3 and 4 in. for primary consolidation. Total compression of the material left in place and of the new embankment was expected to be 5 to 6 in., some of which would occur as the embankment was constructed. No values of pore pressure changes were predicted, since the soils were mostly unsaturated.

Estimates of the time required for the consolidation to occur were performed using consolidation test data. The depth of compressible material was highly variable across the site. Estimates for the time required for 90 percent of primary consolidation varied from 2 to 5 months. The contract time allowed for the consolidation was set at 3 months, based mainly on experience.

The contractor proposed a deep dewatering system as opposed to the shallow system assumed in the initial settlement estimates. Deep dewatering lowered the groundwater level below the compressible material, increasing the settlements of the compressible materials and reducing the required time of surcharge. The effective surcharge due to deep dewatering was calculated to be 940 lb/ft^2. For the deep dewatered condition, the time span required for 90 percent of primary consolidation was calculated to be 8 weeks. Therefore, installing the deep well system reduced the estimated time for the surcharge to remain in place from 3 months to 2 months. Based on the added effect of the deep dewatering, the contractor requested that the surcharge be deleted. The request was denied, but the required surcharge time was reduced to 8 weeks.

Even with the surcharge, some long-term consolidation of the soils was expected to occur due to secondary consolidation. In general, settlements from secondary consolidation were expected to be broadly distributed and not to result in significant differential settlement between the platform and trackway.

REQUIREMENTS FOR INSTRUMENTATION

Monitoring the progress of settlement and pore pressure dissipation under the temporary surcharge was an integral part of the design of the MARTA station. The instrumentation data was used to decide whether the planned duration of the surcharge was sufficient or not. If the data indicated that the surcharge had its desired effect earlier than anticipated, then the surcharge could be removed, allowing the contractor access to the area before the scheduled date. MARTA intended for the surcharge to remain in place until both a zero rate of settlement was attained and the pore pressures recovered to their presurcharge levels.

CHOICE OF INSTRUMENTATION

The original instrumentation plan called for the use of horizontal and vertical inclinometers installed within the select backfill of the Reinforced Earth wall. Vertical inclinometer casings were to be installed just behind the Reinforced Earth wall and were to extend upward as the select backfill was placed. Horizontal inclinometer casings were to be installed in trenches in the existing fill and alluvium under the Reinforced Earth backfill and were to extend under the Reinforced Earth wall footing. The inclinometers were to be supplemented by placing settlement plates on the subgrade, welding pipe risers to the plates and extending the risers upward to the top of the select backfill as fill was placed. The risers were to be monitored by optical survey. Groundwater levels were to be monitored via open standpipe piezometers installed in the fill and likewise extended as select backfill placement continued. However, when the bids were opened, the lowest bid for constructing the station was found to be considerably over MARTA's estimate. Many items, including all of the vertical inclinometers, were eliminated from the contract to reduce costs.

The area of Reinforced Earth backfill presented a very confined work area. It was recognized that the settlement risers and open pipe piezometers would have a very high potential for damage during fill placement. The project specifications required the contractor to replace any damaged units at no additional fee and to pay a fine of $1,000.00 per unit per damage incident. These factors led to the consideration of alternate approaches to the monitoring program.

For settlement readings, consideration was given to the specified horizontal inclinometer system, along with alternate profiling systems utilizing pneumatic and vibrating wire probes. However, due to the small size of the study area, it was decided that settlement profiles were not particularly cost effective, and that discrete measurements at three or four locations within the work area would provide the necessary information. The instruments finally chosen for monitoring settlement were the Model 4600 vibrating wire settlement sensors manufactured by Geokon, Inc. These instruments were supplemented at a later stage by four standard settlement risers, which were installed from the surface of the completed surcharge down into the compacted trackway subgrade and subsequently monitored by optical survey.

