Monitoring Lateral Earth Pressures and Movements of Cut Retaining Walls

W. Allen Marr
Jerry DiMaggio
Dori Ross
Martin Hawkes
Seda Gokyer

Geocomp Corporation

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16. Abstract

The design and performance of retaining walls requires accurate estimates of the lateral earth pressures for strength limit states and service limit states. Several design methods are available and are used to predict the lateral earth pressures acting on the wall as well as the lateral and vertical wall movements of the wall and of the soil behind the wall. The magnitude of the lateral earth pressure is dependent on the lateral deformation of the wall and the deformation is dependent on the lateral earth pressure. This dependence creates a complex soil-structure interaction problem.

Wisconsin Department of Transportation (WisDOT) developed this research project to evaluate how to best predict lateral earth pressures and wall movements for use in their future cut wall projects. The research aims to develop guidance to accurately predict horizontal and vertical movements of cut retaining walls, obtain data for calibration of specific design methodologies, and provide recommendations for limit states that can be used to control wall performance. Two cut retaining walls were instrumented and remotely monitored over a 15-month time period. During this time period automated readings were collected to obtain measurements of the wall displacements, strains in the wall's structural elements, and pore pressures in the retained soil. Comparisons were made by the research team for the cut retaining walls studied to successfully compare actual measured performance of the walls with the predicted design performance using several commonly used design methods.

Based on the results of this study, the research team has provided several recommendations for WisDOT to consider for its future wall design practice. These recommendations include:

- Designers should include all applicable load cases to ensure that worst case loading, or a combination of loading is addressed. WisDOT should also consider developing standard details for protection against pore water pressure buildup and ground freezing behind the wall.
- Designers should consider both undrained and drained cases for each wall design to cover various possibilities that can develop in the field during construction and post construction.
- Designers using the PY-WALL method should obtain soil parameters for actual site conditions using a pressuremeter or lab testing on undisturbed samples rather than using the p-y curves internally generated by the software.
- WisDOT should consider requiring performance testing of a representative number of anchors in its specifications to reduce uncertainty in the actual anchor lock-off loads.
- WisDOT should consider requiring more detailed documentation by contractors of their sequence of work and the dates work is performed for retaining wall construction.
- For unusual cases, and cases where poor wall performance could create significant risks and costs, strong consideration should be given to instrumenting and monitoring representative wall sections.
- For cut walls where the zone of influence of the construction might include existing utilities and/or buildings that could be impacted by ground settlement, methods other than SPW911 and PW-WALL should be used to predict ground settlements as neither of these programs calculate ground settlement behind the wall.

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EXECUTIVE SUMMARY

NEED FOR THE STUDY

Cut-retaining walls constructed from top-down such as cantilevered walls, anchored soldier pile and lagging walls, sheet pile walls, and tangent/secant pile walls are used on numerous transportation projects at the Wisconsin Department of Transportation (WisDOT). The design and performance of these retaining walls requires accurate estimates of lateral earth pressure against the wall to evaluate safety against soil failure at the strength limit state, as well as prediction of wall movements at the service limit state. Several design methods are available and are used to predict the lateral earth pressures acting on a wall as well as the lateral and vertical wall movements of the wall and of the soil behind the wall. The magnitude of the lateral earth pressure is dependent on the lateral deformation of the wall, and the deformation is dependent on the lateral earth pressure. This dependence creates a complex soil-structure interaction problem. WisDOT developed this research project to evaluate how to best predict lateral earth pressures and wall movements for use in design of their future cut wall projects. The research aims to develop guidance to accurately predict horizontal and vertical movements of cut retaining walls, obtain data for calibration of specific design methodologies, and provide recommendations for limit states that can be used to control wall performance.

GOALS & OBJECTIVES

Cut-retaining walls are used for a variety of transportation applications. Controlling the deformations of retaining walls is achieved through the methodology used to design the system and by execution of good construction procedures. The goal for designers and contractors alike is to provide a safe final structure for strength-based limit states and maintain movements within tolerable levels for service-based limit states. Failure to accurately predict lateral earth pressures or movements can create retaining wall system failures or cause significant damage to adjacent structures and utilities.

The research project objectives of this project are to investigate the short-term and long-term performance of cut retaining walls by measuring the top-of-wall lateral deflection of a cut retaining wall over time, the magnitude and distribution of deformations along the height of the wall over time, and the vertical ground settlement behind the wall over time. Using these measurements, the research team will then compare the back calculated lateral earth pressures and retaining wall movement versus estimated wall movement generated from commonly used methods and computer programs and will determine if modifications to existing design methodologies should be made based on the findings of the research.

USED RESEARCH APPROACH

Geocomp teamed with Applied Research Associates (ARA) for this project (research team). WisDOT and the research team jointly developed a research approach to monitor two cut wall sections on an active construction project. One chosen section consisted of a cantilevered retaining wall and the other section consisted of a tied-back retaining wall. The research approach was to analyze the design of each of these two walls with different methods and instrument the walls during construction to determine the earth pressures acting on the walls, the deformation of the walls, and the ground settlement behind the walls. Geocomp also added piezometers behind the walls to measure pore water pressures in case these developed. The measured performance of the two walls was then compared to the predicted performance to assess the adequacy of the design methods. This work resulted in conclusions and recommendations regarding design methods to use in future work and how those methods should be applied.

ANALYSIS

The research team reviewed the Forward 45 (design team) designs for both walls in the research project. The design team designed the walls based on analysis using drained soil parameters using PY-WALL (Case (1) below). Earth pressures calculated by the design team included factored surcharge loads including live load and wind load due to the noise barrier wall considering the worst-case scenario for design purposes following WisDOT Bridge Manual. The design team did not consider any pore pressures in the soil since there was no indication of water during their subsurface exploration program. There were also geometry differences for both the cantilever and anchored walls for the design vs. as-built conditions.

The research team used two computer programs SPW-911 and PY-Wall to predict wall performance for several different loading conditions. The research team conducted analyses for the following conditions:

(1) Without hydrostatic pore pressures on both walls under long term drained conditions (this is the condition used by design team) – referred in the text as <u>drained with no pressures.</u>
(2) With hydrostatic pore pressures on both walls under long term drained conditions with pore pressures applied on the wall – referred to in the text as <u>drained with hydrostatic pore pressures.</u>

(3) With measured pore pressures on both walls under long term drained conditions with measured pore pressures applied on the wall– referred in the text as drained with measured pore pressures.

(4) With hydrostatic pore pressures on both walls under short term undrained conditions– referred to in the text as undrained with hydrostatic pore pressures.

Analyses conducted by the research team considered both short- and long-term conditions as well as the effect of pore pressure on the long-term stability of the walls.

RESULTS

The performance monitoring program was successful and indicated consistent and repeatable results from the instrumentation installed. From the VWPZs, it was determined that positive pore pressures developed behind the walls after construction which indicated that the walls may have been preventing lateral seepage. Temperature readings at the piles varied seasonally as was expected. Corresponding SAA readings at Pile 36 indicate outward movement which appeared to be frost related and rebounded back after the winter season. Wall movements measured with the SAAs were very repeatable and consistent over time. Moments deduced from the wall movements with the SAAs were consistent and appear to be superior to those determined from the strain gages. Back calculations of earth pressures from measured deformations using SAA readings were erratic and sensitive to small changes in the readings.

The research team also compared measured performance of the two wall types to calculated design performance which allowed the research team to assess the adequacy of several design methods. Two wall design methods commonly used in Wisconsin were considered by the research team, SPW-911 and PY-WALL. Results from the analyses with SPW-911 show that a drained analysis with hydrostatic pore pressures in SPW-911 leads to results where a wall is not stable. The SPW-911 design results do not agree well with the measured values of displacement and moments. Results from the analysis with PY-WALL show that the use of p-y curves generated from in-situ pressuremeter data results in more realistic deformation and bending moment design estimates and the use of composite stiffness for concrete encased laterally loaded piles can improve deformation and bending moment design estimates but requires careful consideration of cracking moments of the concrete section to determine appropriate flexural stiffness. The PY-Wall program gave results that better compare with the measured values of moment and displacement than the SPW-911 program.

We also examined the effects of adding water pressures into the analyses since significant pore water pressures were measured after wall construction in the retained ground. These results show the importance of adequately considering pore water pressures in the retained backfill for the design of the wall since they can affect both the maximum moment for wall design and the horizontal wall displacement. Buildup of positive pore pressures behind a wall designed with drained conditions and no pore water pressure can result in excessive moments in the wall and larger than anticipated lateral displacements. AASHTO LRFD states that if retained earth is not allowed to drain, the effect of hydrostatic water pressures shall be added to that of earth pressures. Furthermore, AASHTO states that cohesive backfill walls should be designed assuming the most unfavorable conditions with consideration for the development of pore pressures.

In this study we found that the AASHTO LRFD formulations for estimation of earth pressures are adequate for design when all applicable loading conditions are considered. AASHTO LRFD gave forces larger than measured in these two walls but this is to be expected since AASHTO is an envelope for design whereas these measurements are for the smaller in-service conditions.

As noted above, the measurements on the anchored wall showed that the wall moved outward during freezing weather. It is possible that the anchor in the anchored wall experienced an increased load during this time but the instrumented lock off nut failed so there are no measurements to prove this. Neither of the two design methods have a way to determine the effects of ground freezing on wall performance. There is little to no guidance that we could find in literature to deal with freezing ground. Industry practice is to ignore it or take steps to minimize ground freezing by adding insulation to the wall to prevent freezing directly behind the wall.

CONCLUSIONS

Based on the results of this study, the research team has provided several recommendations for WisDOT to consider for its future wall design practice. These recommendations include:

- Designers should include all applicable load cases to ensure that worst case loading, or a combination of loading is addressed. WisDOT should also consider developing standard details for protection against pore water pressure buildup and ground freezing behind the wall.

- Designers should consider both undrained and drained cases for each wall design to cover various possibilities that can develop in the field during construction and post construction.
- Designers using the PY-WALL method should obtain soil parameters for actual site conditions using a pressuremeter or lab testing on undisturbed samples rather than using the p-y curves internally generated by the software.
- WisDOT should consider requiring performance testing of a representative number of anchors in its specifications to reduce uncertainty in the actual anchor lock-off loads.
- WisDOT should consider requiring more detailed documentation by contractors of their sequence of work and the dates work is performed for retaining wall construction.
- For unusual cases, and cases where poor wall performance could create significant risks and costs, strong consideration should be given to instrumenting and monitoring representative wall sections.
- For cut walls where the zone of influence of the construction might include existing utilities and/or buildings that could be impacted by ground settlement, methods other than SPW911 and PY-WALL should be used to predict ground settlements as neither of these programs calculate ground settlement behind the wall.

