15 Apr 2004, 7:00pm - 8:30pm

Erskine Street Interchange

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ABSTRACT

Erskine Street Interchange is a new roadway interchange constructed in Brooklyn, New York in 2002. Challenges addressed in construction of the interchange included poor foundation conditions, such as the presence of hydraulic sand fill, a weak cohesive deposit, an approximately 30-year old municipal waste landfill beneath portions of the abutment ramps, and the presence of one of the most important commuter roadways in Brooklyn and Queens, the Belt Parkway, immediately adjacent to the construction site. How these challenges are addressed in the final design and construction of the Erskine Street Interchange will be discussed. The design consists of on grade ramps approaching a pile supported abutment. A ground improvement program, including deep dynamic compact, soil surcharge and use of geogrids was implemented. The quality control measures used to verify soil performance and protect the adjacent parkway, and long term monitoring and performance of the embankments will also be addressed.

INTRODUCTION

This paper presents a case history of the design and construction of the Erskine Street Interchange in the East New York Section of Brooklyn, New York. The location of the interchange is shown in Fig. 1.

The interchange was built to provide a bridge overpass as well as entrance and exit ramps from the east and west bound Belt Parkway lanes to the southern edge of Erskine Street. The interchange access was constructed as part of a larger retail development located on the north side of the Belt Parkway.

The Belt Parkway is the primary transportation route through the southern portion of Brooklyn and Queens, and is a major route for Long Island access to Brooklyn and Manhattan. Due to the high volume of traffic along this parkway, the road was to remain open during the entire construction period.

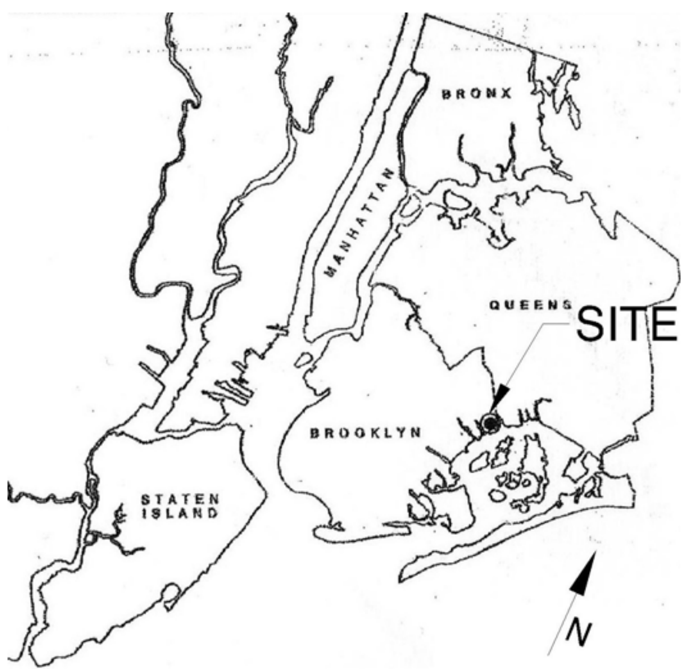

Fig. 1. Site Location Map

The interchange bridge was designed as a single span construction with an approximately 120 ft clear distance across the Belt Parkway between the north and south abutments. Four-entry/exit ramps extending about 400 ft to the east and west of the centerline of the bridge provide access from both the west and east bound traffic. A plan of the interchange is shown in Fig. 2.

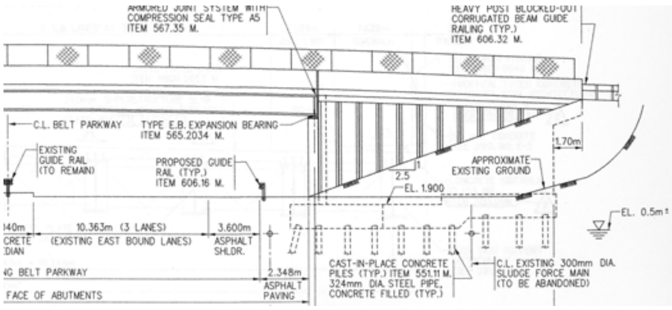

Fig. 2 Interchange Sketch (by Hardesty and Hanover)

The primary purpose of the interchange is to provide vehicle access to the new retail complex so that the additional traffic generated by the commercial center would not congest local streets. The site selected for the interchange was bound to the north by vacant land that consisted of filled in tidal marsh lands, to the south by the Fountain Avenue landfill (FAL) a Class 2a hazardous waste site, to the west by Hendrix Creek and to the east by the New York State Developmental Center and Spring Creek. About a 300 ft footprint in the east - west direction and about half the ramp area in the north-south direction of the south abutment ramps overlapped with the footprint of a "windberm" which formed the northern edge of the FAL and the landfill itself.

