Portland International Jetport Runway 18-36 Extension Project

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ABSTRACT: A Federal Aviation Agency safety area improvement project to extend Runway 18-36 was undertaken at the Portland Jetport in Portland, Maine. The 1,000-ft long runway extension crossed a 25-ft deep gully underlain by up to 30 ft of moderately compressible, low strength marine clay sediments of the Presumpscot Formation. Geotechnical field and laboratory investigations and engineering analyses indicated the silt and clay would consolidate under the weight of the 25 to 30 ft of fill needed to raise the grade to design levels and ground surface settlements up to 25 inches or more were predicted. Ground improvement (i.e., wick drains), a staged-construction sequence, and a temporary earth fill surcharge were used to force the settlement to occur within 12 months. The rate and magnitude of consolidation and settlement were monitored. The paper contains a summary of predicted versus measured ground surface settlement.

1 INTRODUCTION

This paper presents aspects of the geotechnical design, instrumentation monitoring during construction and performance of an earthfill embankment constructed as part of the Runway 18-36, Taxiway C and Perimeter Road Extension (Project) at the Portland International Jetport (Jetport) in Portland, Maine. The Extension Project was part of a larger Federal Aviation Administration (FAA) mandated safety area improvement project that was undertaken at the Jetport between 2009 and 2012. The South Portland and Portland offices of Fay, Spofford & Thorndike (FST; formerly DeLuca-Hoffman Associates, Inc.) and Haley & Aldrich, Inc. (H&A) were retained by the Jetport to provide site civil and geotechnical design and construction services for the Project, respectively.

The Project generally consisted of an extension to the southern end of Runway 18-36 and Taxiway C by approximately 1,000 linear feet (lf) and the Perimeter Road by approximately 3,000 lf. Lengthening the existing alignments required significant grading including filling a 25 to 30-ft deep “gully” underlain by a low strength, moderately compressible marine silt/clay deposit as well as excavation of up to 45 ft of bedrock.

Project development criteria required that site preparation and earthwork activities be completed within approximately 12 months and that post-construction ground surface settlement be limited to no more than 2 inches (in).

2 SITE LOCATION AND DESCRIPTION

As shown in Figure 1 - Site Location, the Project is located in the southeast corner of the Jetport and generally bound by Long Creek and Interstate 295 (I-295) to the south and east. The area was dominated by a low-lying area (“gully”), which generally bisected the site and sloped down in an east-west direction from approximately El. 38 to El. 19. In addition, a sloping hillside was located at the southern end of the Project area with a maximum height of approximately El. 80.

Figure 1 – Site Location

The principal components of the approximate 22-acre Project included a new 1,000-ft long by 180-ft wide extension of Runway 18-36, a 1,550-ft long by 90-ft wide extension of Taxiway C, a new 3,000-ft long perimeter road and a new access road. The proposed grading for the Project ranged from approximately El. 45 (southwestern corner) to El. 47 (match to existing runway and taxiway).

Proposed utilities included a network of 12 to 60-in. diameter storm drain lines used to collect and direct stormwater to an existing man-made stormwater management pond immediately east of the expansion area.

3 SUBSURFACE EXPLORATION PROGRAM AND CONDITIONS

A subsurface exploration program, consisting of a combination of test borings and auger probes were completed within and throughout the Project area in August 2009 as part of the subsurface site characterization effort as shown on Figure 2 – Site Grading and Subsurface Exploration Location Plan. H&A provided full time oversight during the subsurface exploration program and documented the soil, rock and groundwater conditions encountered.

Test borings were drilled to depths ranging from approximately 27 to 56 ft below existing ground surface and were terminated in granular soil present beneath the marine clay or on/in bedrock. In-situ vane shear testing was completed within the marine clay deposit at select test boring locations to measure the undrained shear strength of the soil. In addition, several relatively undisturbed Shelby tube samples of marine clay were collected from various test borings and at various depths for laboratory testing.

The subsurface explorations completed within and adjacent to the "gully" generally encountered the following conditions presented in order of which they were encountered (with increasing depth below existing ground surface):

Soil/Rock Unit	Encountered Thickness (ft)	Consistency	Description
Organic Deposit	2 to 3	very soft to soft	dark brown to black silt with peat and roots
Marine Clay	21 to 42	soft to stiff	gray-brown to gray lean clay with layers and lenses of silt and fine sand.
Marine Sand	2 to 10	loose to medium dense	sandy silt, silty sand and fine sand with variable amounts of clay and coarse to medium sand
Glacial Till	2 to 6	medium dense	well-graded sand with silt and gravel, poorly-graded sand with silt and gravel, to silty sand with gravel.
Bedrock	NA	NA	moderately hard, moderately to slightly weathered, gray, fine-grained schist.