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1. NEED FOR STUDY

Cut-retaining walls constructed from top-down such as cantilevered walls, anchored soldier pile and lagging walls, sheet pile walls, and tangent/secant pile walls are used on numerous transportation projects, particularly where protection of right-of-way, utilities, roads, and structures are required. Design of these walls requires estimates of lateral earth pressure against the wall to evaluate safety against soil failure at the strength limit state, as well as prediction of wall movements at the service limit state. The wall design must also consider the effects of lateral wall movement and settlement behind the wall to prevent adverse effects to adjacent facilities. Design methods are used to predict the lateral earth pressure acting on the wall as well and the lateral wall movements and settlement of the soil and any structures retained behind the wall.

Over prediction of lateral earth pressure and wall movement may result in a conservative design which may lead to unnecessarily high construction costs. On the contrary, underestimation of lateral earth pressure and wall movement may result in wall failure or excessive wall deflection and/or settlement behind the wall. Excessive wall movement can lead to long-term maintenance problems as well as a reduced service life for retaining structures. Also, potential user delays and increased costs can occur if these movements impact existing flow of traffic either to repair damaged retaining walls or to close roadways in locations where retaining wall movements have become problematic. Excessive wall movements can also have a negative impact on adjacent roadways, buried utilities, and structures. WisDOT initiated this research study to determine the more appropriate methods to effectively estimate lateral earth pressures and wall movement for future projects. The research includes measured performance during construction of a cantilevered wall and an anchored wall.

2. GOALS

Cut-retaining walls are used for a variety of transportation applications. Controlling the deformations of retaining walls is achieved through the methodology used to design the system and by execution of good construction procedures. The goal for designers and contractors alike is to provide a safe final structure for strength-based limit states and maintain movements within tolerable levels. Failure to accurately predict lateral earth pressures or movements can create retaining wall system failures or cause significant damage to adjacent structures and utilities.

The design and performance of retaining walls requires accurate estimates of the lateral earth pressures for strength limit states and service limit states. Several design methods are available and are used to predict the lateral earth pressures acting on the wall as well as the lateral and vertical wall movements of the wall and of the soil behind the wall. The magnitude of the lateral earth pressure is dependent on the lateral deformation of the wall and the deformation is dependent on the lateral earth pressure. This dependence creates a complex soil-structure interaction problem. Wisconsin Department of Transportation (WisDOT) developed this research project to evaluate how to best predict lateral earth pressures and wall movements for use in their future cut wall projects. The research aims to develop guidance to accurately predict horizontal and vertical movements of cut retaining walls, obtain data for calibration of specific design methodologies, and provide recommendations for limit states that can be used to control wall performance.

3. OBJECTIVES

Currently, WisDOT follows guidelines that are given in their Bridge Manual for Non-Gravity Cantilevered Walls which address the design of soldier pile walls. (Retaining Wall design is included in Chapter 14 of the WisDOT Bridge Manual which was last updated on 01/2019). The guidelines provide two methods for design. One method uses a simplified earth pressure distribution diagram as shown in the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications 8th Edition (section 3.11.5.6) for permanent soldier pile walls. In the AASHTO method the passive soil resistance is simplified by assuming a concentrated force in combination with a net active/passive pressure. A second method indicated to be the preferred method uses the "Conventional Method" as described in "United States Steel Sheet Piling Design Manual", February 1974. In the "Conventional Method" Active and passive pressures are computed using the AASHTO method, but a concentrated passive force is not used. In the conventional method moments are taken about the base of the wall to determine the wall depth and passive force for rotational equilibrium.

Both design methods do not address deformations of soil behind the wall. This limitation prevents their use in critical applications where tolerable vertical and horizontal movements are a controlling element for design. Such can be the case where structures and utilities exist close to the wall. The research project objectives are as follows:

- Investigate the short-term and long-term performance of cut retaining walls, specifically soldier pile and lagging walls since they are most commonly built by

WisDOT.

- Measure the top-of-wall lateral deflection of a cut retaining wall over time.
- Measure the magnitude and distribution of lateral earth pressure along the height of the wall over time.
- Measure vertical ground settlement at various points behind the wall over time.
- Compare measured lateral earth pressures and retaining wall movement versus estimated wall movements generated from commonly used methods and computer programs.
- Determine if modifications to existing design methodologies should be made based on the findings of the research.

4. RESEARCH APPROACH USED

Geocomp teamed with Applied Research Associates (ARA) for this project (research team). The project was managed by Co-Principal Investigators W. Allen Marr of Geocomp and Jerry DiMaggio of ARA. WisDOT and the research team jointly developed a research approach to monitor two wall sections on an active construction project. One chosen section consisted of a cantilevered retaining wall and the other section consisted of a tied-back retaining wall. The research approach was to analyze the design of each of these two walls with different methods and instrument the walls during construction to determine the earth pressures acting on the walls, deformation of the walls, and ground settlement behind the walls. Geocomp also added piezometers behind the walls to measure pore water pressures in the event that these developed in the instrumented cases. The measured performance of the two walls was then compared to the predicted performance to assess the adequacy of the design methods. This work resulted in conclusions and recommendations regarding design methods to use in future work and how those methods should be applied.

During planning, the work effort was divided into six (6) tasks: Kickoff Teleconference, Literature Review, Instrumentation, LIDAR Survey, Data Analysis, and Final Report. These tasks are described in more detail below. During the project, the research team had periodic check-ins and coordination meetings that included staff from WisDOT, Geocomp, and ARA. The research team also provided written quarterly updates on the progress of the project and several interim reports of instrumentation data once the instrumentation installations were complete. Additional backup materials are provided in the Appendices of this report as noted in the text.

Task 1: Kickoff Teleconference

The Geocomp project team organized and participated in a kickoff meeting on 07/11/17 with WisDOT Project Oversight Committee (POCs). The project Kick-off Meeting PowerPoint and meeting minutes are included in Appendix A. The purpose of the meeting was to review the objectives of the work and project schedule, to discuss the active wall project and potential instrumentation sites, and to discuss the approach to the work and schedule.

Task 2: Literature Review – Lateral Earth Pressure and Movements of Cut Retaining Walls

The Geocomp project team gathered and reviewed literature, reports, manuals, and plans relevant to lateral earth pressure and movements of cut retaining walls. Appendix B provides a summary of relevant literature and a reference section for all cited references and covers representative papers relevant to this work but is by no means exhaustive. Retaining walls have been the subject of soil mechanics research for more than 100 years, so the literature review is limited to more recent work relevant specifically to cut retaining walls for transportation work.

Earth retaining systems are designed and constructed to hold back and support the vertical or near-vertical slopes of soil and rock. Depending on the construction method, retaining systems are classified into fill wall construction or cut construction. In fill wall construction, the wall is constructed from the base of the wall to the top and backfill is placed behind the wall. On the other hand, in excavation operations, the cut wall is constructed from the top of the wall to the base and then material is cut away from the outside of the wall.

In general, retaining walls are categorized as (a) conventional retaining walls and (b) mechanically stabilized earth (MSE) walls. Conventional retaining walls are classified into several categories including gravity and modular gravity retaining walls; semi-gravity retaining walls such as cantilever retaining walls, counterfort retaining walls and buttress retaining walls; and non-gravity cantilever and anchored retaining walls. The literature review focused on the design and performance analysis of non-gravity cantilever and anchored cut retaining walls, constructed top down, which are the main approaches used to build cut retaining walls – the objective of this study. Specific topics covered in the literature review include design of Cantilever and Anchored Retaining Walls (summary of various analytical design methods), studies on Cantilever and Anchored Retaining Walls, Case Studies of Cantilever and Anchored Retaining Walls, Summary of Design Practices of Select Highway Agencies, and soil Arching Effects.

Task 3: Instrumentation

This task consisted of Instrumenting and monitoring two wall sections on the Zoo Interchange project during and after their construction. The Zoo Interchange is a freeway interchange on the west side of Milwaukee, Wisconsin. It forms the junction of interstate 94, I-893, I-41, US Highway 41 and US Highway 45. It is the busiest and was one of the oldest interchanges in the state prior to construction. It is nicknamed as such because the Milwaukee County Zoo is located on the northwest quadrant of the interchange. The control cities at the interchange are; downtown Milwaukee to the east, Chicago to the south, Madison to the west, and Fond du Lac to the north. The Zoo Interchange R-40-516 wall was instrumented at two locations selected by WisDOT. One section was a cantilevered wall and the other section was an anchored wall. The two sections of Wall R-40-516 which were instrumented are Cantilever Soldier Pile Wall Pile #6 and Anchored Soldier Pile Wall #36. The locations of the two wall sections are show in Figure 4-1 below:

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Figure 4-1 Test Pile location Plan

These two wall sections allowed the research study team to examine the differences between these two cut wall types. Based on the wall and pile locations, an instrumentation plan was developed and implemented for each wall section. Table 4-1 and Table 4-2 below summarize the type, quantity, location, installation detail, and purpose of each installed instrument. Appendix C includes the Instrumentation Installation Report which includes installation logs, installation photos, installation configurations, and calibration reports. Appendix D includes the approximate installation details for both piles.

