Numerical settlement analyses and design of bridge pier underpinning due to tunnelling

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ABSTRACT
As part of the East Side Access (ESA) Tunnel Project in New York City, four (4) TBM tunnels will be bored on alignments that pass below Honeywell Street Bridge Piers 3, 4, 5, and 6. These piers are located on the LIRR/AMTRAK Main Line railroad embankment adjacent to Sunnyside Yard, Queens, New York. The tunnel boring operations are expected to cause settlement of the ground below piers 4, 5, and 6. Ground settlement below Pier 3 is expected to be negligible. This paper presents a case study of the settlement analyses of the piers and the proposed solutions for underpinning them. For the settlement analyses, the geotechnical computer program PLAXIS was used to create a finite element model of the embankment, an underlying layer of glacial till, the bridge piers, and the tunnels. The settlement of the piers was evaluated for three construction conditions: 1. good construction practice, 2. normal construction practice, and 3. poor construction practice. The maximum settlement at each end of each pier was determined as was the differential settlement between adjacent piers. The calculated settlements were found to be less than the tolerable settlements for highway bridge substructures as specified in NCHRP Report 343. However, the welded steel superstructure frames of the bridge are relatively unique and it was decided that Pier 6 should be underpinned and that temporary steel frames be used to support the superstructure at Piers 4 and 5.

RÉSUMÉ
En tant qu'élément du projet de tunnel East Side Access (ESA) à New York City, quatre (4) tunnels seront forés par des tunneliers sur les alignements qui passent au-dessous des piliers 3, 4, 5 et 6 du pont situé à la rue Honeywell. Ces piliers sont situés sur le remblai des lignes principales des voies ferroviaires de LIRR/AMTRAK adjacentes à Sunnyside Yard, situé à Queens, New York. On s'attend à ce que les opérations de forage des tunnels engendrent des tassements des piliers 4, 5, et 6. Selon nos prévisions, le tassement du pilier 3 sera négligeable. Cet article présente les résultats d'étude de cas des tassements pour les piliers du pont et des solutions proposées pour les soutenir. Pour les calcules de tassement, nous avons crée un model d'éléments finis composé du remblai, des couches géologiques du terrain (couche glaciaire) les piliers du pont, et les tunnels en utilisant le logiciel PLAXIS spécialisé dans la modélisation des problèmes géotechniques. Le tassement a été évalué pour trois types de forage : bonne, moyenne et mauvaise pratiques en matière de construction. Le tassement maximum de chaque pilier a été déterminé ainsi que le tassement différentiel entre les piliers adjacents. Les tassements calculés sont inférieurs aux tassements limites tolérés pour des ponts spécifiés dans le rapport 343 de NCHRP. Due à la spécificité des armatures en acier soudées de la superstructure du pont, il a été décidé que le pilier 6 devrait être soutenu. Pour soutenir les piliers 4 et 5 il a été décidé d'employer des armatures provisoires en acier.

1 INTRODUCTION

The East Side Access (ESA) Project is a large transportation project currently being undertaken by the Metropolitan Transit Authority (MTA) in New York City. The project will provide Long Island Railroad (LIRR) commuters with direct access to Manhattan's East Side via a new rail line between the Sunnyside Yard in Queens and Grand Central Terminal in Manhattan. The project will utilize the lower level of an existing two-level tunnel under the East River and will construct connecting tunnels in Manhattan and Queens. The Manhattan tunnels are currently under construction. The Queens tunnels are expected to start in 2009 and will consist of four (4) tunnels constructed using pressurized face tunnel boring machines (TBMs). The ESA Project is being managed by the MTA Capital Construction Company.

The internal diameter of the Queens tunnels is 5.94m and they will be lined with 1.53 m wide, 305 mm thick precast concrete segmental lining. Although the tunnels are relatively short, varying in length from 530 m to 1340 m, they will be constructed through an area of densely developed railroad infrastructure, shown in Figure 1. They will pass close to utilities, bridge footings, and large boulders randomly distributed in the overlying glacial till/outwash deposits. The four TBM tunnels will be bored on alignments that pass below Honeywell Street Bridge Piers 3, 4, 5, and 6 in the railroad embankment adjacent to Sunnyside Yard. The bridge carries highway traffic over the mainline tracks (AMTRAK and LIRR) that pass through the yard interlocking. The tracks are on an embankment that is typically 6 m higher than adjacent yard elevations. At Honeywell Street Bridge, the tunnels will be closely spaced and have relatively shallow cover beneath the tracks. The tunnel boring operations are expected to cause settlement of the ground below Piers 4, 5, and 6.