The following sections present a summary of the site geologic and construction history prior to construction of the interchange, a description of the subsurface investigation implemented for the interchange design, and a summary of the design and construction of the ground improvement program utilized for the south ramps.

This paper focuses on the design and implementation of a ground improvement program that was used to minimize long term total and differential settlements of the south ramps.

SITE HISTORY

According to geologic maps, the Erskine Street Interchange site was originally a tidal wetland abutting Jamaica Bay in the Coastal Plain region of New York City. The tidal wetlands were formed on top of glacial outwash sands, which overlie cretaceous rocks of the Magothy and Raritan formations. Bedrock in this area is several hundreds of feet deep. Numerous streams and channels historically crossed the site however these creeks have since been filled in.

Filling of the tidal wetlands occurred in several stages beginning in the 1930's with construction of the Shore Parkway (now known as the Belt Parkway). The area surrounding the parkway was used a disposal area during the 1940's and 1950's. Three major land filling events occurred at the site in the 1960's, during which large amounts of hydraulic fill dredged from Jamaica Bay were placed. Beginning in 1960, fill was placed from the parkway towards the south to create the eastern portion of the FAL. In 1965 hydraulic fill was placed to create the western portion of the FAL. The third filling of the site occurred in 1968 when hydraulic fill was placed over portions of the site to provide additional access to the Parkway and for construction of the New York State Developmental Center.

The FAL was active from the early 1960's, until the mid 1980's. In 1985 the FAL stopped receiving waste. At this time, the landfill was declared a Class 2a inactive hazardous waste site by the New York State Department of Environmental Conservation. Final closure/capping of the landfill began in 2002 following construction of the interchange.

PROJECT HISTORY

Geotechnical investigations had previously been performed at the site for Spring Creek Developments. The investigations consisted of a limited number of borings performed by Ramon Associates in March 1988, by Independent Testing in 1996, and by Tams Consultants, Inc. between 1993 and 1996.

In 1998, Related Retail together with Black Acre Development Companies acquired the right to the property and began to plan the development of a new commercial shopping center and interchange. Langan Engineering served as Geotechnical consultant to the developers for both the shopping center and the interchange.

The alignment and elevations of the Erskine Street Interchange was designed by Philip Habib & Associates (transportation engineers) and Hardesty and Hanover, LLP (bridge engineers).

The interchange project is somewhat unique since initial construction was by a private developer, with ownership and long term maintenance responsibilities assumed by the New York City Department of Transportation (NYCDOT) and the New York State Department of Transportation (NYSDOT) following the construction period. In addition, the Department of Environment Protection and the Parks Department controlled the adjacent landfill. Agency approval was required due to the overlap of the landfill with the interchange right of way.

SUBSURFACE INVESTIGATION

Previous investigations provided limited subsurface information. The Langan investigation was designed to supplement the existing borings and to obtain information required to address each of the identified challenges. Langan's subsurface investigation for the Interchange occurred between 26 August and 6 October 1999. The investigation consisted of twenty-one (21) geotechnical borings of which 11 borings were for the south ramps. The borings were advanced to depths ranging from 16 to 82 ft below the existing grade which ranged in elevation from 6 to about 8. The results of the test borings indicated that the interchange site generally consisted of hydraulic sand fill to about el. -4, underlain by tidal marsh deposits overlying sand at about el. -8. All of the borings terminated within the lower sand layer. Between 10 and 25 feet of refuse fill was encountered in borings drilled south of the windberm, along the south abutment ramps. A generic soil profile is shown in Fig. 3. Groundwater measurements were made in all borings and in wells installed within portions of the site that were currently wetlands (north of the Parkway). Groundwater readings varied from about el 5 to about el -1. The higher water levels were typically measured in wetland areas.