Table 4-1 Cantilever Soldier Pile Instrumentation Installation Summary, Pile #6

Instrument	Quantity	Location	Installation Note	Purpose
Vibrating Wire Piezometer	3	Behind wall	1 string of three sensors installed in borehole 5 feet behind the wall; P6PZ1, P6PZ2, P6PZ3	Measure pore water pressure
Shape Array Accelerometer	13	Inside flange	1 string with sensors spaced at approximately 1.6 feet; SAA L6 attached to pile before installation	Horizontal movement of wall
Resistance strain Gauges	8	Inside flange	Installed in pairs; P6SG1 & 2, P6SG3 & 4, P6SG5 & 6, P6SG7 & 8	Strain at several depths
Solar powered data logger	1	Mounted on wall	Mounting frame and protection	Data collection and comm
Ground settlement monument	2	2 at ground surface behind wall, 2 prisms on off ramp wall	Installed (post construction), baseline reading collected, was not able to monitor due to construction activities and interferences	Settlement - Information not collected

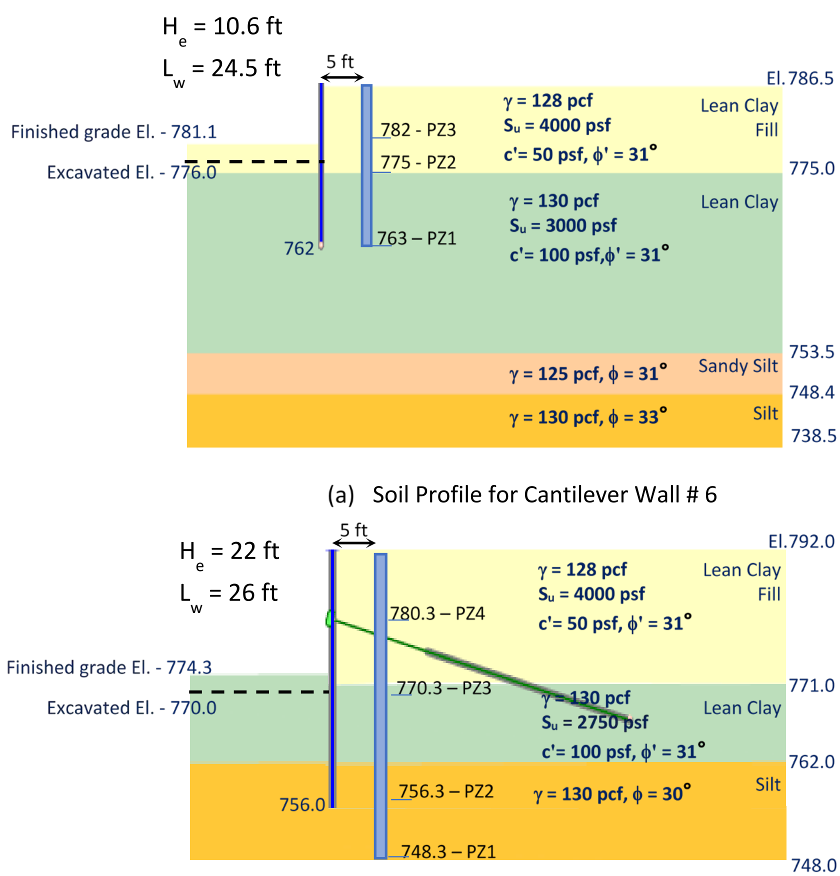
Table 4-2 Anchored Soldier Pile Instrumentation Installation Summary, Pile #36

Instrument	Quantity	Location	Installation Note	Purpose
Instrumented Lock- off nut	1	On tieback	Installed by contractor	Measure load on tieback
Vibrating Wire Piezometer	4	Behind wall	1 string of sensors installed in borehole 5 feet behind the wall; P36PZ1, P36PZ2, P36PZ3, P36PZ4	Measure pore water pressure
Shape Array Accelerometer	22	Inside flange	1 string with sensors spaced at approximately 1.6 feet.; SAA L36 attached to pile before installation	Horizontal movements of wall
Resistance strain gauges	12	Inside flange	Installed in pairs; P36SG1 & 2, P36SG3 & 4, P36SG5 & 6, P36SG7 & 8, P36SG9 & 10, P36SG11 & 12	Strain at several depths
Solar powered data logger	1	Mounted on wall	Mounting frame and protection	Data collection and comm
Ground settlement monuments	5	2 ground behind wall, 1 prism on sound wall, 2 prisms on off ramp wall	Installed (post construction), baseline reading collected, was not able to monitor due to construction activities and interferences	Settlement - Information not collected

Sensor Installation

Geocomp subcontracted a driller to install one borehole at each wall location (2 total). Soil samples were obtained. Grouted-in-place vibrating wire pressure transducers were installed to monitor pore water pressures at specific depths. The soil samples collected in the field were transported to GeoTesting Express Inc., in Acton Massachusetts for laboratory testing. Results of the laboratory tests are included in Appendix E.

The soil profile consists of lean stiff clay below the topsoil to approximately 32 feet and then sandy silt and silt to the bottom of each borehole. Figure 4-2 below shows the generalized soil profile adopted by the research team. The soil properties used in the research team's analyses (and noted on the profile) were based on the geotechnical report by the Forward 45 design team (Geotechnical Exploration and Foundation Evaluation Report Zoo Interchange Corridor Study, 2015). It should be noted that the research team did no work to independently assess these soil parameters.



(a) Soil Profile for Cantilever Wall # 6

(b) Soil Profile for Anchored Wall # 36

Figure 4-2 Generalized soil profile used by the research team

The Shape Array Accelerometers (SAA) and strain gauges were attached/welded to the inside flange of the piles in the contractor's yard by Geocomp. Cover plates were welded on to protect the instruments. The construction contractor then transported and placed the two instrumented piles under Geocomp's supervision. Care was taken when placing the instrumented piles and concrete to avoid damaging the instruments. The general layout where the SAA is installed and welded to the steel pile is shown in Figure 4-4. The strain gauges were also protected by steel cover plates. If welded through the entire length of the pile the SAA steel pipe and strain gage steel cover plates would increase the bending stiffness of the Pile by about 5%. To minimize the stiffening the welds for the pipe and plates are less than 1 inch long and spaced approximately 2 feet to 4 feet apart as shown in Figure 4-3.

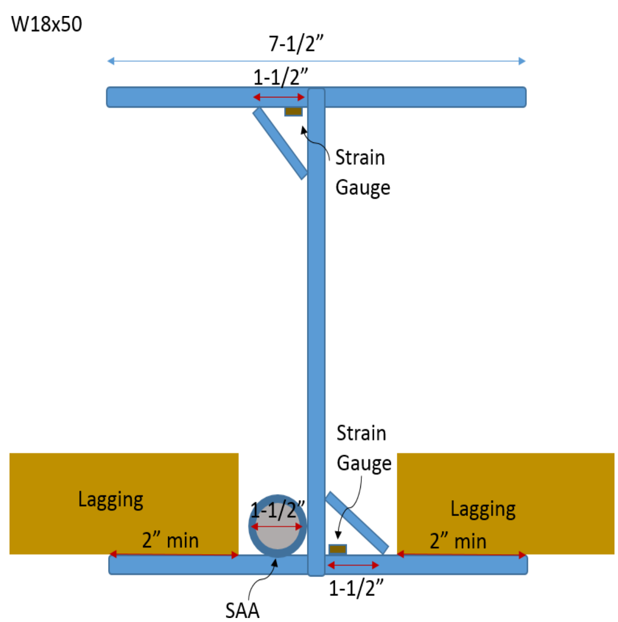


Figure 4-3 Flange Instrumentation Layout Detail for W18x50

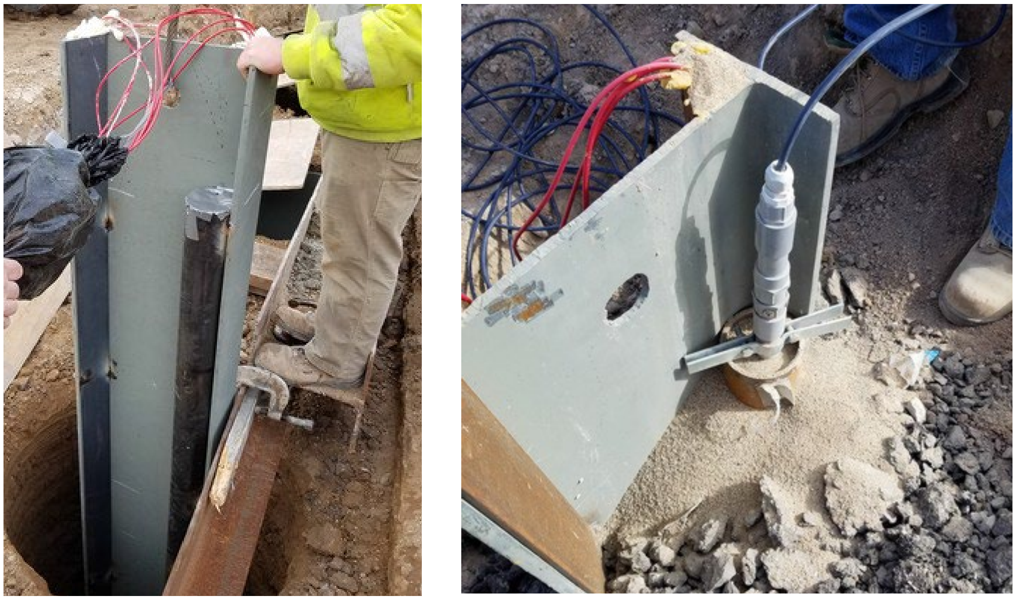


Figure 4-4 Picture of SAA placed and secured at ground surface

Originally, the research team planned to have the contractor place and install a load cell on the tieback for Pile 36 under its supervision. Due to issues with spacing, this configuration was not possible. Instead, the lock-off nut was instrumented with strain gages. These strain gages appear to have been damaged during the concrete placement for the CIP retaining wall face and construction/long term readings were not possible.

Geocomp installed two (2) data loggers and collected data form the instruments through August 2019 using our *i*SiteCentral™ system. *i*SiteCentral is a robust, scalable, cloud-based automated data collection and management system used to continuously monitor the performance of infrastructure projects in real-time. Data were logged each hour (to determine temporal effects) from prior to excavation through the first year of service life. A total of approximately 11,300 readings on each sensor were obtained during the monitoring period. Photos of the instrument installations are provided in Appendix D.

Geocomp provided WisDOT with periodic instrumentation and monitoring reports through the active construction and post construction periods and all data were continually available on the *i*SiteCentral web site. Appendix D provides a data report in the form of graphs of all readings of the instrumentation from April 2018 through August 2019. Appendix D also includes the detailed Instrumentation Data Report produced in April 2019 summarizing instrument performance as discussed in a meeting between the research team and WisDOT on April 10, 2019.

Data collection was stopped in August 2019. At the time of this report, all equipment remains in place and data collection could be restarted if WisDOT chooses to do so in order to collect additional data regarding seasonal effects on the instrumentation and changes in pore pressures behind the wall over time. An extended monitoring period could provide a better understanding on the development of long-term deformations.

Task 4: LiDAR Survey

The research team had originally planned to use WISDOT's LiDAR equipment to perform a LiDAR survey using field staff from the research team's (ARA) Wisconsin office. Once the project began, the WISDOT LiDAR group notified the research team that the equipment could not be loaned out and that the WisDOT team needed to perform the LiDAR survey. Coordination was on-going for several months between the research team and WISDOT, but due to coordination issues with acquiring survey control, the LiDAR survey was not able to be performed during construction and was not done for this project.