In order to prevent overstress and/or damage to the steel superstructure of the bridge, Pier 6 will be underpinned with jet grout columns. Piers 4 and 5 will be allowed to settle with the ground. The superstructure above will be supported on temporary steel frames constructed on top of the piers. The elevations of the superstructure girders and bridge deck will be maintained by a jacking system at the top of the frames. The base plates for the existing steel columns that support the superstructure girders will be replaced when the settlements are considered stabilized.

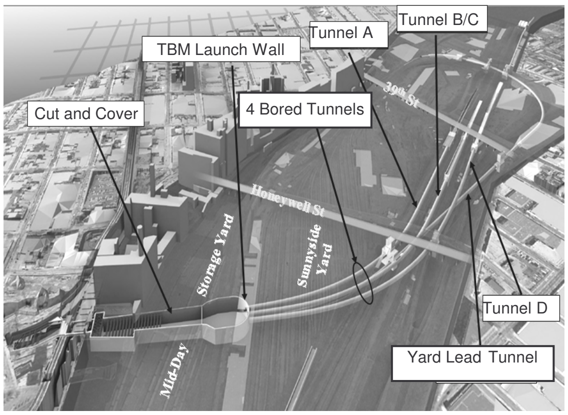


Figure 1. Aerial view of bored tunnels in Queens

2 GEOTECHNICAL DATA AND SUBSURFACE CONDITIONS

The subsurface conditions for the project site in Sunnyside Yard and Harold Interlocking, Queens were derived from over three hundred borings drilled for project design. Additional information was obtained from the logs of older borings that were drilled in the same general area for past engineering studies.

The Queens site is underlain by metamorphic bedrock that is covered by glacial, interglacial, and post-glacial deposits. In the vicinity of Honeywell Street Bridge, the bedrock is expected to be more than 30 m below the ground surface.

The Honeywell Street Bridge piers are founded on spread footings in the mainline tracks embankment. The bottom of the footings are approximately 3 m below the top of the embankment and the embankment is approximately 6 m higher than adjacent ground elevations (see Figure 2). In this area the embankment largely consists of fill. Below the fill is an 8 to 30 m thick layer of glacial till/outwash deposits.

The embankment fill consists of a loose to dense mixture of coarse to fine sand with silts, gravels, and miscellaneous debris. Typical standard penetration test (SPT) blow count, N values range from 3 to over 100. The underlying layer of glacial till/outwash deposits consists of dense to very dense sand with silts and gravels. Cobbles and boulders are expected to be common. Typical SPT N values for the till/outwash deposits range from 7 to greater than 100. The water table is a few meters below the surface of Sunnyside Yard and approximately ten meters below the top of the mainline tracks embankment.

At Honeywell Street Bridge, the tunnels will pass below the embankment and through the layer of glacial till/outwash deposits. The top of tunnels A, B/C, and D will be about 7 m below the top of the embankment in this area. The top of the tunnel LL will be about 21 m below the embankment surface.

3 HONEYWELL STREET BRIDGE

Honeywell Street Bridge is a 22-span bridge that carries four (4) lanes of highway traffic over the mainline tracks (AMTRAK and LIRR) embankment. The original bridge was constructed in the early 1900's just prior to the construction of Sunnyside Yard. In the early 2000's, the steel superstructure was demolished and replaced with eight (8) lines of new steel girder and column frames. In addition, the existing crashwalls were reconstructed but the existing spread footings were used to support the new superstructure.

At each pier, the bottom of each steel girder is welded to the top of its supporting steel column with full penetration groove welds in order to achieve a rigid (moment) connection. The bottom of each column is connected to the top of the footing with a base plate and two anchor rods. This detail acts as a pinned connection and minimizes the transfer of horizontal loads from the superstructure to the footing.

