

TEMPORARY EXCAVATION SUPPORT SYSTEM FOR A NEW RAILWAY BRIDGE AND HIGHWAY UNDERPASS IN PITTSBURGH

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ABSTRACT: Until recently, traffic exiting Pittsburgh would use the West End Bridge to cross the Ohio River only to become entangled in the West End Circle, which consisted of two railway overpasses each out-of-alignment with the West End Bridge. A roadway improvement project involving the construction of a new railway bridge and highway underpass was undertaken to eliminate this traffic circle and provide a direct connection between the West End Bridge and State Route 19. Brayman Construction Corporation has since completed this project.

This paper provides a summary of the design and construction sequencing of a temporary sheet pile excavation support system that was constructed to facilitate the completion of this project. The excavation support system consisted of opposing sheet pile walls (shoring and anchor walls) cross-tied with prestressed tendons beneath the railroad corridor. The general arrangement of this system is shown in Figure 1. The prestressed tendons limited the horizontal wall displacement to permissible levels specified by the railway owner, and the reuse of the phase one shoring wall as the shoring wall for the second phase of bridge construction eliminated the need to extract and re-drive sheeting, resulting in reduced displacements and construction time.

INTRODUCTION

For the construction of the West End improvement project described above, railway traffic on the existing embankment could not be interrupted, except for a brief period to shift tracks for each phase of construction. Since the excavation support system was offset approximately 3 m (10 ft) from the centerline of the nearest track, the railway owner specified that the cumulative horizontal displacement at the top of the wall be limited to 19 mm (0.75 in) to control vertical settlement of the railbed.

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To satisfy these stringent constraints, a continuous AZ-36 sheet pile shoring wall of 12.5 m (41 ft) maximum height was cross-tied beneath the railway corridor to an anchor wall consisting of discrete pairs of AZ-18 sheet piling. The use of prestressed tendon ties limited the horizontal movement of the top of the wall to 13 mm (0.5 in). Photos of the shoring wall during construction are provided in Figures 2 and 3, and the key factors governing design and construction are described below.

GEOTECHNICAL PROPERTIES AND RAILWAY LOADING

Embankment materials at the site consist of random mixtures of soils, slag, cinders and rock fragments, with a silty to clayey soil component predominating. Excluding outlier values, SPT N values between 10 and 35 blows per 305 mm penetration were typical. The properties for these embankment soils were adopted in accordance with AREMA (2001) as follows: $\bar{\phi} = 30^\circ$ and $\gamma = 17.3 \text{ kN/m}^3$ (110 pcf).

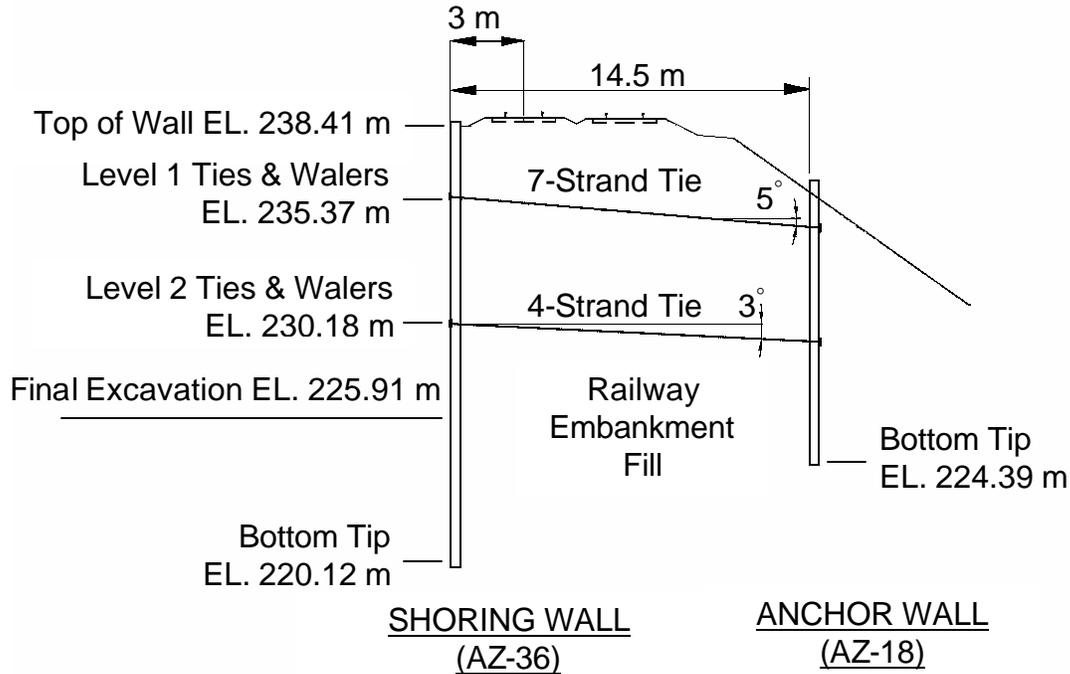
The railway loading applicable to the structural design of all wall components was governed by the Cooper E-80 configuration (AREMA, 2001), with the maximum 356 kN (80 kip) axle load distributed over the minimum 1.52 m (5 ft) axle spacing and 2.59 m (8.5 ft) tie width, resulting in a 90 kPa (1.88 ksf) vertical surface surcharge. The horizontal component of this vertical stress acting on the shoring wall was conservatively calculated using a Boussinesq distribution for a frictionless, rigid wall, as prescribed by the railway owner for structural design. In analyzing the final condition, each of the two tracks was loaded to the 90 kPa (1.88 ksf) vertical surcharge. However, for the construction conditions, the railway owner permitted the design load for the far track to be reduced to a vertical surcharge of 45 kPa (0.94 ksf).