Task 5: Data Analysis

The research team conducted a detailed data analysis which is presented in section 5 of this report. The Analysis section includes a brief description of the original wall design by the Forward 45 design team, and then catalogs and describes the analyses made by the research team for the two instrumented walls.

Task 6: Final Report

The research team produced this final report detailing and summarizing the work performed for this research project. As part of the contract deliverables, Dr. W. Allen Marr of Geocomp presented the research findings to WisDOT on December 9, 2019.

This section catalogs and describes the analyses made for the two walls. It gives a brief explanation of the original wall design method by the Forward 45 design team (who performed the original wall design). Then, the following section entitled "Results" presents the results from the analyses compared with measurements from the field instrumentation.

5. ANALYSIS

The Forward 45 team (design team) designed the walls based on analysis using drained soil parameters following WisDOT Bridge Manual. Earth pressures calculated by the design team included factored surcharge loads including live load and wind load due to the noise barrier wall considering the worst-case scenario for design purposes. The design team did not consider any pore pressures in the soil since there was no indication of water during their subsurface exploration program. Calculations of lateral earth pressures used by the design team as well as a table of results from their PY-Wall analysis are included in Appendix F. Analyses conducted by the research team considered both short- and long-term conditions as well as the effect of pore pressure on the long-term stability of the walls. These conditions are detailed in the following section.

5.1 COMPUTER ANALYSES

Two computer programs SPW-911 and PY-Wall, were used by the research team to predict the wall performance. The research team conducted analyses for the following conditions:

(1) Without hydrostatic pore pressures on both walls under long term drained conditions which is the condition used by design team – referred in the text as <u>drained with no pressures.</u>

(2) With hydrostatic pore pressures on both walls under long term drained conditions with pore pressures applied on the wall – referred to in the text as <u>drained with hydrostatic pore pressures.</u>

(3) With measured pore pressures on both walls under long term drained conditions with measured pore pressures applied on the wall– referred in the text as <u>drained with measured pore pressures.</u>

(4) With hydrostatic pore pressures on both walls under short term undrained conditions– referred to in the text as <u>undrained with hydrostatic pore pressures.</u>

Incorporation of pore pressures in computations was crucial for the analyses conducted by the research team. Measured pore pressures were compared with hydrostatic pressures as shown in Figure 5-1. Measured pore pressures presented in Figure 5-1 are maximum pore pressures measured during the one-year period of monitoring. For the cantilever wall the measured pressures were a little lower than the hydrostatic pressures. For the anchored wall, measured pore pressures were hydrostatic for about 12 feet below grade. Then, below 12 feet lower than hydrostatic pressures were measured.

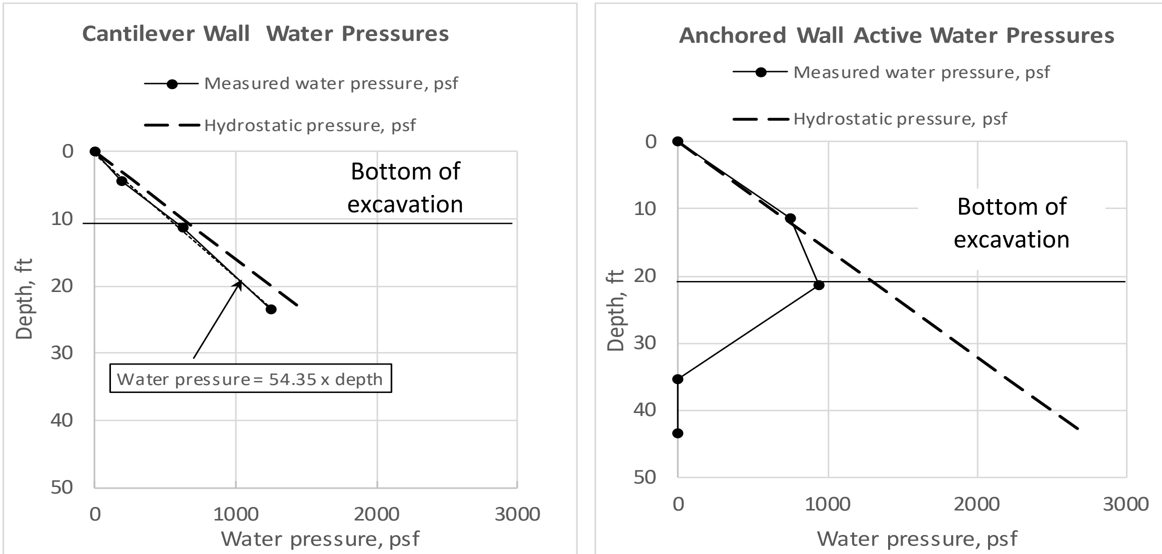


Figure 5-1 Measured pore pressures compared with hydrostatic pressures

5.1.1. Analysis Using SPW-911

SPW-911 is a design and analysis tool for cantilevered and propped sheet pile and soldier pile walls using limit equilibrium theory. Different pressure models (Rankine, Coulomb, Terzaghi and Peck, etc.), different sheet pile penetration models (free, fixed earth, etc.), and different factor of safety calculations (net/gross pressure) can be used in this program. Multi-layered excavations may be defined with different soils on each side of the excavation (Pile Buck Sheet Piling Design Manual, 2007).

The research team performed analyses with hydrostatic pressures behind the wall (water level at top of retained soil) as well as with maximum measured pore pressures. At the cantilever wall, measured pore pressures were approximately 87% of full hydrostatic (using water level at top of retained soil). Measured pore pressures at the anchored wall were approximately hydrostatic for the top half of the wall and lower than hydrostatic for the bottom half of the wall. This geometry could not be modeled by SPW-911 program due to program's limitations. Therefore, the anchored wall analysis with SPW-911 program was conducted considering hydrostatic pore pressures only.

Additional assumptions used in the analyses of the instrumented soldier pile walls using SPW-911 (Soldier Pile Design Using SPW-911, 2003) are as follows:

- No surcharge load was included.
- Earth pressures were not factored. The earth pressures were determined following AASHTO Service I Limit State.

- The soldier piles resist all the force and moment within the design section (assume the lagging transfers all force and moment to the soldier piles).
- Beneath the excavation level, earth pressures (active and passive) act on the soldier piles (lagging is not present below this level).
- Drilled holes for soldier piles are backfilled with concrete.
- The effective width, W_{eff} of the pile is assumed to be the diameter of the concrete socket. The diameter of the concrete socket is 3' for cantilever wall and 2.5' for anchored wall.
- Cohesion values are reduced below the excavation level for discrete soldier pile elements based on the ratio of the effective width of the pile to the pile spacing, f, which is 0.4 for cantilever wall and 0.33 for anchored wall.
- The Rankine pressure model was used for the cantilever wall and anchored wall, and the Terzaghi pressure model was used for the anchored wall. Figure 5-2 shows the empirical pressure model developed by Terzaghi and Peck for different types of soils. The pressure model for stiff clay presented in Figure 5-2 is used for the achored wall.

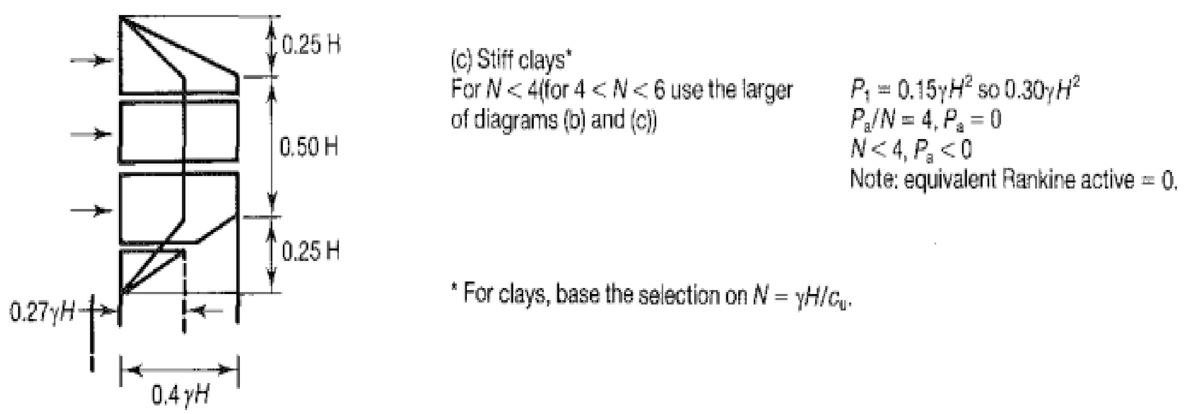
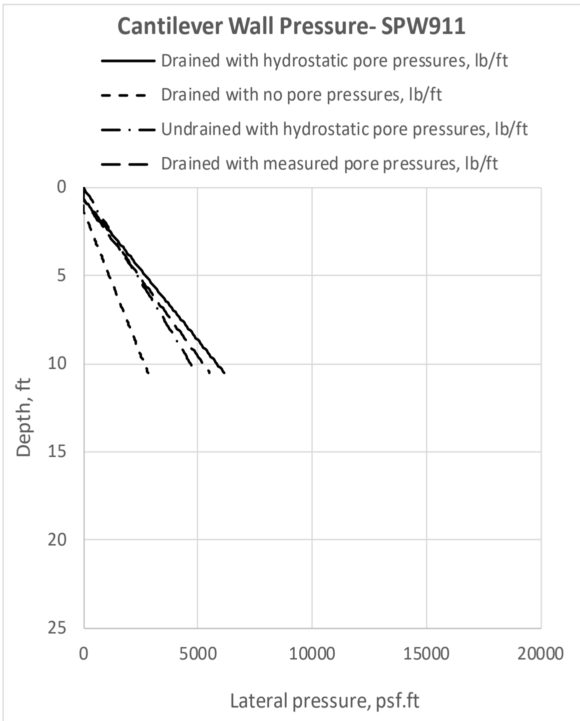


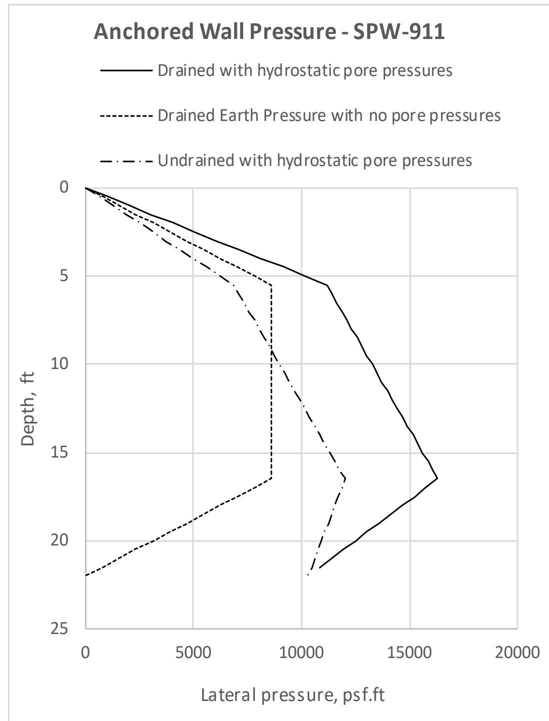
Figure 5-2 Pressure envelope method – Terzaghi and Peck (after Williams and Waite, 1993, "The Design and Construction of Sheet-Piled Cofferdams")

Earth pressures were calculated for the four different conditions listed in Section 5. Earth pressures used in SPW-911 program for the cantilever and anchored walls are shown in Figure 5-3 below. Please note the earth pressures for the anchored wall does not include undrained conditions with measured pore pressures due to a software limitation as mentioned earlier.