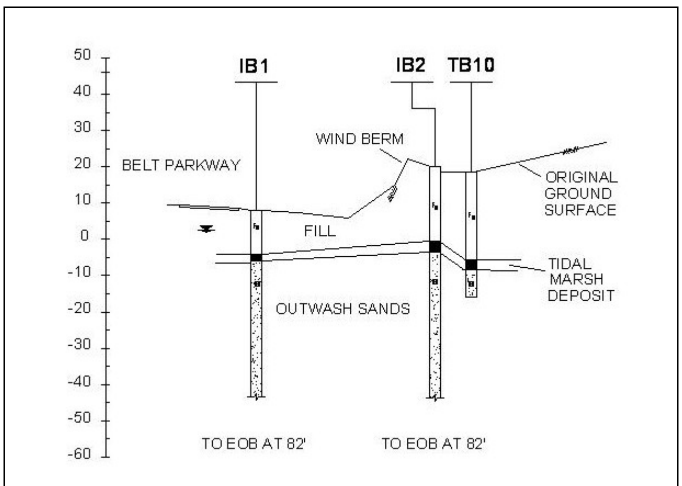

Fig. 3 Subsurface Profile along the South Abutment

Five test pits were excavated on the south abutment in December 1999 along the windberm area to further characterize the type of refuse that was deposited in this area. Three test pits were excavated along the crest of the windberm, one test pit was excavated to the north of the windberm (inside the landfill limits), and one test pit was excavated about 15 ft south of the windberm.

The test pits confirmed that large quantities of household refuse/landfill deposits existed south of the windberm. The material encountered in the test pit south of the windberm consisted of a heterogeneous mixture of refuse, construction debris and soil. The debris encountered included masonry, glass, fabric, metal, timber and tires. The percentage of refuse within the test pit generally ranged from 50 to 70% by volume, however zones of 100% non-soil materials were encountered.

Non-soil, debris material was also encountered in the four test pits excavated near the crest of the windberm. The material excavated from these test pits generally consisted of a mixture of medium dense soil and general construction–type debris, including paper, timber, and metal. The percentage of debris within the windberm test pits ranged from about 10 to 40% by volume.

INTERCHANGE DESIGN

Following the subsurface investigation, Langan began to evaluate the design alternatives for support of the abutment and approach ramps. The three primary geotechnical design issues to be addressed for the interchange were:

1) Potential short and long term settlement of the tidal marsh deposits and of the land fill deposits under the approach ramps
2) Slope Stability of the approach ramp embankments during and following construction of the interchange; and,
3) Seismic response and liquefaction potential of the underlying soils.

A fourth issue which needed to be addressed prior to construction was the potential impact of construction to workers, the public and the surroundings during construction in the landfill area.

Additional design and construction challenges that needed to be addressed in the design phase included the proximity of the Belt Parkway, which needed to remain open and undisturbed during all phases of construction, and the restricted access to the portion of the site that was within the landfill. All earthwork within the landfill area needed to be completed during the winter months.

It was decided early in the process that the abutments would be pile supported due to:

1) limited tolerance regarding differential settlement across the ramp;
2) the proximity of the bridge to the Belt Parkway.

The abutments were to be constructed immediately adjacent (within 5-ft) to the existing Belt Parkway roadway. The parkway was required to remain open during bridge construction (with the exception of a few hours of closure allowed at night during placement of the overpass steel).

A pile supported abutment was determined to be the best way to minimize the potential impact of construction on the parkway, and to reduce the effects of settlement of the tidal marsh deposit.

Three alternatives were evaluated for the support of the approach ramps:

1) a deep foundation system;
2) removal and replacement of undesirable bearing soils;
3) support on grade with a ground improvement program.

The potential presence of Class 2a hazardous wastes, and large obstructions behind the windberm made driving piles for the south abutment approach ramps undesirable. Removal of the landfill deposits in the vicinity of the ramps was also analyzed, however this posed concerns regarding worker exposure and cost of disposal.

The third alternative considered a technique of dynamic compaction in conjunction with surcharging. This method had been used successfully by Lewis and Langer for an interchange built on a similar aged/composition landfill in New Jersey. (Lewis and Langer, 1994).