It was noted that the welded connections at the top could become overstressed or develop cracks due to potential settlement and/or horizontal movement of the pier footings following the tunnel borings. Other potential problems would be overstresses in the girders and damage to the expansion joints in the steel grid deck. In order to prevent these problems, it was considered imperative to develop an underpinning scheme that would protect the steel superstructure. It was determined from the settlement analyses and from recommendations from the geotechnical team that Piers 4, 5, and 6 would require some form of underpinning prior to boring the tunnels.

4 PRELIMINARY SETTLEMENT ANALYSIS

An approximate method of analysis (proposed by Peck 1994) was first used to estimate the magnitudes of potential settlements below the bridge piers. A transverse and longitudinal settlement profile was developed for each tunnel using the inverted normal distribution curve method. A 1% ground loss coefficient was assumed. Overlapping transverse profiles were combined to create a resulting transverse settlement profile. Estimated pier settlements were determined from this profile. The results from the preliminary analyses showed that the bridge piers would experience large settlements. This would require that these piers be underpinned. In order to confirm these results, it was decided that detailed analyses using finite element modeling should be performed.

5 FINITE ELEMENT SETTLEMENT ANALYSIS

The finite element analysis software PLAXIS was used to determine the settlement of bridge piers 3, 4, 5, and 6. Figure 2 shows the cross section of the finite element mesh with the mainline tracks embankment, the underlying layer of glacial till, the bridge piers, and the four tunnels.

An elasto-plastic, Mohr-Coulomb model was used to approximate the behaviour of the two soil layers in the analyses. This model requires input for five soil properties, the unit weight γ, Young's modulus E, Poisson's ratio ν, cohesion *c'*, and friction angle ϕ'. Soil properties for the embankment fill and the glacial till are listed in Table 1.

Table 1. Typical Parameters for Soil and Bridge Foundation

Material		Layer 1 Fill	Layer 5 Glacial till	Bridge foundation
Type		Mohr-Coulomb		Elastic
γ	[kN/m³]	20.43	22.00	23.57
E	[kN/m²]	20.68	68.95	24855.60
ν	[-]	0.3	0.3	0.2
c'	[kN/m²]	0.001	0.001	---
ϕ'	[°]	30.0	38.0	---

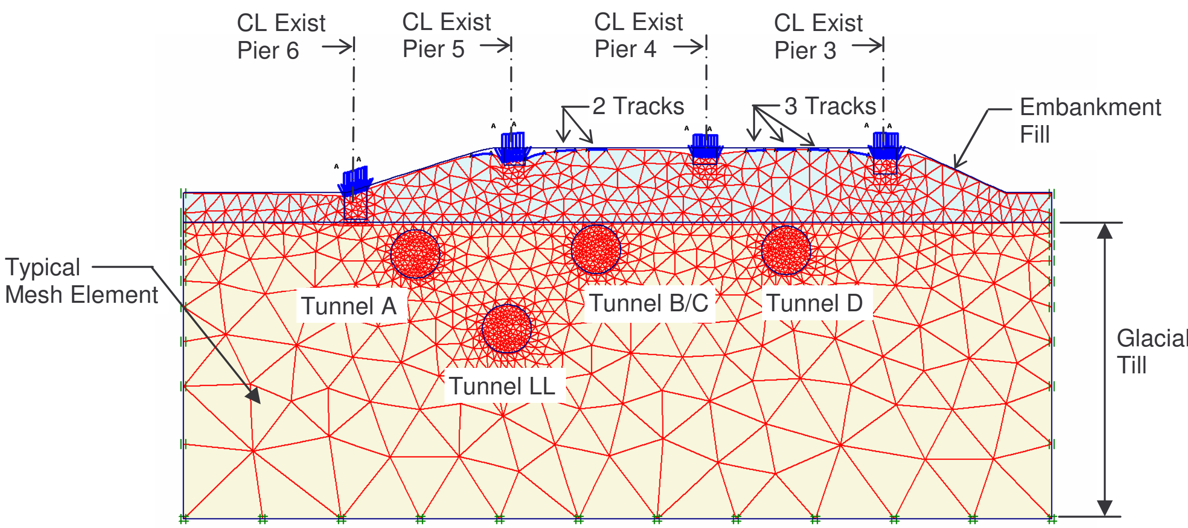


Figure 2. Typical Cross Section of Finite Element Model at Honeywell Street Bridge