The vibrating wire settlement system consisted of a vibrating wire pressure transducer connected via a fluid-filled tube to a reservoir. The transducer sensed the pressure created by the head of the fluid within the tube. Changes in head (pressure) provided a measure of the difference in elevation between the reservoir and the sensor. The pressure transducer was located in stable ground, and the reservoir was attached to a plate located at the top of the borehole, which settled along with the material around it. The transducers used were capable of discerning settlement changes with an average resolution of 0.047 in. An electrical cable extended from the sensor to a remote readout location (Figure 6). This arrangement called for the drilling of a borehole, but it avoided the need to run fluid-filled tubes laterally through the fill. The instrument is supplied filled and ready for installation, so there was little concern about performance of the system with respect to a discontinuous fluid column. Simple electrical continuity checks made on site prior to installation confirmed a continuous fluid column.

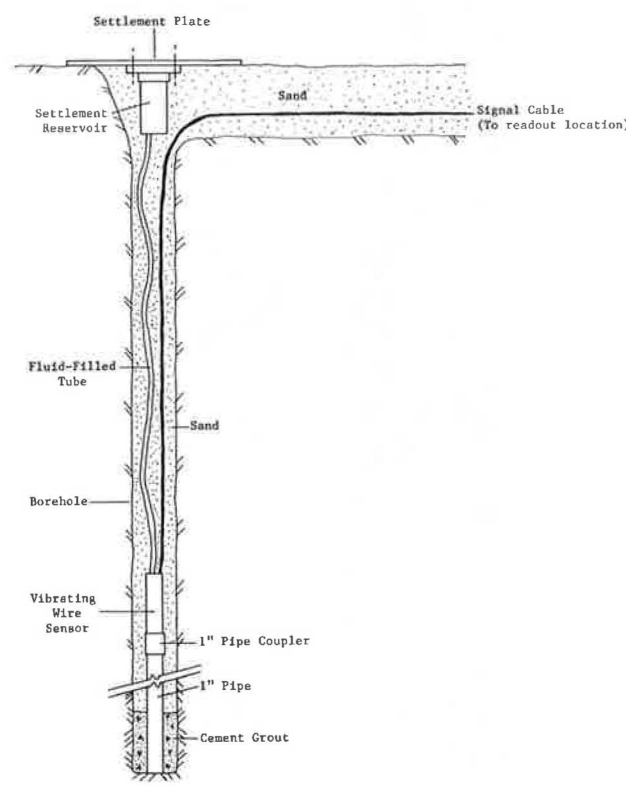

FIGURE 6 Vibrating wire settlement system.

For groundwater monitoring, pneumatic and vibrating wire piezometers were considered. The final decision was to use the vibrating wire units so that the same switching terminal and readout required for the settlement monitoring system could be used for groundwater monitoring. The instruments chosen were the Geokon Model 4500DP drive point piezometers. These piezometers utilized a vibrating wire pressure transducer, mounted to an EW drill coupling with a pointed nose cone attached (Figure 7). The average sensitivity of the piezometers was 0.023 pounds per square inch (psi). The alternate instrument selections were all submitted to MARTA and accepted.

INSTALLATION OF INSTRUMENTATION

The project specifications required the general contractor to procure the services of a specialty contractor or consultant to install all instruments. The general contractor chose to expand the duties of the consultant to include additional laboratory testing, settlement magnitude, and time frame prediction, in addition to instrument monitoring and interpretation. The installation of the settlement sensors required drilling a borehole through the alluvial soils and underlying soft residual soils into firm rock using hollow stem augers. The sensor was attached to a 1 in. diameter steel pipe and lowered through the hollow auger to the bottom of the hole. The length of steel pipe was selected to keep the elevation difference between the reservoir and transducer small, thus allowing the use of a low pressure range transducer, which resulted in greater sensitivity. A 2 to 3 ft grout plug was placed in the bottom of the hole as the augers were retracted. The hole was then backfilled with sand and the reservoir attached to a 2 ft square, 3/8 in. thick plywood settlement plate. The reservoir-plate assembly was placed over the top of the borehole and the area around the borehole (a 2 ft square by 1 ft deep excavation) backfilled with sand.

The piezometers were placed either by advancing a borehole to the planned elevation and pushing the piezometer into place with drill rods, or by excavating via backhoe to the planned elevation and driving the piezometer into place with a hand penetrometer. In both cases, the hole above the piezometer was sealed with a bentonite plug. Prior to placement, the piezometer filters (1 bar, high air entry ceramics) were saturated and the cavity between piezometer diaphragm and filter purged of air. Cables from all units were buried in trenches and connected to the readout switch panel (Figure 8).