In the upland area (ground surface generally above El. 47) the explorations encountered roughly 1 to 3 ft of soil overlying bedrock.

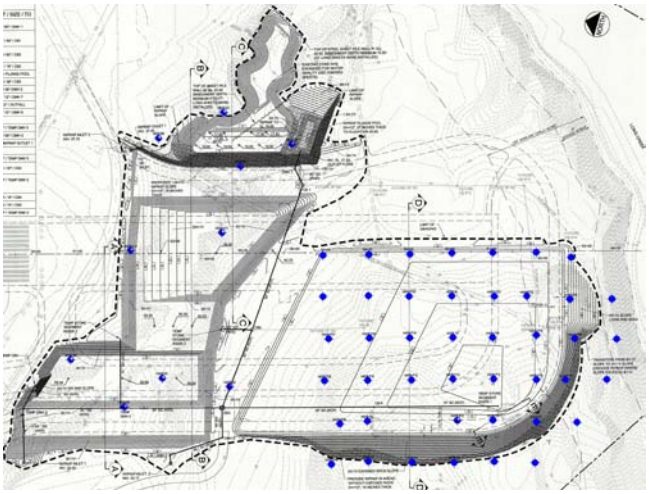

Figure 2 – Site Grading and Subsurface Exploration Location Plan

4 IN-SITU AND LABORATORY TESTING PROGRAMS

As discussed above, in-situ vane shear testing was completed within the marine clay deposit at select test boring locations to estimate the undrained shear strength of the soil. Rectangular vanes, measuring 55 mm x 110 mm or 65 mm x 130 mm were used. Measured peak undrained shear strengths ranged from approximately 450 to 1,200 pounds per square foot (psf) and typically decreased with increasing depth below ground surface unless impacted by the presence of fine sand lenses present in the lower portions of the marine clay deposit.

A laboratory testing program consisting of natural water content (13; ASTM D 2216-05), Atterberg limits (4; ASTM D 4318-05) and one-dimensional consolidation (3; ASTM D 2435) tests were completed on select marine clay samples collected from test borings drilled during execution of the subsurface exploration program. Laboratory testing was completed to aid in soil classification and to determine the stress history and compressibility characteristics of the marine clay deposit present at the site. Results of the laboratory index testing are presented below.

- Liquid Limit – 31% to 41% (avg. = 35%)
- Plastic Limit – 17% to 20% (avg. = 18%)
- Plasticity Index – 14% to 21% (avg. = 17%)
- Water Content – 37% to 47% (avg. = 41%)

Laboratory testing was completed by GeoTesting Express of Acton, Massachusetts.

5 STRESS HISTORY AND COMPRESSIBILITY CHARACTERISTICS OF MARINE CLAY

As discussed above, three relatively undisturbed Shelby tube samples of marine clay were collected during execution of the subsurface exploration program and were submitted for laboratory one-dimensional consolidation testing to determine the stress history and compressibility characteristics. A summary of applicable properties is summarized below.

- Compression Ratio (CR) - 0.24 to 0.25
- Recompression Ratio (RR) - 0.02
- Coefficient of Consolidation (c_v) - 0.08 ft^2/day
- Rate of Secondary Compression (C_α) - 1.0 % strain per log cycle of time

Based on an evaluation of the in-situ and laboratory test data as well as historic marine clay data contained in H&A project files, most of the marine clay was judged to be lightly to moderately overconsolidated. The upper 10 ft of the stratum consisted of a stiff "crust", which was overconsolidated by approximately 3,000 psf, likely due to the result of groundwater fluctuation, weathering and desiccation. H&A judged that the softer clay present at depth was overconsolidated by approximately 700 to 1,000 psf with higher degrees of overconsolidation present at higher elevations, directly below the "crust". The data also suggested that the “gully” was the expression of a post-glacial erosion feature (i.e., the ground surface within the “gully” was higher during a previous geologic time).