Table 2. Construction phase information

Phase No.	Starting phase	Actions	Purpose
1	*0*	Initialize ground condition	Initialize displacement field due to slope
2	*1*	Activate bridge DL	Settlement of piers under dead load
3	*2*	Activate bridge DL+LL	Settlement of piers under both dead and live load
4	*3*	Excavate Tunnel LL and apply relaxation factor	Additional settlement due to Tunnel LL construction
5	*4*	Activate tunnel liner LL	
6	*5*	Excavate Tunnel A and apply relaxation factor	Additional settlement due to Tunnel A construction
7	*6*	Activate tunnel liner A	
8	*7*	Excavate Tunnel D and apply relaxation factor	Additional settlement due to Tunnel D construction
9	*8*	Activate tunnel liner D	
10	*9*	Excavate Tunnel BC and apply relaxation factor	Additional settlement due to Tunnel BC construction
11	*10*	Activate tunnel liner BC	

Since the pier footings are shallow foundations, they are modeled as embedded concrete blocks in the top of the embankment. A linear elastic model with three parameters, the unit weight γ, Young's modulus E, and Poisson's ratio ν was used for each footing. This simple model was considered sufficient because it was not intended to determine structural forces in the footings. The material properties for the footings are listed in Table 1.

Each tunnel lining was modeled as a continuous elastic beam with an axial stiffness EA and a flexural rigidity EI. The flexible rigidity was reduced to 75% of their full value to account for the presence of joints. Alternative approaches to estimate the stiffness of the tunnel lining were investigated including modeling the joints as pins or springs with a prescribed stiffness. The analyses indicated that the continuous beam with reduced stiffness results in a more conservative but reasonable estimate of the liner stresses. More details can be found in Smith et al. (2008).

The PLAXIS analysis simulates the stages of the bored tunnel construction by means of a series of construction phases which are activated in sequence. Table 2 lists the sequence of construction phases (loading steps) used in each analysis. This sequence follows the actual construction schedule which is to first bore Tunnel LL, followed by Tunnel A, Tunnel D, and finally Tunnel B/C.

There are two ways to model the bored tunnel construction in PLAXIS. The first one is the stress relaxation method where the soil is allowed to deform a certain amount due to stress relaxation before the tunnel liner is activated (installed). The second one is the volume loss method, where the tunnel liner is activated and the soil inside excavated, and then the tunnel liner contracts a certain percentage of the tunnel area. Both methods were used in each settlement analysis that was performed.

Since the intent of the analyses was to check pier settlements (a serviceability condition), the working stress method was used. That is, no load factors were applied to the loads.

6 ASSUMPTIONS

The existing bridge has been in service for nearly 100 years. It was assumed that the ground under the piers is fully consolidated and that no additional settlement of the piers would occur if the ground were left undisturbed. Therefore, the intent of the analysis discussed in this paper was to determine pier settlements due only to tunnelling.

For each loading step (phase) of the analysis, it was assumed that the full dead load of the piers and tributary superstructure were activated in the model. Live load on the bridge is relatively small but was also activated.

7 CONSTRUCTION CASES FOR ANALYSIS

As noted earlier, the tunnels in the vicinity of Honeywell Street Bridge will have relatively shallow cover below the ground surface. As a result, the amount of ground settlement will depend largely on how carefully the tunnel boring operations are controlled.

Because it is difficult to predict the construction problems that may arise during tunnel boring, the settlement analysis was performed for the following three construction cases:

1. Case 1 – Good Construction Practice
2. Case 2 – Normal Construction Practice
3. Case 3 – Poor Construction Practice

For the Case 1 analyses, it is assumed that tunnel boring operations proceed with almost no construction problems. A stress relaxation factor of 0.5 or a volume loss of 0.5% was used for every tunnel.

For the Case 2 analyses, it is assumed that tunnel boring operations proceed with some construction problems but with none of them significant. A stress relaxation factor of 0.6 or a volume loss of 1% was used for every tunnel.

For the Case 3 analyses, it is assumed that instability develops in the ground ahead of the face of the TBM. Note that it is highly unlikely that this condition would occur in all four tunnels at the same section. Therefore, for each Case 3 analysis, it was assumed that only one of the four tunnels would experience poor conditions at the face and that this tunnel would have a volume loss of 3%. The other three tunnels would experience normal construction conditions and have a volume loss of 1%.