WALL CONFIGURATION AND GENERAL EXCAVATION AND CONSTRUCTION SEQUENCE

Notably, the support system described herein constituted a more constructable and better performing alternate to the original concept, which required continuous sheet pile anchor walls and numerous tie rods, inherently imposed strict installation tolerances, and presumed sustained track outages which were deemed unacceptable by the railway owner. The alternate anchor wall was configured with discrete pairs of sheet piling and discrete wales to afford the contractor generous tolerances for tie installation. Also, strand ties were selected in lieu of tie rods to greatly reduce the number of ties and drilling requirements and provide better control of wall deflections. Additionally, the anchor wall alignment was shifted further from the railroad tracks to enable the contractor to install the wall without being restricted by railroad operations.

Driving of Anchor and Shoring Walls

The anchor wall sheet piling (AZ-18, Grade 50) was first installed to the depth limits shown in Figure 1. After the railway tracks were relocated for the Phase 1



NOTE: Horizontal Tie Spacing = 2.5 m (TYP.)

FIG. 1. Cross Section of Temporary Cross-Tied Sheet Pile Shoring and Anchor Walls, Phase 1 Construction, at a Maximum Shoring Wall Height of 12.5 Meters

detour, the shoring wall sheet piling (AZ-36, Grade 50) was driven to the depth limits shown in Figure 1, with the top of the shoring wall at El. 238.41 m (782 ft).

Initial Excavation for Temporary and Permanent Free-Cantilever

An initial 1.52 m (5 ft) high excavation to El. 236.89 m (777 ft) was advanced in front of the shoring wall to establish a construction platform and work area. The railway owner would not permit a temporary (construction) cantilever condition greater than 3.66 m (12 ft) high or a permanent cantilever condition greater than 3.05 m (10 ft) high and restricted the exposed length of the temporary 3.66 m (12 ft) high cantilever to 30.5 m (100 ft) prior to Level 1 tie installation. Recognizing these restrictions, a limited excavation was conducted in front of the shoring wall to El. 234.76 m (770 ft; a temporary 3.66 m (12 ft) high cantilever) for installation of an initial section of Level 1 ties at El. 235.37 m (772 ft).

Level 1 Strand Tie Installation and Prestressing

An excavation in front of the anchor wall to El. 233.54 m (766 ft) was required to install the Level 1, 7-strand ties, which were spaced every 2.5 m.



FIG 2. Shoring Wall Under Railway Loading



FIG 3. Shoring Wall After Installation of Level 2 Ties

The strand ties were prestressed to 0.45 GUTS, usually from the shoring wall side. As the strand ties along a given section of wall were stressed and locked off, the excavation limits were extended by an equivalent length to advance the installation of the remaining Level 1 ties.

Excavation for Level 2 Strand Tie Installation and Pre-stressing

After incrementally installing the Level 1 ties, the Stage 2 excavation to El. 229.57 m (753 ft) and El. 228.96 m (751 ft) in front of the shoring and anchor walls, respectively, commenced to enable the installation of the Level 2, 4-strand ties spaced every 2.5 m at El. 230.18 m (755 ft). Once the Level 2 ties were installed and prestressed, backfill was placed in front of the anchor wall to approximate the original grade of the embankment.