6 GEOTECHNICAL EVALUATIONS

As noted, the proposed finish grading within the Project area required significant fills (up to 25 ft) within the low-lying area, which was underlain

by 20 to 40 ft of soft to medium stiff, lightly to moderately overconsolidated marine clay.

Engineering analyses were undertaken to determine how the subsurface conditions within the low-lying area would affect the design and construction of the embankment fill and pavement section for the new runway, taxiway and perimeter road. The principal analyses included 1) determining the rate and magnitude of settlement that could be expected as a result of consolidation of the underlying marine sediments from the weight of the fill placed to raise the grade to design levels, and 2) determining the stability of the embankment slope at the eastern edge of the fill area adjacent to the stormwater detention pond.

Settlement analyses were based on the compressibility and stress history characteristics described in Section 5. The results of the settlement analyses indicated that a permanent grade rise of up to 25 ft of earthfill would cause consolidation settlement of the clay and settlement of the ground surface. The calculations indicated that consolidation-related ground surface settlement could be approximately 20 to 25 in. in areas where more than about 15 ft of fill would be needed. Calculations also indicated that the consolidation process would be relatively slow and could continue to occur for 10 or 20 years or more if ground improvement techniques were not used to accelerate consolidation.

At the eastern edge of the expansion area, adjacent to the stormwater management pond, the required grade rise to design levels ranged from about 20 to 25 ft. A series of evaluations were completed to assess the factor of safety against a global shear failure through the embankment fill and underlying marine soils. The stability analyses, using the undrained shear strengths measured in the field explorations, indicated that the calculated factor of safety against global shear failure at the end of construction would be approximately 1.0, indicating likely significant slope movement and incipient failure.

Due to the relative magnitude and time rate of ground surface settlement and the marginal factors of safety against global embankment shear failure, it was judged that placement of normal weight earthfill to construct the Project would not be feasible unless alternative economically feasible and technically practicable solutions could be developed. Engineering evaluations were undertaken to assess ways to manage the ground surface settlement and slope stability issues associated with the required filling in the low-lying area. Considered alternatives included:

- Support the fill along the eastern edge of the low-lying expansion area on a series of rammed aggregate piers (RAPs) installed through the compressible and low-strength marine clay, to improve slope stability
- Use lightweight fill, including EPS Geofoam (EPS), Tire Derived Aggregate (TDA) and/or lightweight foamed concrete (LWC), for the bulk of the grade rise to reduce consolidation settlement and improve slope stability. The lightweight fill would be covered with 5 to 6 ft of earth fill and pavement section materials for protection and to control elastic movements from aircraft landing, and
- Use ground improvement techniques, staged construction methods and a temporary surcharge to facilitate the filling within the low-lying areas using earth fill and rock fill materials to 1) accelerate the rate of consolidation and ground surface settlement, and 2) improve the stability of the slope at the eastern end of the fill area.

It was determined that all three options summarized above were technically feasible, however the use of RAPs and lightweight fill were not considered to be economically feasible. Preliminary cost evaluations suggested that the premium costs (cost in excess of only placing earth fill to raise the grade to design levels) associated with using RAPs or lightweight fill within the approximate 5-acre low-lying fill area would be on the order of $6 to $10 million and $10 to $11 million, respectively. It was determined that a design using ground improvement techniques, staged construction, and a temporary surcharge would be economically feasible and technically practicable. This option also facilitated the reuse of soil and rock excavated from the hillside in the southwestern corner of the site.

The ground improvement techniques, staged construction, and a temporary surcharge alternative selected for the Project are described in the following section.

7 CONSTRUCTION SEQUENCE

As shown on Figure 3 – Staged Embankment Construction Details, the following general construction sequence was developed:

- Stage 1 - Remove organic soils and topsoil from the ground surface and place a 3 to 4-ft thick sand drainage layer. Install prefabricated vertical (wick) drains in a triangular pattern with a nominal 6-ft center-to-center spacing through the entire layer of marine clay (15 to 40 ft). Install geotechnical instrumentation to monitor ground surface settlement, pore water pressure and lateral earth movements.
- Stage 2 and Stage 3- Place and compact rockfill (processed blast-rock from adjacent hillside) over the entire area in lifts. Raise grade to El. 35 throughout the entire area (Stage 2) and to El. 45 in the central portion of the fill area (Stage 3) and hold for approximately two weeks while monitoring instrumentation and evaluating data. (A buffer area approximately 90 ft wide, extending from the eastern edge of the fill area, at El. 35 was provided to increase the stability of the fill as it interfaces with the stormwater management pond),
- Stage 4 - Raise the grade in the buffer area to El. 42, and continue filling in the central portion of the expansion area to approximately El. 55 in the runway and taxiway areas and El. 50 in the median between the runway and taxiway, or to about 5 to 8 ft above design grades (create surcharge). Monitor instruments to measure time rate and magnitude of ground surface settlement, pore water pressure dissipation, and lateral movement of soil in slope adjacent to stormwater management pond.
- Stage 5 - After raising the grade to El. 55 within the general fill area, then return to the buffer area and raise the grade to El. 45.
- When instrumentation data indicate that consolidation under the design grade loading conditions in the runway and taxiway expansion area is essentially complete (minimum 95 percent of calculated primary consolidation and one cycle of secondary consolidation) the surcharge material can be removed and the runway/taxiway construction can be completed.

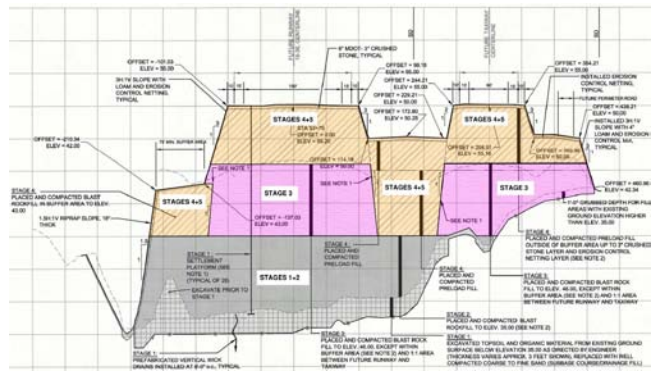

Figure 3 – Staged Embankment Construction Details.

The primary goal of using the wick drains and temporary surcharge was to force settlement equal or exceeding the values calculated during design (recompression, virgin compression and one cycle of secondary compression) during the FAA-mandated time period (12 months for site preparation, fill placement and surcharge period), and limit post-construction ground surface settlement to a maximum of 2 in. The staged construction sequence was also designed to allow for consolidation and associated gains in shear strength to occur within the silt and clay beneath the rock fill slope at the eastern edge of the site, to increase the factor of safety against a global shear failure beneath the slope.

8 GEOTECHNICAL INSTRUMENTATION

Geotechnical instrumentation was installed within the limits of the earth fill surcharge area and was used to monitor the time rate and magnitude of ground surface settlement, pore pressures within the marine silt and clay units, and lateral deformation of soils located beneath the rock fill slope at the easterly limits of the site. Ultimately, the instrumentation was used to determine when

sufficient settlement had occurred and when the surcharge material could be removed. More specifically, geotechnical instrumentation consisted of the following:

- Twenty-six (26) settlement platforms (2 ft by 2 ft plywood base with 2-in diameter steel riser pipe) were installed on a prepared subgrade consisting of medium to coarse sand (wick drain drainage fill) at an approximate 200 ft by 200 ft grid pattern, prior to placement of rock fill. The platforms were used to measure ground surface settlement caused by densification and consolidation of the marine silt, clay and sand soils under the weight of the rock fill and earth fill surcharge materials.
- Seven (7) vibrating wire piezometers were installed at mid-depth of the marine silt and clay layer to monitor changes in pore water pressures during rock fill placement and throughout the surcharge period. The piezometers were used to monitor pore pressures in the saturated marine soils as an indication of the rate and progress of consolidation. The piezometers were installed at settlement platform locations so that we could assess both settlement and pore pressure changes.
- Two (2) inclinometers were installed near the top and near mid-slope of the rock fill slope at the eastern edge of the preload site. The inclinometers were used to monitor lateral deformations of the marine silt and clay soils and the rock fill embankment.

Instrumentation locations are shown on Figure 4 – Instrumentation Location Plan.