8 SETTLEMENT ANALYSIS RESULTS

The results for the Case 1 analyses (good construction practice) are shown in Table 3 and 4 below. These results are based on the stress relaxation method using a factor of 0.5. Table 3 shows the cumulative settlement below piers 3, 4, 5, and 6 after each tunnel is bored. All settlements are assumed to be uniform along the length of the footings. The maximum pier settlement is expected at Pier 5 with a settlement of 10.92 mm. Table 4 shows the differential settlements between adjacent piers. The maximum differential settlement of 9.40 mm will be between Piers 5 and 6.

Table 3. Settlement of Bridge Piers due to Tunnelling, mm
Case 1 – Good Construction Practice
Stress Relaxation Factor = 0.5

Pier No.	6	5	4	3
After Tunnel LL	1.27	6.60	1.52	0.51
After Tunnel A	1.78	7.87	1.78	1.02
After Tunnel D	1.78	7.87	5.84	3.05
After Tunnel BC	1.52	10.92	7.62	3.05

Table 5 shows the results for the Case 2 analyses (normal construction practice) using both the stress relaxation factor and volume loss methods. The maximum pier settlement will be approximately 26 mm at Pier 5 after all of the tunnels have been constructed.

Table 4. Differential Settlement between Bridge Piers, mm
Case 1 – Good Construction Practice
Stress Relaxation Factor = 0.5

Span No.		6		5		4	
Pier No.	6		5		4		3
After Tunnel LL		5.59		5.08		1.02	
After Tunnel A		6.10		6.10		1.02	
After Tunnel D		6.10		2.03		2.79	
After Tunnel BC		9.40		3.30		4.57	

Table 5. Settlement of Bridge Piers due to Tunnelling, mm
Case 2 – Normal Construction Practice

Pier No.	6	5	4	3
Using a volume loss 1%	6.10	26.16	16.51	6.10
Using a stress relaxation factor	3.05	17.02	11.94	8.64

The results for the Case 3 analyses (poor construction practice) are shown in Table 6 below. As noted earlier, it was assumed that only one of the four tunnels would experience failure conditions with a volume loss of 3%. The other three tunnels would experience normal construction conditions with a volume loss of 1%.

Table 6. Settlement of Bridge Piers due to Tunnelling, mm
Case 3 – Poor Construction Practice
Volume Loss Method

Pier No.	6	5	4	3
Volume loss 3% at tunnel A	7.62	64.52	18.29	11.18
Volume loss 3% at tunnel BC	5.33	50.80	16.26	10.67
Volume loss 3% at tunnel D	5.08	24.64	36.58	24.64
Volume loss 3% at tunnel LL	9.14	52.32	21.84	7.87

9 TOLERABLE MOVEMENT OF BRIDGE AND UNDERPINNING SOLUTIONS

Tolerable vertical movements of highway bridge piers are specified in Sect. 4.11.3.3. of AASHTO Design Specifications for Highway Bridges, 17th Edition (AASHTO 2002) and related publications (Duncan & Tan 1991 and Moulton et al. 1982). The criteria for tolerable vertical settlement of piers for continuous bridges in expressed in terms of angular distortion, 'A'. Angular distortion is a measure of the differential settlement between adjacent bridge piers and is defined, $A = \delta/S$. δ is the computed differential settlement between the piers at two ends of the span and S = span length. This criterion recommends that the maximum angular distortion between adjacent piers for continuous bridges be limited to 0.004.

The maximum differential settlement of 56.9 mm occurs between Piers 5 and 6 under the Case 3 (poor conditions) analysis. (See Table 6, failure at the face of Tunnel A.) The span between these piers is approximately 21.2 m (21 200 mm). The tolerable differential settlement is calculated as 0.004 x 21 200 = 84.8 mm which is greater than the 56.9 mm settlement expected from the tunnelling. In other words, the expected differential settlement would easily be tolerable if the AASHTO criteria were followed rigorously.

However, the welded steel superstructure frames of the Honeywell Street Bridge are relatively unique and it is expected that they would incur damage or become overstressed if a pier settled too much. Furthermore, it is assumed that the owners of the bridge (NYSDOT) would probably not want the differential settlement to exceed say 25 mm. Based on two similar projects (Gomes et al. 2004 and Takahashi et al. 2004) and experiences from Taisei Construction Corp (Liu 2005), it was decided that using the AASHTO Movement Criteria would be nonconservative and that no differential settlement should exceed 10 mm. The results of the analyses were reconsidered and it was decided that Piers 4, 5, and 6 would require underpinning.