For designing soldier piles in SPW-911, the effective width concept to compute active and passive earth pressures on the soldier pile wall below the excavation level is suggested in the program manual (Soldier Pile Design Using SPW-911, 2003). When concrete was used to backfill the drilled holes, the effective width of the pile used was the diameter of the drilled hole. The SPW-911 program is written for designing continuous walls. In order to design soldier pile walls in this program with an effective width of 1xPile width, earth pressure coefficients or the cohesion values (undrained shear strength) below the excavation line were modified with a ratio of the effective width of the pile to the pile spacing.



(a)



(b)

Figure 5-3 Earth pressures used in SPW-911 analyses (a) Cantilever Wall - Drained with no pore pressure, lb/ft (b) Anchored Wall - Drained Earth Pressure with no pore pressures

This ratio represents a reduction relative to a continuous wall. In the analysis conducted by the research team, cohesion values below the excavation line were modified using this ratio. The soil properties used in the research team's analyses were based on the geotechnical report by the Forward 45 design team (Geotechnical Exploration and Foundation Evaluation Report Zoo Interchange Corridor Study, 2015). The soil properties used above and below the excavation line are shown in Table 5-1 and Table 5-2 for cantilever and anchored walls, respectively.

Table 5-1 Input Parameters - Soil properties used in SPW 911 analyses for cantilever wall

Soil Layer	Top El. (ft.)	Bottom El. (ft.)	γ (pcf)	ϕ (°)	S_u (psf)	c' (psf)	f	S_u modified* (psf)	K_a	K_p	K_a*	K_p*
Lean Clay Fill (above excavation)	786.5	778.0	128	0	4000	50	0.4	-	1	1	-	-
Lean Clay Fill (below excavation)	778.0	775.0	128	0	4000	50	0.4	1600	1	1	-	-
Lean Clay	775.0	753.5	130	0	3000	100	0.4	1200	1	1	-	-
Sandy Silt	753.5	748.4	125	31	0	0	0.4	-	0.33	3.00	0.13*	1.2*

Note: *Modified values of cohesion or earth pressure coefficient used in SPW-911 analysis

Table 5-2 Input Parameters - Soil properties used in SPW 911 analyses for anchored wall

Soil Layer	Top El. (ft.)	Bottom El. (ft.)	γ (pcf)	ϕ (°)	S_u (psf)	c' (psf)	f	S_u modified* (psf)	K_a	K_p	K_a*	K_p*
Lean Clay Fill (above excavation)	792.0	774.3	128	0	4000	50	0.33	-	1	1	-	-
Lean Clay (above excavation)	774.3	771.0	130	0	2750	100	0.33	-	1	1	-	-
Lean Clay (below excavation)	771.0	762.0	130	0	2750	100	0.33	917*	1	1	-	-
Silt	762.0	753.4	130	30	0	0	0.33	-	0.33	3.0	0.11*	1.0*

Note: *Modified values of cohesion or earth pressure coefficient used in SPW-911 analysis

Wall Properties

Table 5-3 presents wall dimensions and parameters used in the SPW-911 analyses.

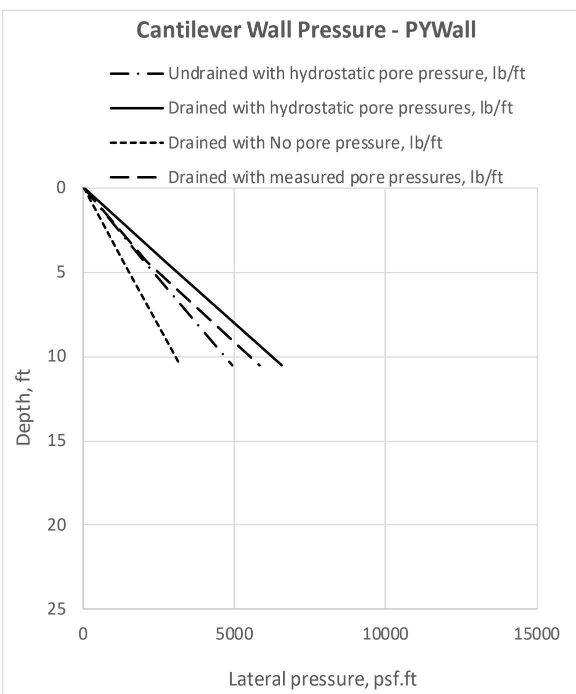
Table 5-3 Input Parameters - Wall dimensions and properties used in SPW 911 analyses

Soldier Pile	Cantilever Wall (W18X50)	Anchored Wall (HP14X73)
Pile Spacing, ft	7.5	7.5
Effective width, ft	3.0	2.5
Moment of inertia per foot, I (in^4/ft)	106.67	97.20
Elastic modulus, E (psi)	29,000,000	29,000,000
Section modulus per foot, Z (in^3/ft)	11.85	14.27
Working stress of steel, f (psi)	33,500	33,500
Pile length, (ft)	24.5	36.0
Maximum bending moment (lb.ft/ft)	33090	39837
Depth of embedment, (ft)	14.0	14.0

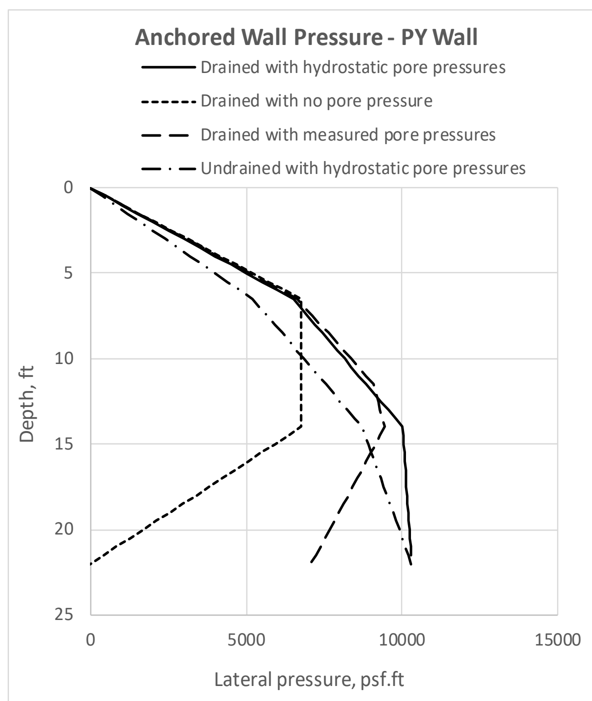
5.1.2 Analysis Using PY-WALL

This section gives details of the input soil and wall parameters and different options used in PY-WALL program. PY-WALL program uses the subgrade reaction method to model the deformations and bending of the wall with the soil represented as non-linear springs. The subgrade reaction method can be improved if the soil reaction represented by linear springs is based on in-situ test results.

The PY-WALL program enables users to define soil types and assign any of the different earth pressures which are internally generated, and user specified. The research team used the program option for user specified earth pressures which allowed direct data entry of total pore pressures with depth. This option allowed direct entry of the measured pore pressures in PY-Wall program. The earth pressures were determined following AASHTO Service I Limit State. Earth pressures for a cantilever wall was taken as a triangular distribution (Figure 3.11.5.6-5 in AASHTO LRFD Bridge Design Specifications 8th Edition- Section 3.11.5.6). Earth pressures for an anchored wall was taken as a trapezoidal distribution (Figure 3.11.5.7.2b-1 in AASHTO LRFD Bridge Design Specifications 8th Edition- Section 3.11.5.7).



(a)



(b)

Figure 5-4 Earth pressures used in PY-Wall analyses (a) Cantilever Wall - Undrained with hydrostatic pore pressure, lb/ft (b) Anchored Wall - Undrained with hydrostatic pore pressure

The following additional assumptions were made in PY-WALL analyses conducted by the research team:

- No surcharge load was included.
- The clay layers are modeled as stiff clay with undrained shear strength for short term undrained analyses and modeled as silt layer with c -ϕ for long term drained analyses (PY-WALL user's manual (Section 3.2.5 in PY-WALL User's Manual 2015).
- The silt layer is modeled as c – ϕ soil as described in PY-WALL user's manual (Section 3.2.5 in PY-WALL User's Manual 2015).
- Values of strain at 50% of the maximum stress, ε_{50}, for soil layers were determined from PY-WALL's user manual (Tables 3.3 in PY-WALL User's Manual 2015).

Constant modulus values, k(py) for soil layers were determined from PY-WALL's user manual (Tables 3.4 in PY-WALL User's Manual 2015).

- The soil properties used in the PY-WALL analysis are shown in Table 5-4 and Table 5-5 below for cantilever and anchored walls, respectively.