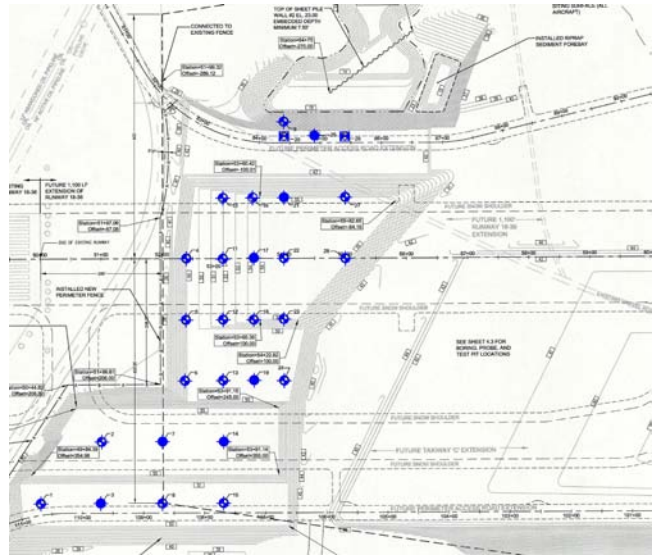

Figure 4 – Instrumentation Location Plan

9 CONSTRUCTION MILESTONES

Construction commenced at the site on 13 September 2010 and earthwork activities continued through the end of December 2010 (3-1/2 months). Principal earthwork activities included:

- Removal and disposal of peat and organic material in low-lying area - 15 September to 12 October,
- Drilling and blasting for bedrock removal - 22 September to 1 November,
- Placement and compaction of wick drain drainage fill -15 September to 12 October,
- Installation of wick drains - 8 to 22 October,
- Placement of rock fill and earth fill surcharge material - 12 October to 25 December.

The surcharge period extended from the end of December 2010 to the end of September 2011 (9 months). The surcharge fill was removed and the sub-base and base course materials were placed during the period from the end of September 2011 to mid-March 2012.

10 EASTERN ROCKFILL SLOPE DEFORMATION

The inclinometers installed in the rock fill embankment slope at the eastern end of the site,

adjacent to the man-made stormwater detention pond were monitored during the period 6 December 2010 through 6 September 2011 and showed maximum lateral movements of approximately 3.1 in. The maximum lateral movements occurred within the stiff marine clay soils near the interface between the clay and the rock fill embankment materials. The rate of lateral movement at the point of maximum deformation was evaluated for each inclinometer.

During initial fill placement operations to raise the grade from about El. 22 to El. 35 (Stage 2) the maximum lateral deformation was measured at approximately 0.3 in, and the embankment slope was considered to be stable. During fill placement generally between El 35 and El 42 (Stage 4) lateral deformation increased significantly. By the end of December 2010 the maximum lateral deformation was about 0.9 in. (see Figure 5). The magnitude and rate of lateral deformation were evaluated and a decision was made to develop measures to control the movement.

It was recommended that modifications at the toe of the slope be undertaken to improve the stability of the rock fill slope. A rockfill buttress measuring approximately 6 ft high and 15 ft wide was placed against the toe of slope. In addition, a planned 6-ft deep excavation in the bottom of the stormwater detention near the toe of slope was deleted for a distance of 30 ft from the toe of slope. The intention of the changes was to provide additional lateral support at the toe of the embankment. At that time it was also decided to limit the top of fill to final design grade (El. 42) and eliminate the planned surcharge to El. 45 with the limits of the perimeter road.

The area adjacent to the top of slope was to be occupied by the perimeter road, so post-construction settlement was judged not to be critical and no surcharge was needed.

During the approximate six-month period after site filling and construction of the toe buttress fill (end of December 2010 to end of June 2011) the rate of lateral movement decreased significantly from the near 1 in/week to approximately 0.06 to 0.02 in/week. For the remaining duration of the monitoring period the rate of lateral movement was about 0.02 in/week. It was concluded that the inclinometer data indicated that the embankment and foundation soils had stabilized.