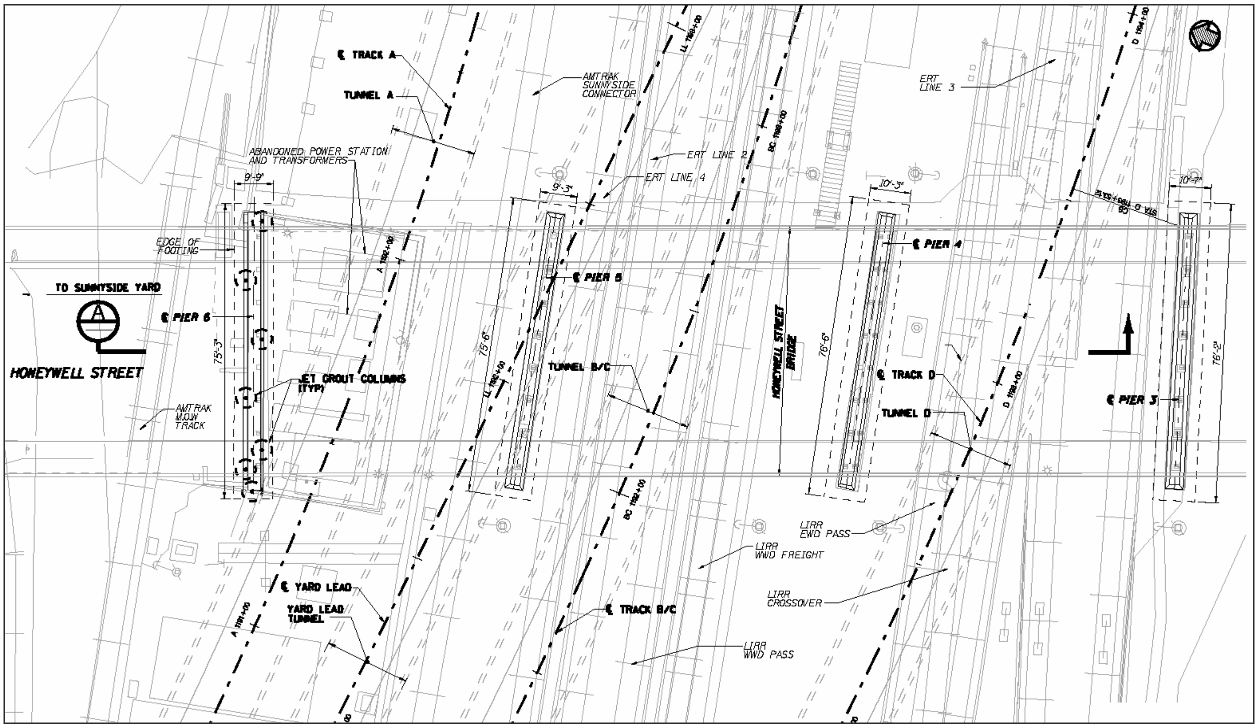
10 UNDERPINNING SOLUTIONS

Because construction access to the site is difficult and there are too many physical restraints (utilities, tracks, etc), several underpinning schemes were considered for Piers 4, 5, and 6. It was also noted that Tunnel A will pass relatively close to the south side of Pier 6 while the other tunnels will pass nearly 8 m below piers 4 and 5.