The vibrating wire piezometer and settlement monitoring systems were installed during the week of April 21, 1986. Sixteen optical settlement reference points were also cast into the leveling footing of the Reinforced Earth wall.

INSTRUMENTATION RESULTS

The settlement transducers and pore pressure transducers were read independently by both the general contractor's instrumentation consultant and by MARTA's general engineering consultant. All data were shared by both parties to assure full coverage and consistency. Important dates in the construction sequence of the project are shown on Table 2. The results of the piezometer and settlement transducer data are shown on Figures 9 and 10.

Although the location (elevation) of the piezometers was not changed when the deep dewatering scheme was accepted

FIGURE 7 Vibrating wire piezometer.

FIGURE 8 Rotary switch panel and readout box.

TABLE 2 CALENDAR DATES VERSUS CONSTRUCTION DAYS AND EVENTS

PHASE	CALENDAR DATES	CONSTRUCTION DAYS	CONSTRUCTION EVENT
N/A	April 25-May 9	0	Initial instrument readings
1	May 9	1	Beginning of select backfill placement for Reinforced Earth wall
2	May 9-July 30	1-81	Placement of Reinforced Earth backfill
3	July 30-Aug 7	81-89	Placement of temporary surcharge
4	Aug 7-Sep 16	89-129	Approximate date of pore pressure maximum to removal of temporary surcharge
N/A	Sep 16	129	Final instrument readings

(the piezometers were installed above the drawndown groundwater level), the majority of the piezometers indicated changes in pressure to the applied surcharge loading (Figure 9). The response of piezometer PZ-4 was not as pronounced as the four other units. The reason for the poor response of this piezometer was not clear, but smearing of the ceramic filter was suspected. Since pumping for the dewatering was continuous, the pore pressure measurements indicated a general decreasing trend throughout the life of the project.

The settlement transducers indicated similar settlement patterns and magnitudes. The measured settlements shown in Figure 10 were much greater than originally anticipated. The early start and the depth of the dewatering accomplished by the contractor increased the settlements.

The time period of interest in the surcharge area can be divided into four phases (Table 2), as follows:

1. Phase 1 (April 25–May 9, 1986) began with the first instrumentation readings and ended at the beginning of select backfill placement for the Reinforced Earth wall.
2. Phase 2 (May 9–July 30, 1986) began with the initial placement of the select backfill, continued through the completion of the Reinforced Earth wall, and ended at the beginning of the placement of the temporary surcharge.
3. Phase 3 (July 30–Aug. 7, 1986) began with the initial placement of the temporary surcharge and ended when the pore pressures reached their maximum. Placement of the temporary surcharge began on July 30 and was completed on August 4.
4. Phase 4 (Aug. 7–Sept. 16, 1986) began when the pore pressures reached their maximum and ended when the surcharge was removed.

Typical data from the piezometers and settlement transducers are presented in Figures 9 and 10. Figure 9 represents the change in settlement measured at the site versus time. The data are presented by phase, as described above, so that increments of loading and its effects can be examined. During the course of the project, the settlement and pore pressure data were analyzed versus time using semilogarithmic plots. Discussion of each phase is presented below.

During Phase 1, measured settlements ranged from 1.9 to 3.0 in. Pore pressures measurements were all below atmospheric and decreased by 0.3 to 0.8 psi (Tables 3 and 4).

During Phase 2, settlements increased to accumulations varying from 9.3 to 11.9 in. The rate of settlement varied from about 4 to 7 in. per logarithmic cycle. The pore pressures continued to decline, and initial rates varied from −0.5 to −0.9 psi per logarithmic cycle. The settlements and pore pressures varied more or less logarithmically during this time period. Settlement transducer ST-1 and piezometers PZ-1, PZ-3, and PZ-4 displayed breaks in their curves about June 13. Settlement transducer ST-1 began settling at a slower rate, while piezometers PZ-1 and PZ-3 began showing higher rates of decreasing pore pressures. Instruments ST-1 and PZ-1 were located close to each other, but the cause of these changes in rates is unknown.