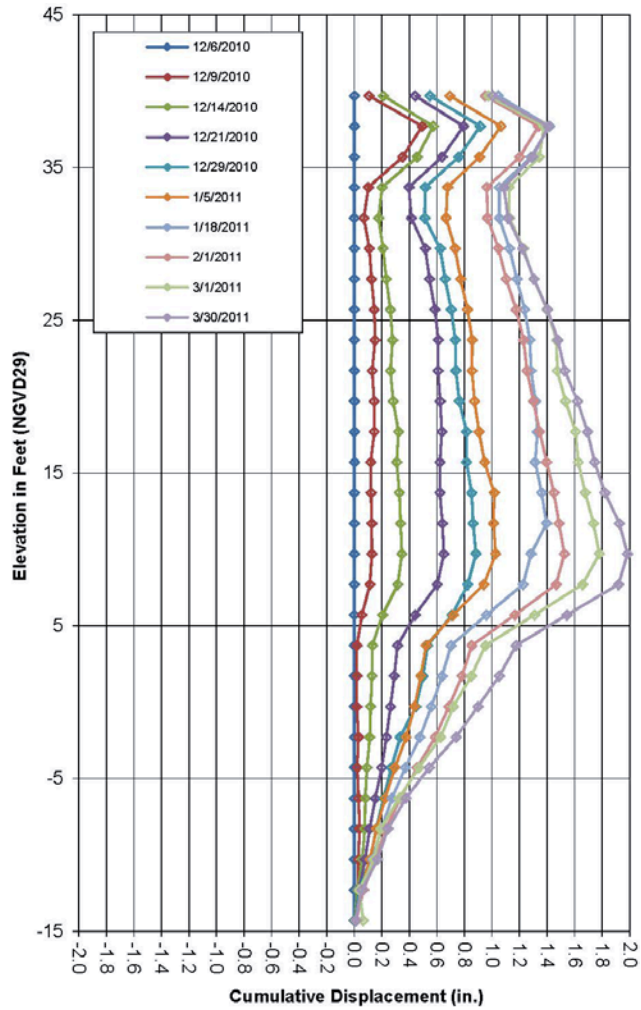

Figure 5 – Top of Slope Inclinometer Data

11 CONSOLIDATION AND GROUND SURFACE SETTLEMENT

The settlement platforms and the piezometers were monitored throughout the rock and surcharge fill placement operations (3 months) and the surcharge period (9 months). Plots of settlement versus log time were maintained for each platform to show the magnitude and time-rate of ground surface settlement. At the seven piezometer locations, the settlement platform data plots were augmented with plots of ground surface elevation showing the progress of the site filling and piezometric head.

Refer to Figures 6 and 7 that show the ground surface settlement, grade (elevation) at platform/piezometer location and piezometric head data plotted versus elapsed time in days (plotted as log time) at platforms P-7 and P-21. The data show clearly the piezometric response to the placement of rock and surcharge fill, and the resulting ground surface settlement. The shape of the settlement versus log time plot is characteristic of the consolidation of clay soils. The piezometric head plot indicates that there was a time lag between the rise in grade and the increase in pore pressure during filling, and the decay of the pore pressure as a result of consolidation. At P-21 there was also a significant drop in piezometric head as a result of the removal of the surcharge material. Similar data plots were observed at the other six piezometer/platform locations. The settlement versus log time plot at the other settlement platform locations are similar to that shown on Figures 4 and 5. The data are considered to be characteristic of consolidation, pore pressure response and ground surface settlement for clay soils under load.