Table 5-4 Input Parameters - Soil properties used in PY-WALL analyses for cantilever wall

Soil Layer	Top El. (ft.)	Bottom El. (ft.)	γ (pcf)	ϕ (°)	S_u (psf)	c' (psf)	ε_{50}	k(py) (pci)
Lean Clay Fill	786.5	778.0	128	0	4000	50	0.005	1000
Lean Clay Fill	778.0	775.0	128	0	4000	50	0.005	1000
Lean Clay	775.0	753.5	130	0	3000	100	0.005	1000

Table 5-5 Input Parameters - Soil properties used in PY-WALL analyses for anchored wall

Soil Layer	Top El. (ft.)	Bottom El. (ft.)	γ (pcf)	ϕ (°)	S_u (psf)	c' (psf)	ε_{50}	k(py) (pci)
Lean Clay Fill	792.0	774.3	128	0	4000	50	0.005	1000
Lean Clay	774.3	771.0	130	0	2750	100	0.006	750
Lean Clay	771.0	762.0	130	0	2750	100	0.006	750
Silt	762.0	753.4	130	30	0	0	0.007	500

p-y curves

PY-WALL enables the use of *p-y* curves both internally generated by the program and user specified. Analyses by the research team were conducted using both options. When the option of internally generated *p-y* curves is selected, the program automatically generates *p-y* curves based on input parameters of initial modulus, k(py), and strain at one-half of the maximum stress, ε_{50} and embedded soil types in PY WALL (Table 5-4 and Table 5-5). Figure 5-5 shows the *p-y* curves internally generated by PY-WALL program for both walls.

User specified *p-y* curves were based on in-situ pressure meter test (PMT) data conducted at Boreholes CR-40, CR41a, and CR-44 provided to Geocomp by WisDOT. Figure 5-6 presents the user specified *p-y* curves used in the PY-WALL analyses. The original in-situ pressuremeter data supplied to Geocomp is included in Appendix G. Incorporation of in-situ pressuremeter test data improved the estimations of displacement and moments in PY-WALL. Results are given in Section 6.

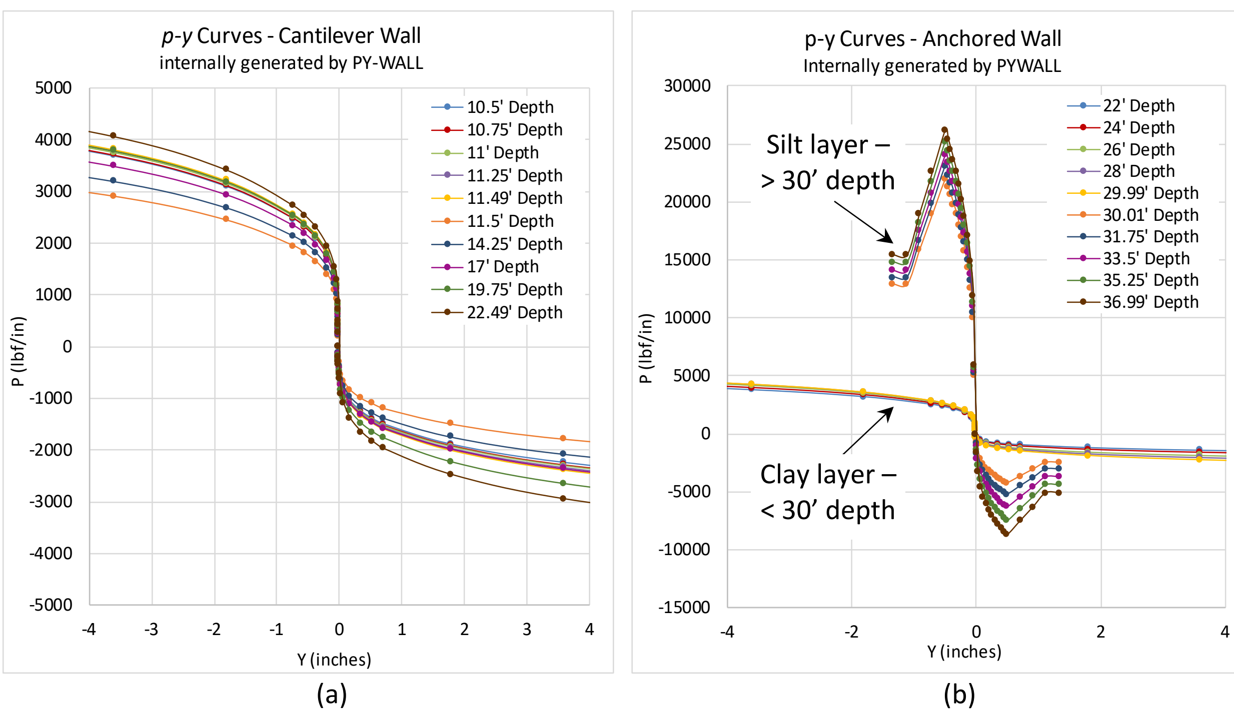


Figure 5-5 Input Parameters - Internally generated *p-y* curves used in PY-WALL analyses (a) Cantilever wall (b) Anchored wall

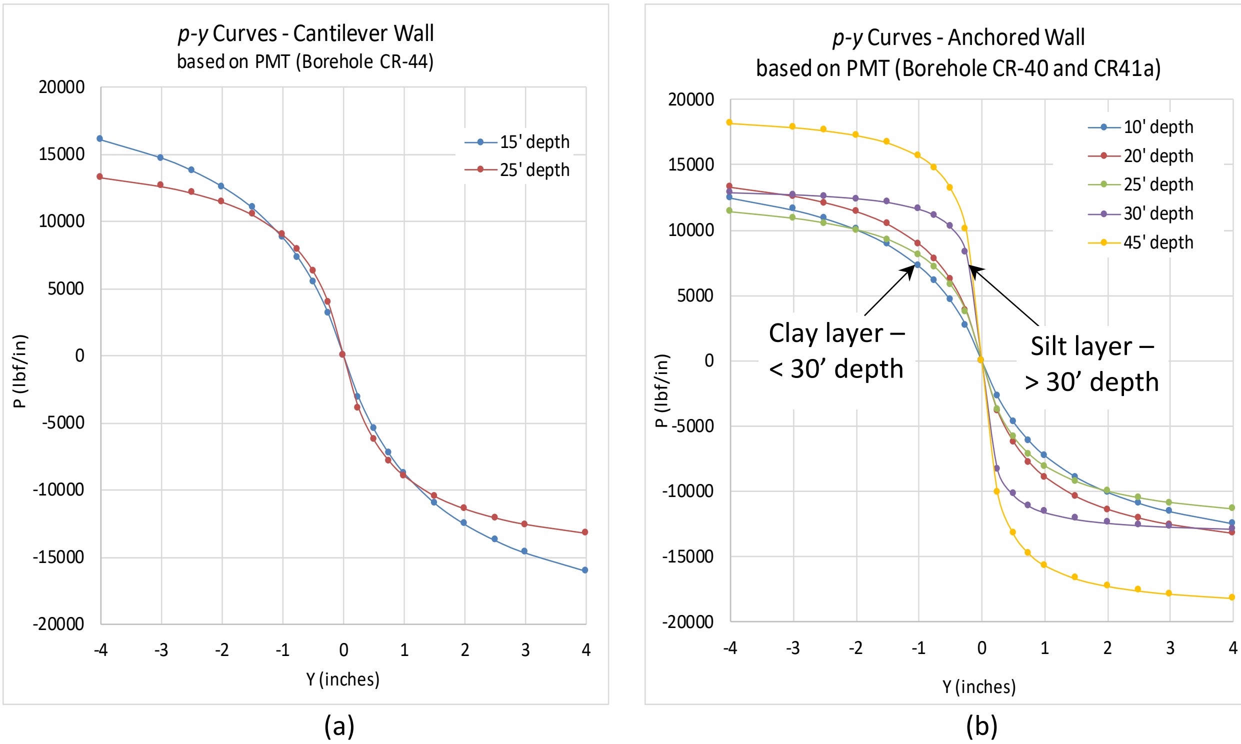


Figure 5-6 Input Parameters - *p-y* curves determined based on PMT used in PY-WALL analyses (a) Cantilever wall (b) Anchored wall

Wall Properties in PY-WALL

PY-WALL enables assignment of pile stiffness parameters which can be variable with depth. Soldier piles were assigned separate flexural stiffnesses for the steel section and the concrete embedded section. Three different approaches were used by the research team to estimate the flexural stiffness of the composite section of the pile. In the first approach, the flexural stiffness of the pile was assumed to be the stiffness of steel. In the second approach, called "full composite flexural stiffness", the flexural stiffness of concrete and steel were superimposed. In the third approach, in order to model the disengagement of concrete stiffness with cracks due to wall movement, half of the fully superimposed composite flexural stiffness was used. The three different approaches for the flexural stiffnesses used are as shown below:

(1) Flexural stiffness = E_sI_s

(2) Full composite flexural stiffness = $E_sI_s + E_cI_c$

(3) Half of full composite flexural stiffness = $0.5*(E_sI_s + E_cI_c)$

Table 5-6 shows the wall dimensions and parameters used in PY-WALL analyses including flexural stiffnesses determined with the three different approaches mentioned above.

Table 5-6 Input Parameters - Wall dimensions and properties used in PY-WALL analyses

Soldier Pile	Cantilever Wall W18X50			Anchored Wall HP14X73		
Flexural stiffness, EI (lb-in^2)	(1) $2.32\text{x}10^{10}$	(2) $2.8\text{x}10^{11}$	(3) $1.4\text{x}10^{11}$	(1) $2.1\text{x}10^{10}$	(2) $1.46\text{x}10^{11}$	(3) $7.32\text{x}10^{10}$
Pile spacing, ft	7.5			7.5		
Width of the pile, ft	3.0			2.5		
Free height of the wall, ft	7.5			22		
	10.5					
Unbonded length of anchor, ft	---			15		
Angle of anchor (degrees)	---			20		
Anchor spacing, ft	---			7.5		
Tie-back stiffness, lb/in	---			$3\text{x}10^6$		

6. RESULTS

This section contains results including calculated displacement and moments from computer analyses under four conditions discussed in the previous section. Calculated results are followed by the measured displacements, moments, and pore pressures from instrumentation data collected during construction and the 1-year monitoring period at the instrumented walls. Displacements and moments that were computed by the research team and Forward 45 design team were then compared with measured displacements and moments. Apparent earth pressures computed from the measurements are compared with those implicit to the analysis methods. The results section is separated into two sections to present results from both wall types in a similar format.

6.1. CALCULATED AND MEASURED RESULTS FOR THE CANTILEVER WALL AT PILE #6

6.1.1.1. Calculated Results for Cantilever Wall

This section presents displacements and moments for the cantilever wall at pile #6. It includes results computed by the Forward 45 design team who used long term drained conditions with no water pressures on the wall. Design calculations conducted by the Forward 45 design team were provided to Geocomp by WisDOT and are included in Appendix F. This section also presents computed displacements and moment values by the research team using SPW-911 and PY-WALL under four different conditions; undrained with pore pressures, drained with hydrostatic pore pressures, drained with measured pore pressures, and drained with no pressures. Outputs from the analyses performed by the research team are included in Appendix H.