During Phase 3, four piezometers showed an increase of pressure, with increments ranging from +0.75 to +1.25 psi. At this time piezometer PZ-3 displayed the only algebraically positive pressure of the entire series of readings. Settlement transducers ST-1 and ST-3 began showing a higher rate of settlement than before the surcharge was applied. However, transducer ST-7 did not accelerate, probably because it was near the wall and not directly under the high portion of the surcharge. During this phase, settlement transducers ST-1 and ST-3 settled 1.1 and 1.3 in. per logarithmic cycle.

During Phase 4, all three settlement transducers continued settling initially, reaching cumulative settlements ranging from 10.9 to 12.5 in. The transducers did not indicate any significant settlements after August 25. On August 25, the contractor installed four risers in the surcharge to confirm that the majority of settlement had ceased. The piezometers showed rates of pore pressure change varying from −0.7 to −1.1 psi per logarithmic cycle. The pore pressure returned to their respective presurcharge pressures on dates ranging from August 20 to September 4. The similarity of these time spans further confirmed that the data were reliable.

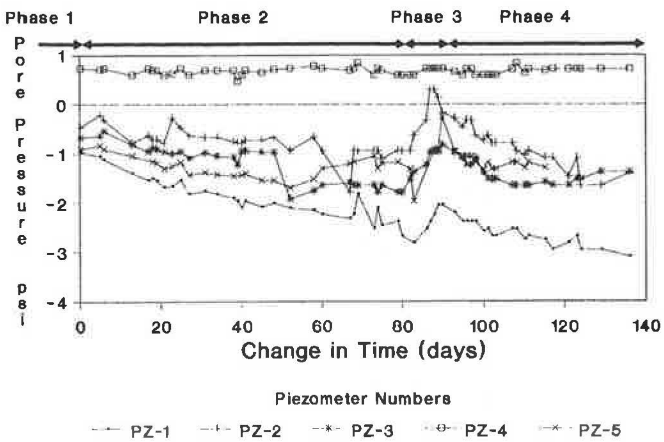

FIGURE 9 Pore pressure versus time.

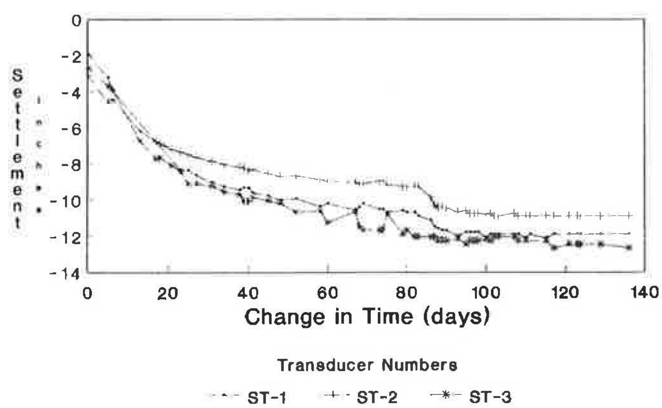

FIGURE 10 Settlement versus time.

TABLE 3 SETTLEMENT UNDER SURCHARGE AREA

DATE INTERVAL (1986)	CUMULATIVE SETTLEMENT (INCHES)			RATE OF SETTLEMENT (INCHES/LOG CYCLE)		
	ST-1	ST-3	ST-7	ST-1	ST-3	ST-7
PHASE 1						
April 25-May 9	1.9	3.0	2.5			
PHASE 2						
May 9-June 13	9.2			7.3		
June 13-July 30	10.7			3.9		
May 9-July 30		9.3	11.9		4.0	6.8
PHASES 3 and 4						
July 30-Aug 25(1)	12.0			1.1		
July 30-Aug 28(1)		10.9			1.3	
July 30-Sep 4(1)			12.5			

(1) - Date of settlement cessation.