The primary concern regarding the third option was potential for large differential movements within the landfill deposits, and between the pile supported portion of the bridge and the abutment ramps. The ground improvement program was designed to generally reduce the amount of overall settlement, and specifically the differential settlement, to a level acceptable to both the developer and the public authorities. After lengthy discussions with the NYCDOT and NYSDOT, it was agreed to support the approach ramps on improved ground. Part of the agreement included long-term monitoring for a period of three years following paving.

The surcharge was intended to decrease long term compression and consolidation of the organic soils. The deep dynamic compaction program was designed to consolidate or compress the existing material, and eliminate any large voids in the refuse prior to placement of the surcharge.

The design supplemented the ground improvement program with a five foot zone of reinforced soil immediately beneath the pavement sub-base coarse. The reinforced soil consisted of two layers of a biaxial geogrid with two to three feet of compacted sand between the layers of geogrid. The geogrid/soil combination was used to create a soil matrix that could span isolated pockets of differential soil movement.

Tolerance for differential settlement between the pile supported abutments and the on grade ramps was built into the design through the use of a transition slab. The transition slab had hinges at the interface of the bridge where the abutments meet the ramps. The hinged slab was designed to accommodate several inches of differential settlement.

Settlement Analysis

The potential short and long term settlements of the tidal marsh deposits below the approach ramps was evaluated using laboratory data from "undisturbed" Shelby tube samples obtained at the site, and from settlement data from past experience in the vicinity. Laboratory analysis indicated that the tidal marsh deposits beneath the interchange were generally over consolidated, with an over consolidation ratio ranging from about 1.8 to 2. The compression ratio (CR) and secondary compression ratio of the tidal marsh deposit was increased by a factor of 2 to estimate a "range" of anticipated settlements.

Due to the variability in municipal solid waste landfill deposits, a range of compression ratios (CR) was also used; the lower values for the analysis were considered a "best case" value. The best case value was determined from the actual performance of similar aged/composition landfills, reported by literature (such as Zamiski et al, 1994) and from Langan's experience with landfills in the tri-state area. The parameters were doubled to produce a "worst case" estimate. Settlement parameters used for the TMD and refuse deposits are summarized in Table 1.

Table 1. Settlement parameters (Design)

Layer	CR	Ca	CRr	Car
Refuse (best case)	0.05	0.005	0.005	0.0005
Refuse (worst case)	0.10	0.010	0.010	0.0010
Organic (best case)	0.18	0.009	0.018	0.0009
Organic (worst case)	0.36	0.018	0.036	0.0018

The estimated settlements of the tidal marsh deposits and refuse under the weight of the embankment ranged from about 6 to 8 inches, including up to 4 inches of long-term (secondary) consolidation.

The short term settlements of the TMD were estimated to occur within 5 months of the embankment construction. A parametric study was performed to analyze the effect of different surcharge heights on the potential reduction of long term settlements which could result in up to 4 inches of differential settlement between the abutment and the ramps. Field data supporting the use of a surcharge to reduce secondary consolidation in peat deposits has been reported based on experience in Florida's wetlands (Frizzi and Yu, 1994).

Surcharges of 5, 7 and 10 ft of sand were evaluated. Long-term settlement under a 5 ft preload was estimated at about 1.5 inches, dropping to below one inch under a 7 ft preload.

Langan's recommendation for minimization of differential settlement of the embankments for the on-grade ramp support consisted of the use of a 7 ft surcharge, and the implementation of an acceptable performance curve for the long term settlement to satisfy NYCDOT and NYSDOT maintenance criteria A cross section of the design surcharge is shown on Fig.4.

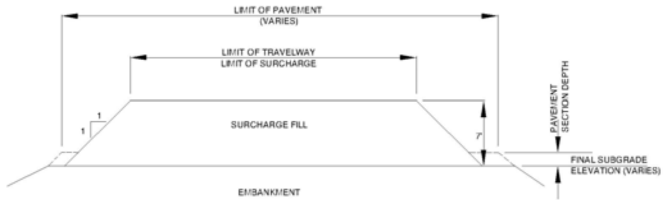

Fig. 4. Surcharge Design for South Approach Ramps

Stability of the Embankments

Stability of the embankment slopes was evaluated using the modified Bishop method for circular failures, and a block sliding analysis. Two cases were analyzed, long term drained and short term undrained analysis. For the long term condition a live load of 480 psf was used to represent an HS15 truck. For short term stability a 7 ft surcharge was included to account for the surcharge period. No other live load was considered for the short term condition. A pseudostatic slope stability analysis was used for both short and the long term to evaluate the dynamic condition. An average shear strength for the cohesive soil of about 600 psf was used based on laboratory tests and correlations.