Computed Displacements

The Forward 45 design team calculated a maximum displacement of 0.81 inches for the Service I Limit State using the PY-WALL program. It should be noted that the pile length and maximum exposed height of wall that were used by the Forward 45 design team are 2 feet longer than as built.

Deformation calculations by the research team are shown in Table 6-1. Maximum displacements were computed at the top of the pile. Analyses were done for two stages of excavation which was 7.5 and 10.5 feet. Finished grade is approximately 5.5 feet from the top of the pile. The results for corresponding depths of excavation are presented in Table 6-1. Displacements computed for drained analyses with hydrostatic pore pressure were the largest of those calculated. The computed factor of safety for this condition was less than 1 by both methods indicating the possibility of instability of the wall if full hydrostatic conditions developed in the retained soil. Displacements computed with user-defined *p-y* curves determined from in-situ pressuremeter tests were more realistic compared to the displacements computed by SPW-911 and internally generated *p-y* curves based on selected soil parameters. The effect of pore pressures on the computed displacements are significant. Computed values are compared with measured displacements in the following section entitled “Comparison of Results”.

Table 6-1 Calculated displacement results for Cantilever Wall #6

Computed Displacements (in) – Cantilever Wall #6							
	Analysis Assumptions			Analysis Results			
Software Program	Flexural stiffness	*p-y* curves	Excavation Depth, ft	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]
SPW-911	E_sI_s	-	7.5	0.10	0.70	0.50	0.10
			10.5	0.60	2.60[(1)]	2.10	0.40
PY-Wall	E_sI_s	Int. generated	7.5	0.19	0.80	0.59	0.27
			10.5	1.00	13.00[(1)]	4.50	1.10
		User-defined	7.5	0.68	0.90	0.74	0.44
			10.5	0.80	1.10	0.88	0.53

Notes: [1]F.S < 1.0, the wall is not stable.

Additional analyses were conducted using the composite flexural stiffness to investigate its effect on the calculated displacements. Calculated deformations by the research team using PY-WALL program under undrained conditions with water to top of wall on the retained side and water at the bottom of excavation on the cut side of the wall are shown in Table 6-2. Table 6-2 also shows the computed displacements with varied flexural stiffness of the pile elements. Like Table 6-1, the results are presented for two stages of excavation, 7.5 and 10.5 feet. Finished grade is approximately 5.5 feet from the top of the pile.

Table 6-2 Computed Displacements with different flexural stiffness – Cantilever Wall #6

Computed Displacements with different flexural stiffness (in) (Undrained with pore pressures) – Cantilever Wall #6				
Analysis Tool	Analysis Assumptions		Excavation Depth, ft	Displacement, in
	Flexural stiffness	*p-y* curves		
PY-Wall	E_sI_s	Internally generated	7.5	0.19
			10.5	0.99
		User-defined	7.5	0.68
			10.5	0.80
	$E_sI_s+ E_cI_c$	Internally generated	7.5	0.10
			10.5	0.34
		User-defined	7.5	0.16
			10.5	0.34
	$0.5*(E_sI_s+ E_cI_c)$	Internally generated	7.5	0.11
			10.5	0.40
		User-defined	7.5	0.17
			10.5	0.39

Computed Bending Moments

The Forward 45 design team calculated a maximum bending moment of 85 kip-ft for the Strength I Limit State using the PY-WALL program. Forward 45 design maximum moment was obtained directly from the design documents. The research team could not reproduce this value with the PY-WALL software. We suspect a typographical error in the reported value. It should be noted that the pile length and maximum exposed height of the pile used by the design team in their design is 2 feet longer than as built.

Maximum moments calculated by the research team under the four conditions using the two programs are shown in Table 6-3. Like the displacement results, maximum moments calculated under drained conditions with hydrostatic pore pressures were the largest. The effect of pore pressures on the computed maximum moments were prominent.

Table 6-3 Calculated maximum moments for Cantilever Wall #6

Computed Maximum Moments (kip-ft)– Cantilever Wall							
Software Program	Analysis Assumptions			Analysis Results			
	Flexural stiffness	*p-y* curves	Excavation Depth, ft	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]
SPW-911	E_sI_s	-	7.5	37	103	81	47
			10.5	109	289(1)	235	68
PY-Wall	E_sI_s	Int. generated	7.5	46	117	90	48
			10.5	123	292(1)	233	122
		User-defined	7.5	92	123	100	60
			10.5	108	146	118	71

Notes: [1]F.S < 1.0, the wall is not stable.

Results from the analyses that were conducted using composite flexural stiffness to investigate its effect on the calculated moments are shown in Table 6-4. Maximum moments computed with user-defined *p-y* curves determined from in-situ pressuremeter tests were more realistic compared to the displacements computed by SPW-911 and internally generated *p-y* curves based on selected soil parameters. The composite stiffness of the wall has less effect on the computed maximum moment than on the computed maximum displacement.

Table 6-4 Calculated maximum moments results for Cantilever Wall #6

Computed Maximum Moments with different flexural stiffness (kip-ft) (Undrained with pore pressures) - Cantilever Wall #6				
Analysis Tool	Analysis Assumptions		Excavation Depth, ft	Maximum Moment, kip-ft
	Flexural stiffness	*p-y* curves		
PY-Wall	E_sI_s	Internally generated	7.5	46
			10.5	123
		User-defined	7.5	92
			10.5	108
	$E_sI_s + E_cI_c$	Internally generated	7.5	53.3
			10.5	130
		User-defined	7.5	54.2
			10.5	120
	$0.5*(E_sI_s + E_cI_c)$	Internally generated	7.5	51.3
			10.5	130
		User-defined	7.5	51.7
			10.5	118

6.1.2. Measured Results for Cantilevered Wall

This section presents summaries of measured displacements, moments, and pore pressures from instrumentation data collected throughout the construction and the 1-year monitoring period at the instrumented cantilever walls. Appendix D provides time history graphs of the measurements. Data was collected from the time period starting in April 2018 and ending in August of 2019. Automated instrumentation was installed and consisted of Shape Accelerometer Arrays (SAA), strain gages, and piezometers. These instruments were connected to Geocomp's *i*SiteCentral automated data collection and management system and were read hourly throughout the monitoring period. Data was also collected from the General Mitchell Airport detailing a time history of rainfall and snowfall amounts during the time period. Weather data such as this can be helpful in determining seasonal and storm related effects on the instrumentation.

Displacement Measurements:

Only measurements of wall displacement were obtained during the monitoring period. The system to measure displacements of the ground surface behind the wall did not work as anticipated prior to the start of construction (as explained in Section 4 of this report). Wall displacements were measured with SAA inclinometers fastened to the inside of the soldier pile flange. The summaries of measurements on representative dates for the cantilevered wall are included in Appendix D. We make the following conclusions from these measurements:

- Deformation profiles obtained at the cantilever wall indicate that the as-built pile embedment achieved fixity at the pile toe.
- A change in the slope of the deformation profile of the wall is observed at about the elevation of the top of the concrete footing (El. 778 feet)
- After approximately 16 months of monitoring, the maximum measured deformation was 0.33 inches and occurred at the top of wall.
- Approximately 0.20 inches of total deformation was measured between the start of excavation and the completion of the wall construction.
- Some of the deformation measured within the first 2 weeks (after the first excavation cut was complete) appears to be related to the increase in water pressure within the retained soil behind the wall.

- Approximately 0.10 inches of additional deformation was measured from the end of August 2018 to the beginning of August 2019.
- Between the fall of 2018 and the beginning of August 2019 increases in pore pressure within the retained soil may have contributed to the measured deformations.
- It is not clear if the placement of the anchor slab and the noise barrier wall had an effect on the measured deformations. The research team was not provided with installation dates by the contractor and could not determine exactly when the anchor wall or noise barrier were placed.

Bending Moment Measurements:

- The curvature of the beam deflection is directly proportional to the bending moment. For a purely cantilever beam the bending moment increase from zero at the free end to a maximum at the point of fixity. For a cantilever retaining wall the point of fixity is located somewhere in the embedded depth below the bottom of the excavation. Indeed, the bending moment distribution obtained from strain gages and SAA displacement profiles indicate that maximum moment developed below the bottom of the excavation level.
- However, both the strain and SAA computed bending moments show a reversal in the bending moment from positive to negative before reducing to zero at the top of the wall Figure 6-1. The implication is that near the excavation level the beam is bending towards the excavation while at the top the beam is bending away from the excavation. The only way that this could occur is if something is reacting against the wall to hold the top of the wall back. Closer inspection of the as built conditions shows that a potential cause to be a concrete slab on top of the wall supporting a barrier. The slab is providing restraint to the top of the wall.
- The inferred maximum bending moment from the SAA displacement profile was calculated at approximately 100 kip-ft, located between the finished grade and top of footing, approximately 1.5 feet below the finished grade.
- The maximum bending moment measured by strain gages is about 30 kip-ft, located near the finished grade (approximately 0.5 feet below the finished grade), but given the wide spacing of the strain gages this may not have been the maximum moment in the pile.
- As shown in Figure 6-1, the difference in maximum moment magnitude between strain gages and SAA horizontal displacement measurements is likely due to the locations of the measurements. The more refined spacing available from the SAA data enabled measurements closer to the

location of the maximum bending moment as compared to the widely spaced strain gages located along the pile which may not have captured the maximum moment.