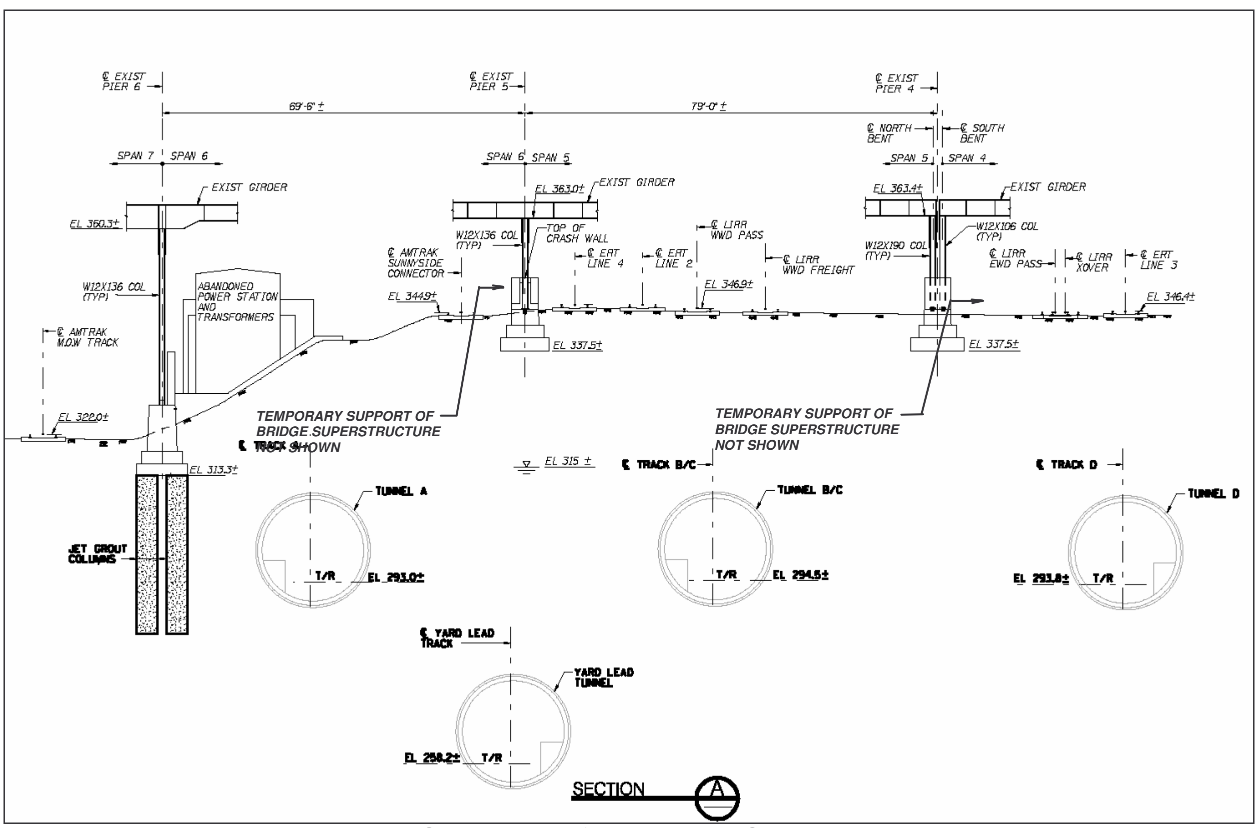
Due to the proximity of Tunnel A to Pier 6 and following the recommendations of the geotechnical team, it was decided that this pier should be underpinned with jet grout columns. As shown in Figure 3, there will be jet grout columns which carry the loads from the pier well below the bottom of the tunnel.

Direct underpinning of piers 4 and 5 with jet grout columns was considered but rejected because of difficult access and limited space for the drilling rig. (See Figure 3). Therefore, at these piers the bridge superstructure will be supported on steel frames which will be supported on the footings. The intent will be to maintain the superstructure at its existing elevation with jacks as the tunnels are bored and for some time after.

The underpinning schemes for piers 4, 5, and 6 are suggested solutions developed by the PB/PTG/STV design team. The contractor will be allowed to propose alternative but similar schemes based on his experiences and preferences. However, any alternative schemes must meet the criteria set forth in the construction contract.



a) Plan View of Site Layout



b) Section View of Underpinning Solutions

Figure 3. Underpinning Solutions for Honeywell Street Bridge Piers

11 CONCLUSIONS

A case study of numerical settlement analyses of bridge piers due to tunnelling is presented in this paper. Two methods were used to simulate the effects of the tunnelling in the analysis, the stress relaxation factor and volume loss percentage were used to quantify the tunnelling construction risks. The maximum settlement at each end of each pier was determined as was the differential settlement between adjacent piers. The calculated settlements were compared to the tolerable settlements for highway bridge substructures as specified in NCHRP Report 343. It was determined that bridge piers 4, 5, and 6 should be underpinned during tunnelling and for a short time after. The proposed design solution is to underpin Pier 6 with jet columns and to directly support the bridge superstructure at Piers 4 and 5 with steel frames.

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