Final Depth of Excavation, Shoring Wall Side

The critical portions of the shoring wall employed two tie levels, as previously described, and required a final excavation to El. 225.91 m (741 ft), as shown in Figure 1, resulting in a 12.5 m (41 ft) high wall. Only one tie level (8-strand ties) was required for the remaining portions of the wall where the final excavation line ranged from El. 229.57 m (753 ft) to El. 228.20 m (748.5 ft). The final excavation for the abutment construction was phased to closely coincide with the initiation of foundation construction.

HORIZONTAL DISPLACEMENT PREDICTIONS AND MEASUREMENTS

As noted previously, the design of the excavation support system was mainly driven by the railway owner's requirements to limit disruption of train traffic and control horizontal wall displacements and settlement of the railbed. In the following subsections, horizontal wall displacements for the cantilever (both temporary and permanent) and single tie level conditions are evaluated. In addition, the observed displacements are discussed.

Predicted Displacement – Temporary and Permanent Cantilever Conditions

The shoring wall horizontal displacement predictions presented in this section apply to the temporary and permanent cantilever conditions (i.e., Stage 1 excavation for tie installation and permanent free cantilever wall sections). The method used to predict these cantilever wall displacements involved a two-step process. First, the maximum horizontal displacement at the top of the wall associated with the development of the active earth pressure was determined, without consideration of the effect of the railroad surcharge. The relationships reported by Clough and Duncan (1991) were employed. As a consequence of the pre-loading effect of the railway traffic on the existing embankment soils, the relationship for medium-dense sand was adopted. On this basis, the horizontal displacement required to mobilize the active earth pressure is approximately 0.002 times the exposed cantilever wall height h_c .

The additional effect of the railroad surcharge on the horizontal displacement was determined by introducing the factor f_{qh} , which is computed as follows:

$$f_{qh} = 1 + \frac{\Delta\sigma_h}{\bar{\sigma}_h} = 1 + \frac{\Delta\sigma_v}{\bar{\sigma}_v} \quad (1)$$

where:

$\Delta\sigma_h$ = change in horizontal stress at depth h_c (cantilever height) due to the horizontal component of the railroad surcharge;

$\bar{\sigma}_h$ = horizontal effective stress at depth h_c due to the overburden (embankment) soil;

$\Delta\sigma_v$ = change in vertical stress at depth h_c due to the vertical component of railroad surcharge; and

$\bar{\sigma}_v$ = vertical effective stress at depth h_c due to the overburden (embankment) soil.

The values $\Delta\sigma_h$, $\Delta\sigma_v$, $\bar{\sigma}_h$, and $\bar{\sigma}_v$ are related to the coefficient of active earth pressure, K_a as follows:

$$\frac{\Delta\sigma_h}{\Delta\sigma_v} = \frac{\bar{\sigma}_h}{\bar{\sigma}_v} = K_a = \frac{1 - \sin \bar{\phi}}{1 + \sin \bar{\phi}} \quad (2)$$

The combined effect of active earth load and railway surcharge on the horizontal displacement (δ_h) at the top of a cantilever wall of height h_c can then be expressed as:

$$\delta_h = 0.002 h_c f_{qh} \quad (3)$$

A friction angle ($\bar{\phi}$) of 30° and unit weight (γ) of 17.3 kN/m^3 (110 pcf) were used to calculate K_a and $\bar{\sigma}_v$, consistent with the prescribed properties. The results of this two-step procedure are summarized in Table 1.

TABLE 1. Summary of Predicted Horizontal Displacements at Top of Wall for the Temporary and Permanent Cantilever Conditions

Exposed Cantilever Height h_c , m (ft)	Maximum Horizontal Displacement at Top of Wall, δ_h mm (in)	
	Temporary Cantilever Condition	Permanent Cantilever Condition
3.66 (12)	12.78 (0.503)	N/A
3.05 (10)	N/A	14.33 (0.564)

Concerning values of $h_c = 3.66$ and 3.05 m (12 and 10 ft), the predicted maximum horizontal displacements at the top of the shoring wall for both the

temporary and permanent cantilever conditions are less than the specified limit of 19 mm (0.75 in), with values for the temporary condition approximating 13 mm (0.5 in).