Long term sliding wedge type failures were noted to be more critical than circular failures, with factor of safety for static condition of about 1.5 for the sliding wedge analysis. The short term factor of safety was above 1.2 when allowing for strength increase of TMD. For the dynamic condition of calculated factors of safety were 1.2 and 1 respectively, for ground accelerations of 0.1 g and 0.2 g.

The strength of the organic tidal marsh deposits was determined to be the controlling factor in the stability analysis. The analysis indicated that under the existing (preconstruction) condition the tidal marsh deposits might not be strong enough to support the additional load of the new embankment. Staged construction would allow pore water pressures to begin to dissipate and subsequent strength increase in the cohesive layer. Monitoring of the tidal marsh deposit would be performed during the construction using inclinometers and piezometers so that rate of fill placement could be increased/decreased as necessary.

Seismic Analysis

The Erskine Street Interchange was considered a critical structure by the NYCDOT (Weidlinger, 1988). Critical structures are required to remain operational during a lower level (500-year or 10% probability of exceedence in 50 yrs) earthquake, and are to remain standing during a higher level (2500 year or 2% probability of exceedence in 50 years) earthquake. In addition to not collapsing during the higher level event, critical structures are required to be available for limited access by emergency vehicles within 48 hours after the earthquake, and are to accessible to the public within a few months.

Design earthquake motions were developed by Weidlinger Associates as part of a New York City Seismic Hazard study for the New York City DOT (Weidlinger, 1988). Three synthetic rock motions were developed for the upper and the lower level earthquakes. The rock motions generated during the Weidlinger study were used for the design of the Erskine Street Interchange.

The Shake91 code (Idriss and Sun, 1991) was used to model soil response to the design rock motions, and to develop design response spectra for the bridge engineer. The Shake program was also used to evaluate the predicted strains in the soils for performance of liquefaction analyses.

Two zones of potentially liquefiable soils were identified based on the analyses. The abutment piles were designed assuming no support in these liquefiable zones. The potential effects of the liquefied soil was also modeled in the stability analyses of the approach road embankments.

Deep Dynamic Compaction

The deep dynamic compaction program at Esrkine Street was designed using guidelines in the Federal Highway Association Geotechnical Engineering Circular No. 1 – Dynamic Compaction (USDOT, 1995), and from experience on past dynamic compaction projects.

Dynamic compaction is a ground improvement technique which results in the application of a large amount of dynamic energy to densify soil deposits at depth. The dynamic compaction process relies on the use of a heavy weight, typically varying from a few tons to as many as twenty tons, dropped from heights, ranging generally from 20 to 70 ft. The weight is dropped along a specified grid, average of about 10 ft. The dynamic compaction process is often designed for 2 passes along a grid, with the grid staggered/offset between passes to insure overlapping coverage. The weight is dropped several times at each location of the grid depending on equipment capabilities and crater size. Craters are backfilled in between passes.

The selected program consisted of a two pass design approach on a 5-ft offset grid. During the primary pass, a lower energy is applied to densify the upper layers. During the secondary pass, a higher energy is applied to the subgrade; the second pass energy typically transmits more efficiently through the densified upper layers and tends to densify the deeper portion of the deposit. In addition, the area of influence of the applied compaction effort increases due to the more efficient energy transfer

The project specifications for the deep dynamic compaction required a primary pass with a 5 ton pounder dropped from a height of 30 ft, and a secondary pass with a 10 ton pounder dropped from a height of 40 ft. . (see Fig. 5).