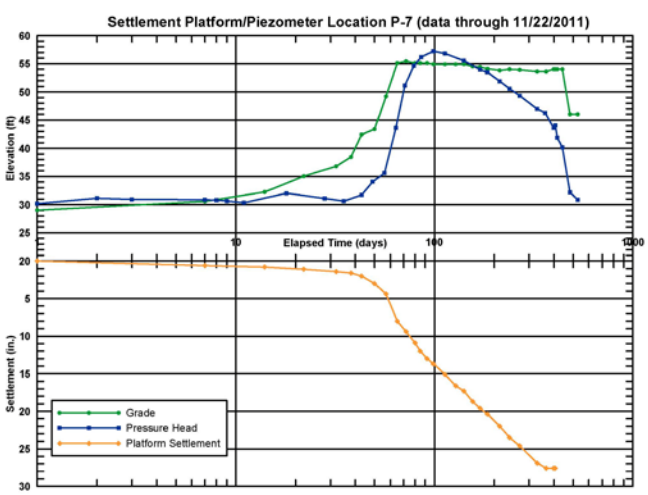

Figure 6 – P-7 Instrumentation Data

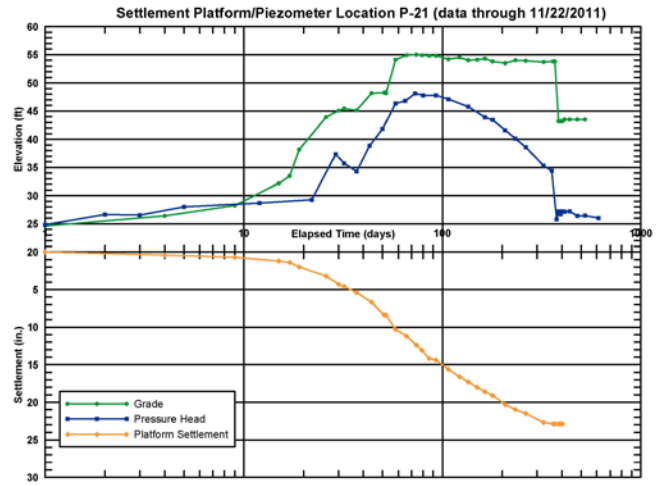

Figure 7 – P-21 Instrumentation Data

A summary of the maximum recorded ground surface settlement measured at the settlement platform locations is provided below. The measured settlements are referenced to the approximate thickness of marine clay as well as the combined thickness of rock and earth fill at each settlement platform location.

Marine Clay Thickness (ft)	Fill Thickness (ft)	Settlement (in.)	
		Range	Average
25	15 to 20	5.8 to 10.9	7.4
	20 to 25	8.8 to 17.8	13.3
	25 to 30	9.6 to 22.4	15.8
30	20 to 25	9.6 to 13	11.6
	25 to 30	18.7 to 22.6	20.6
35	25 to 30	20.6 to 26.9	22.8

Settlement estimates representing maximum conditions (clay thickness and fill height) made during the design studies were compared with the settlement measured at settlement platform SP-7. The design-phase settlement estimate was made for the following conditions and soil properties:

- Clay thickness - 30 ft.
- Fill height(permanent grade rise plus surcharge, and fill to compensate for settlement) - 30 ft.
- Average fill unit weight - 130 pcf
- Recompression Ratio (RR) - 0.02
- Compression Ratio (CR) - 0.24
- Top 10 ft of clay preconsolidated by 3,000 psf
- Middle 10 ft of clay preconsolidated by 1,000 psf

- Bottom 10 ft of clay preconsolidated by 700 psf

The estimated settlement was 29.5 in. including about 3.7 in of recompression and 25.8 in of virgin compression. Secondary compression was estimated to be approximately 3.6 in. per log cycle of time. The settlement plots at SP-7 (Figure 6) were evaluated and suggest recompression settlement of about 2 in, virgin compression of about 25 in, and secondary compression of about 1 in. Based on this, the estimated and measured settlement demonstrates that the soil properties used in design were well matched with the actual conditions present at the site.

The settlement plots clearly demonstrate that primary consolidation under the surcharge load was essentially complete, secondary compression was occurring and that the requirement for a maximum of 2 in. of post-construction settlement under the final design loads would be met.

12 CLOSURE

This paper has presented the results of the design, construction and monitoring of an extension of Runway 18-36, Taxiway C, and Perimeter Road at the Portland Jetport in Portland, Maine. The extension project included crossing a post-glacial erosion gully that required raising the grade within the gully by as much as 25 ft to reach design grades. The gully contained 2 to 3 ft of peat and organic soil overlying up to about 35 ft of moderately compressible and low-strength marine clay sediments of the Presumpscot Formation.

Engineering evaluations indicated that the silt and clay soils would consolidate under the load of the added fill to reach design grades and that ground surface settlements ranging up to about 25 to 30 in would occur. A staged-construction sequence using ground improvement techniques (prefabricated vertical - wick drains), re-use of blast rock fill from on-site excavations and a temporary earth fill surcharge was developed. The sequence was designed to complete the earthwork and surcharge within a 12 month period and to control post-construction ground surface settlement to a maximum of 2 in.

Geotechnical instrumentation was installed to monitor the rate and magnitude of ground surface settlement, pore pressure dissipation in the consolidating silt and clay, and lateral deformations of silt and clay foundation soils and rock fill embankment material adjacent to a man-made stormwater detention pond.

The measured settlement was compared with predicted settlement. There is a good correlation between expected and measured settlement, suggesting that the consolidation properties and stress history characteristics of the Presumpscot Formation silt and clay soils at the site were well-predicted using field and laboratory test results and published properties from previous investigations and construction projects.

The site preparation activities and temporary earth fill surcharge were accomplished within the required 12-month, and the extensions to the runway, taxiway and perimeter road were completed and placed in service.