Tie Prestress and Predicted Wall Displacement - Single Tie Level

In determining the required prestress for the upper tie level, the following incremental displacements for three stages of construction were considered:

Stage 1: *Temporary 3.66 m (12 ft) cantilever with ground surface at El. 238.41 m (782 ft) and cut to El. 234.76 m (770 ft)*

Displacement for this stage is addressed in Table 1.

Stage 2: *Installation and Prestressing of Level 1 Strand Ties at El. 235.37 m (772 ft)*

Displacement reduction due to wall movement into the existing embankment.

Stage 3: *Excavation to El. 228.20 m (748.5 ft)*

Displacement at this stage has two potential components:

3a.) Strand tie elongation (if the prestress level is less than the actual induced reaction [presumed to be 0.6 GUTS])

3b.) Structural cantilever movement (with $h_c = 3.05 \text{ m} = 10 \text{ ft}$) above the tie level, which occurs in response to the increase in railroad surcharge between the temporary and permanent conditions. The average increase in horizontal surcharge pressure within the top 3.05 m of the shoring wall is approximately 4.8 kPa (0.1 ksf).

The calculations demonstrated that a prestress level of 0.42 GUTS would control the maximum horizontal displacement of the top of the shoring wall to less than 19 mm (0.75 in). Therefore, a target prestress level of 0.45 GUTS was subsequently specified. Incremental and cumulative horizontal displacements for Stages 1 through 3 are summarized in Table 2 for a prestress level of 0.42 GUTS. For sections of walls with two rows of ties, and a final excavation to El. 225.91 m (741 ft), the effects of prestressing the bottom row of ties to 0.45 GUTS and the subsequent additional excavation were approximately equal and opposite, resulting in no appreciable influence on the resulting cumulative lateral displacement for the single tie condition provided in Table 2.

Although prestressing the cross-tie tendons to the full design load of 0.6 GUTS would have resulted in lower calculated horizontal wall displacements, this option was not selected due to concern for potential strand overstress, especially considering that a significant proportion of the design load was transient.

Observed Displacements

The maximum horizontal displacement at the top of the shoring wall during construction was no more than 13 mm (0.5 inch), which was somewhat less than the predicted value of 18.7 mm. The conservative nature of the displacement estimate was most likely attributable to the fact that the prescribed Cooper E-80 loading was

developed for steam locomotives, which are generally heavier than modern day diesel engines.

TABLE 2. Summary of Predicted Incremental and Cumulative Horizontal Displacements at Top of Wall for Single-Tie Wall Section, with 0.42 GUTS Tie Prestress

Stage	Excavation El. m (ft)	Vertical RR Surcharge	Baseline Horiz. Displacement @ T.O.W., mm (in)
1 – Temp. Cantliever	234.76	Near: 90 kPa	$\delta_1 = 12.8 (0.503)$
2 – Level 1 Tie Prestressing	(770.0)	Far: 45 kPa	$\delta_2 = -23.3 (-0.916)$
3 (a) – Tie Elongation	228.20	Near: 90 kPa	$\delta_{3a} = 28.9 (1.139)$
3 (b) – Added RR Surcharge	(748.5)	Far: 90 kPa	$\delta_{3b} = 0.3 (0.012)$
<i>Predicted Cumulative Displacement Single Tie Wall Section</i>			$\mathbf{d}_i = 18.7 (0.738)$

SUMMARY

This paper has summarized the design and construction sequencing of a temporary excavation support system consisting of opposing sheet pile walls (shoring and anchor walls) cross-tied with prestressed tendons beneath a railroad corridor to facilitate the construction of a new railway bridge and highway underpass in Pittsburgh’s West End. The horizontal offset of the proposed anchor wall from the shoring wall, as shown in Figure 1, was greater than that set forth in the original design, which permitted continuous operation of railway traffic during all phases of construction, with the exception of brief periods for track relocation for each phase of bridge construction. Also, the use of prestressed tendon ties limited the horizontal wall displacement to permissible levels specified by the railway owner. In addition, the reuse of the shoring wall from Phase 1 bridge construction as the shoring wall for the second phase eliminated the need to extract and re-drive sheeting, which resulted in reduced displacements and construction time.

REFERENCES

- AREMA (2001). “Manual for Railway Engineering,” American Railway Engineering and Maintenance-of-Way Association.
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