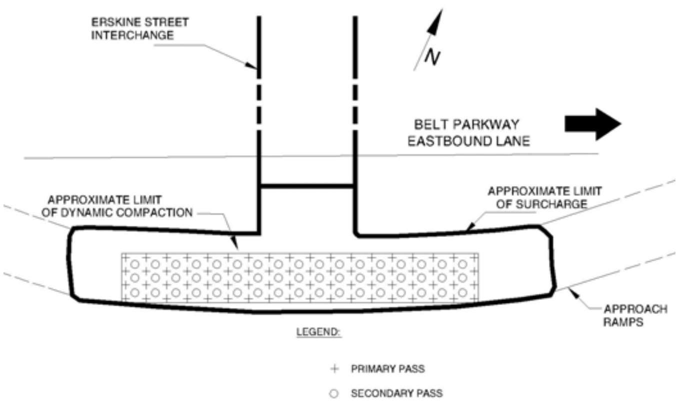

Fig. 5. Deep Dynamic Compaction Plan

<u>Instrumentation Program</u>

The instrumentation program consisted of 2 pneumatic piezometers, 8 observational settlement plates, 4 vibratory wire settlement plates, and 2 inclinometers. The purpose of the instrumentation was used to monitor the performance and behavior of the underlying soils during construction and to modify the design and construction schedule as necessary. The location of instrumentation is shown on Fig. 6. The vibratory wire settlement plates were intended to remain in place to monitor settlements following final paving.

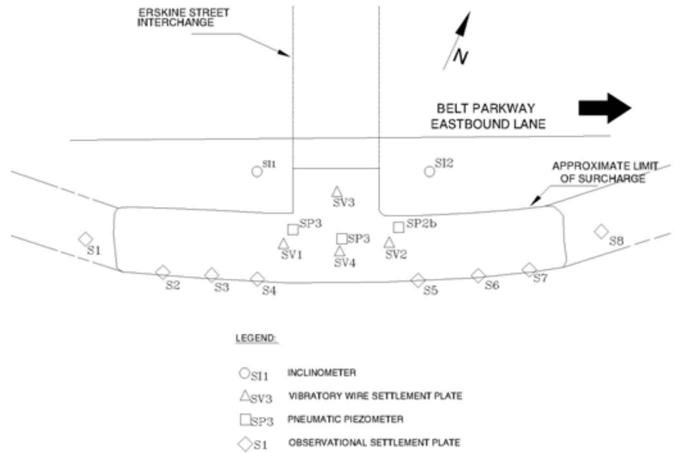

Fig. 6. Instrument location plan –south abutment

CONSTRUCTION

Construction of the interchange began with driving of a portion of the abutment piles in December 2002. Installation of monitoring instrumentation followed in January and February 2002.

Preparation for the deep dynamic program included removal of brush and leveling of the windberm to form a working platform for the deep dynamic compaction equipment. The working platform consisted of a 2 ft thick layer of sand compacted with a 10 ton vibratory roller. The working platform was used to cover any loose superficial materials to minimize foreign materials “flying away” onto the – Parkway during the compaction. The leveling platform also served as a denser top layer to efficiently transfer and distribute the compaction energy to lower soil/debris deposits. Additional protective measures included a tall fence parallel to the parkway and a sand filled trench was installed between the dynamic compaction platform and the Belt Parkway to minimize impact to the roadway and passing vehicles. The fence was installed north of the dynamic compaction area to capture any material that broke off during compaction, potentially hitting vehicles on the parkway. The trench was excavated north of the fence and backfilled with loose sand. The purpose of the trench was to dampen any vibrations created during the compaction activities.

The contractor provided an 11 ton pounder. The drop height for the primary pass was reduced to 20 ft to compensate for the different size pounder weight and to approximately achieve the specified energy input, the pounder was dropped from a height of 40 ft for the secondary pass. Crater depths from the first and second pass ranged from 2 to 3 ft deep. Piezometers installed prior to the start of deep dynamic compaction were damaged during backfilling of the craters. The vibrating wire settlement plates and observation settlement plates were installed after the deep dynamic compaction was completed.

Surcharge Construction

The surcharge program was designed using information from the settlement and stability analyses. For the south abutment, the design surcharge consisted of 7 ft of sand fill, placed within the footprint of the proposal roadway. The surcharge was designed to remain in place for a minimum of 6 months, during which surcharge performance was monitored through an instrumentation program.

Placement of compacted soil for the south abutment and ramps began at the end of March 2001. An approximately 7 ft high surcharge was placed between 18 and 23 April 2001.