TABLE 4 PORE PRESSURES UNDER SURCHARGE AREA

DATE INTERVAL (1986)	INCREMENT OF PRESSURE CHANGE (PSI)				RATE OF PRESSURE CHANGE (PSI/LOG CYCLE)			
	PZ-1	PZ-2	PZ-3	PZ-5	PZ-1	PZ-2	PZ-3	PZ-5
INITIAL READINGS								
April 25	-0.5	-0.1	-0.1	-0.1				
PHASE 1								
April 25-May 9	-0.5	-0.3	-0.5	-0.8				
PHASE 2								
May 9-June 13	-0.8		-0.4		-0.9		-0.5	
June 13-July 30	-1.0		-0.8		-1.9		-1.9	
May 9-July 30		-0.6				-0.5		
May 9-July 7				-0.7				-0.7
July 7-July 30				+0.3				
PHASE 3								
July 30-Aug 7	+0.8	+1.3	+0.9	+1.1				
PHASE 4								
Aug 7-Sep 4(1)	-0.8				-0.7			
Aug 7-Aug 26(1)		-1.3				-1.1		
Aug 7-Aug 26(2)			-0.8				-0.7	
Aug 7-Aug 20(1)				-1.1				-0.9
FINAL READINGS								
Sep 16	-3.0	-1.5	-1.6	-1.3				

(1) - Date that pore pressure declined to pre-surcharge value.
(2) - Date that pore pressure almost declined to pre-surcharge value.

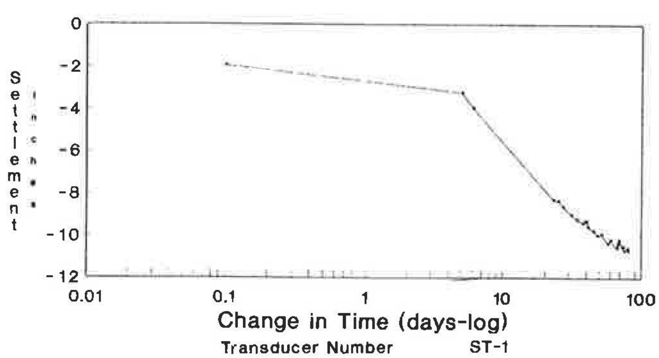

FIGURE 11 Phase 2 settlement (instrument ST-1).

On September 2, 1986, based on the instrumentation data, MARTA concluded that the settlement of the compressible soils under the temporary surcharge was complete, and allowed the contractor to remove the surcharge. By September 15, the southern end of the surcharge had been removed and excavation proceeded northward. Between July 25 and September 8, the entire length of the leveling footing of the Reinforced Earth wall settled about 1 in. (0.08 ft). All 16 of the optical settlement monitoring reference points showed similar values, within the precision of the survey. These measurements compared well with settlement measurements of 1.3 in., 1.6 in., and 0.6 in. from transducers ST-1, ST-3, and ST-7, respectively, during the same time frame.

CONCLUSIONS

The instrumentation provided data that were used in the decisions to stage important construction events. Instrumenting the temporary surcharge in the Chamblee Station enabled MARTA to implement a reliable design for a direct fixation trackway similar to the trackways in most of the other MARTA stations.

The deep dewatering accomplished throughout the platform area increased the magnitude of anticipated settlements of the compressible materials. The deep dewatering also provided a general improvement of the construction conditions. The dewatered soils, when exposed, were less sensitive to disturbance from construction equipment than they would have been if only shallow dewatering had been accomplished. In addition, the compressible soils apparently consolidated more rapidly under embankment and surcharge loads than they would have if they had remained saturated. Both of these factors benefited the contractor by saving time during construction.

The measured settlements were greater than predicted, even when the effect of the deep dewatering was considered. However, the data appeared reasonable, and no instrument malfunctions were determined.

The trends in pore pressure were not surprising, considering the dewatered nature of the soils that existed on the site throughout the time span of the monitoring program. Under these conditions, pore pressures increases were measured when surcharge was placed and the dissipation of the pore pressures with time was observed.

Visual observations of the Reinforced Earth wall have shown no obvious distortion or distress. The Chamblee Station opened to the public in December 1987 and has performed satisfactorily to date.

ACKNOWLEDGMENTS

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