Monitoring Results

The inclinometers indicated some movement in the tidal marsh deposits. The movement was typically measured when the contractor’s rate of fill placement exceeded 5 ft per week, confirming the design recommendation regarding staged construction.

The pneumatic piezometers were initially installed prior to dynamic compaction operations; however the piezometers were damaged during Department of Design and Construction operations. Replacement piezometers were installed during the first week of fill placement, and therefore were not able to stabilize prior to measurement of pore pressure buildup in the tidal marsh deposit. The maximum pore pressure measurement was recorded at the completion of placement of the surcharge. Figure ## shows the recorded pore pressures during construction.

The surcharge remained in place for a total of 6 months. The observation settlement plate recordings during these 6 months are shown in Fig. 7. The total recorded movement in each plate is listed in Table 2.

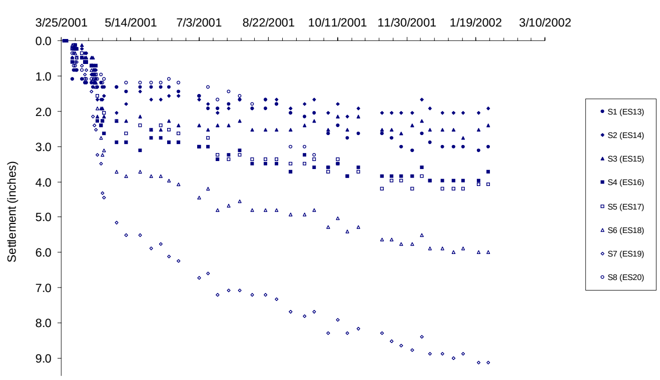

Fig. 7. Observational Settlement Plate Radings

Surcharge Performance

Observational Plate	**Settlement**	**Recorded Settlement (inches)**
S1		**3.0**
S2		**2.0**
S3		**2.5**
S4		**4.0**
S5		**4.2**
S6		**6.0**
S7		**9.0**
S8		**3.5**

Settlement Plate S1 was located outside of the landfill/windberm area, in a location where total fill increase due to embankment construction was less than average. Settlement here is mainly due to consolidation of the Tidal Marsh Deposits.

Observational settlement plates S2 through S6 were located south of the windberm in an area that was dynamically compacted. The settlement recorded with observational settlement plates S2 and S3 was similar to the rate and amount estimated for the tidal marsh deposits. Sudden increases in settlement are from compression of the underlying refuse. Settlement plates S4, S5, S6 and S8 were all located north of the windberm in areas in which the largest amount of fill was placed. The steepest gradient in settlement on all plates was recorded within the first month of the consolidation placement. The occurrence of the “jump” in settlement at the same time on all plates leads to suspicion that this “observed” movement may have been due to a disturbance of the surveyors reference points

The largest amount of settlement was recorded at plate S7. This observational settlement plate was located north of the windberm and therefore out of the deep dynamic compaction area. Review of the ground surface contour map of conditions prior to embankment construction indicates that this area may have contained some refuse material (i.e., higher elevation than surrounding non landfill areas). The greater amount of settlement is partly due to compression of the refuse that was not dynamically compacted.

Long-Term Monitoring

Settlement of the tidal marsh deposits and the FAL beneath the new ramps area being monitored post construction through the use of vibratory wire settlement plates and p-k nails. The Vibratory wire settlement plates are installed beneath the embankment fill, and are connected to a reference reservoir and datalogger that is stored adjacent to the approach ramps.

The P-K mails are essentially pins embedded into the road asphalt that are surveyed periodically through optical methods.

At the completion of about 1 year of post construction settlement monitoring recorded settlement of the underlying soils is less than 1/16 inch. This is well within the range of the acceptable performance criteria agreed to by the authorities. It is also much less than the estimated secondary settlement that would have occurred if the ground improvement program had not been implemented.

CONCLUSIONS

Construction and performance of the Erskine Street Interchange has confirmed the use of Deep Dynamic Compaction in conjunction with surcharging as a method of improving poor subgrade. The Erskine Street Interchange was a success due to the cooperation of the developer, the public agencies, and the design team in evaluation and selecting a cost effective innovative solution. Advanced ground improvement techniques used in cooperation with a geotechnical instrumentation program allowed for the successful/cost effective construction of large embankments over relatively poor soils adjacent to a critical roadway.

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