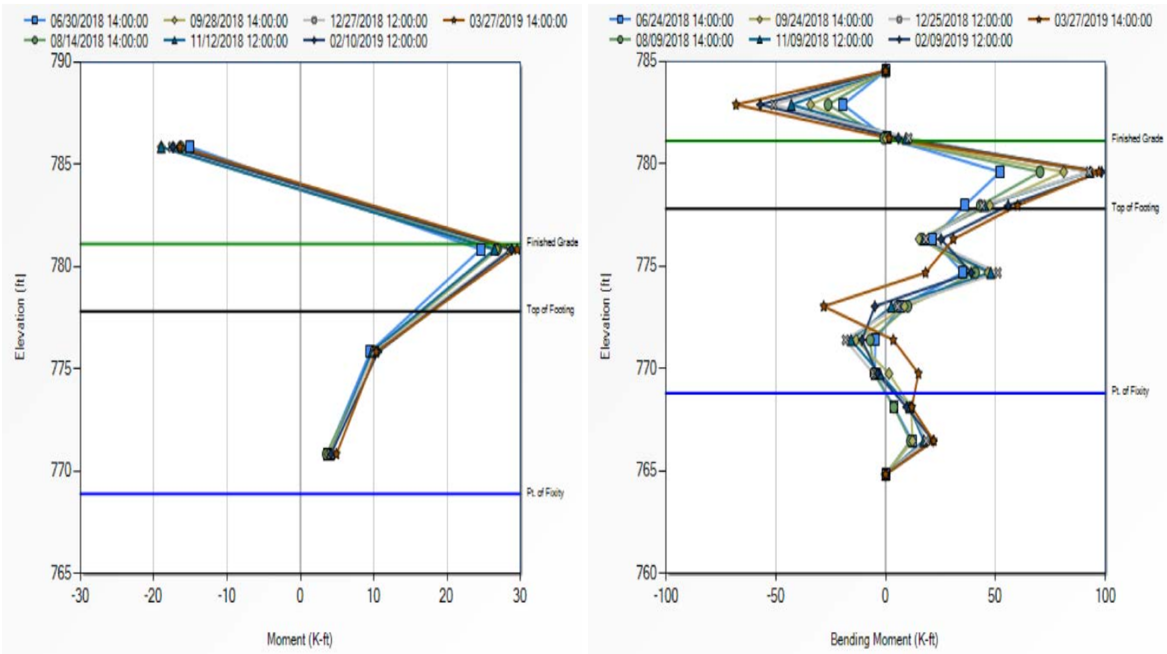


Figure 6-1 Bending moment profiles (a) measured by strain gages (b) inferred from SAA measurements

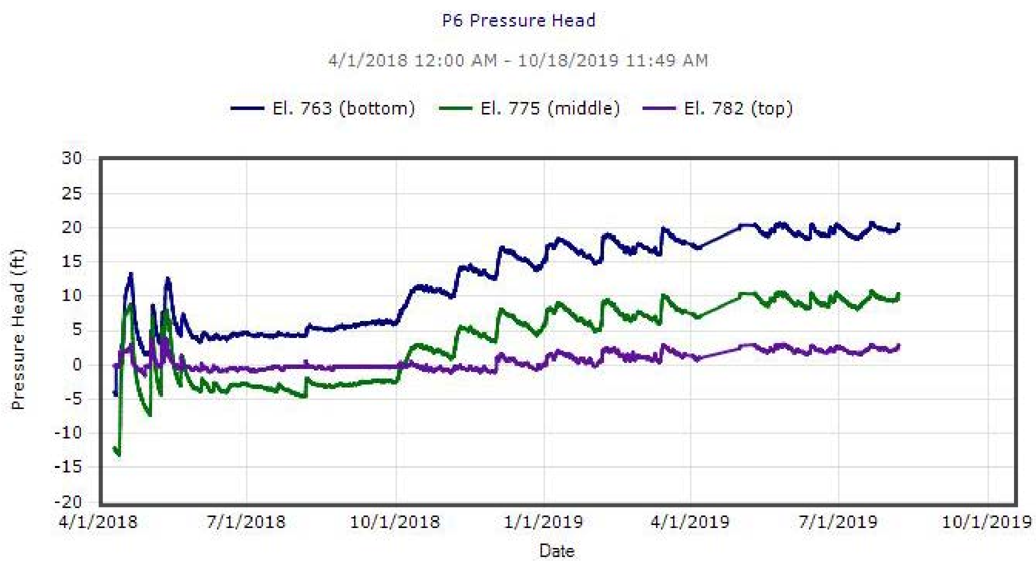
Table 6-5 summarizes the measured displacement and bending moment for the cantilever wall at Pile #6.

Table 6-5 Measured displacement and bending moment from instrumentation data for Cantilever Wall at Pile #6

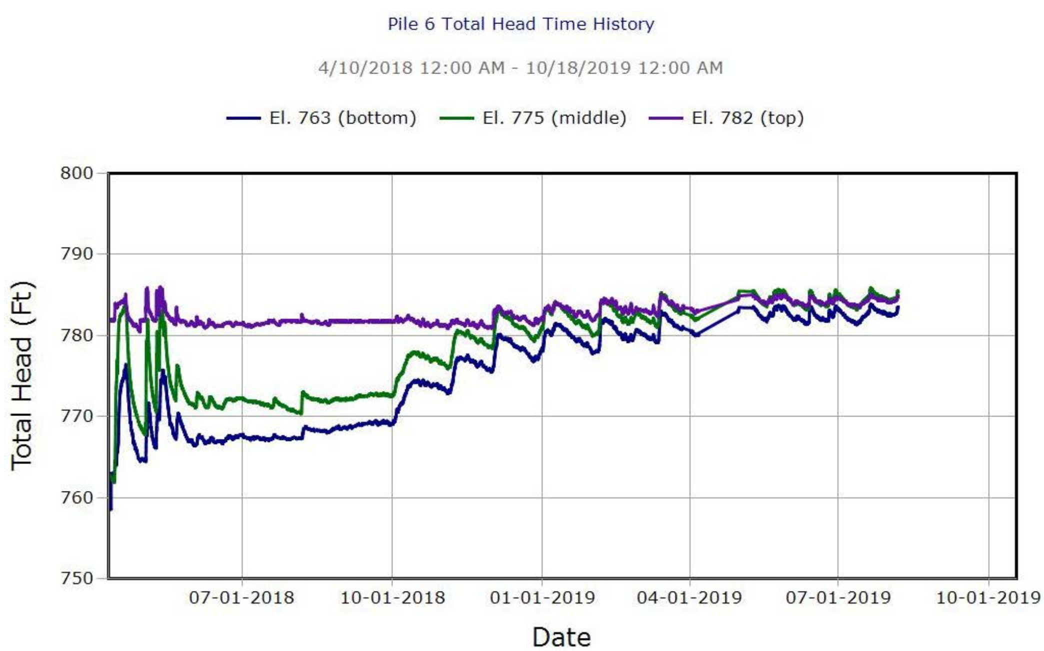
Instrumentation (Research Team)			
Time of measurement	Displacement, in	Maximum Measured Bending moment, kip-ft (strain gauges)	Maximum Measured Bending moment, kip-ft (SAA)
Maximum during 16 months of monitoring (including the pore pressure rise in 2019 winter)	0.33 at the top of the wall	30 below excavation level (~9.4 feet depth from the top of the wall)	100 below excavation level (~11 feet depth from the top of the wall)

Pore Water Pressure Measurements:

The Forward 45 design team assumed that the retained soil was dry and no active water pressures would exist in the retention system. Figure 6-2 shows a time history of the pressure heads for cantilever wall #6 and Figure 6-3 shows the total head profile from start of construction (April 2018) till the end of observation period (August 2019).



(a)



(b)

Figure 6-2 Time history of pressure head (a) and total head (b) measurements from start of construction (April 2018) till end of observation period (August 2019) for cantilever wall #6

We can make the following observations from these data:

- Pore water pressure measurements indicate the presence of positive pressure heads in the foundation and retained soils.
- All piezometers registered periods of positive heads during the construction period.

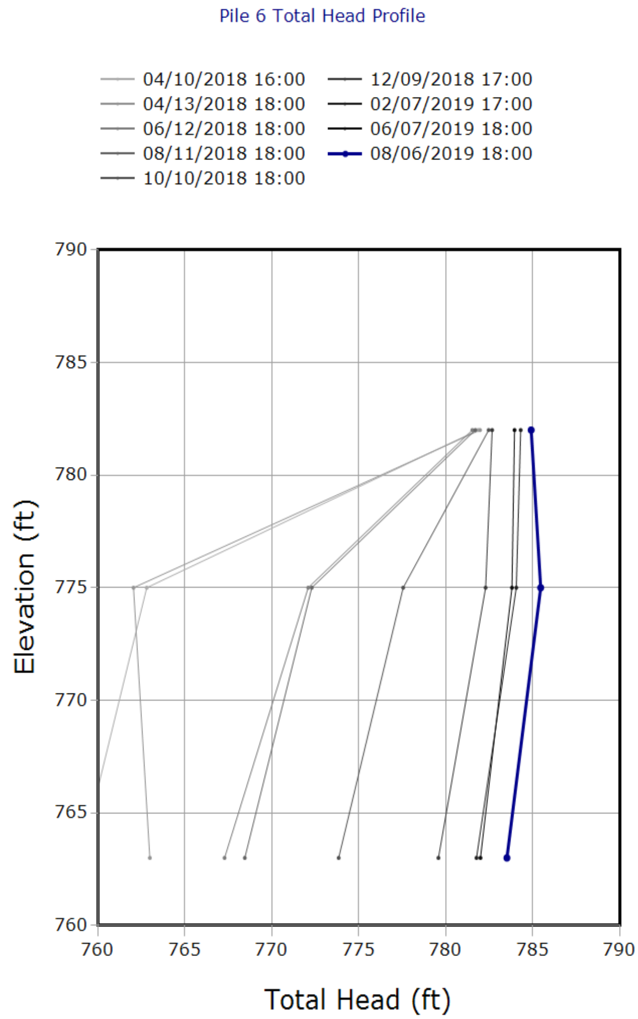


Figure 6-3 Total head profile from start of construction (April 2018) till end of observation period (August 2019) for cantilever wall #6

- From 10/1/2018 to 8/6/2019 positive pressure heads steadily increased to total heads between 1 to 3 feet below the top of pile. This was unexpected and not considered in the original design. The pressure heads then remained at these higher levels for the duration of the monitoring period.
- The bottom most piezometer (≈2 feet below pile toe) recorded positive pressure heads throughout the entire monitoring period with maximum head reaching about 3 feet below the top of pile elevation.
- The middle piezometer (≈3 feet below top of concrete pile encasement) fluctuated between positive and negative pressure heads during the construction period, then increased and fluctuated within about 1 foot below the top of pile elevation from January to August 2019.

- The upper piezometer (≈6.5 feet below top of pile) fluctuated between positive and negative pressure heads during the construction period, then increased and fluctuated within about 1 foot below the top of pile elevation from January to August 2019.
- None of the piezometers registered temperatures below freezing during the winter monitoring season. All piezometers are more than 8 feet from any exposed surface.

To provide insight into the cause of water pressures within five (5) feet of the drain we performed a simplified flow analysis using Plaxis. For the clay we used a permeability of $1x10^{-8}$ cm/sec, and for the sandy silt we used $1\ x10^{-4}$ cm/sec. We assigned a far field total head of EL 785 feet and we computed the steady state pressures at the locations of the piezometers (i.e. five (5) feet behind the wall). The analysis showed that the zone of influence of the drain did not extend 5 feet back to the piezometer locations. The computed steady state heads at the piezometer locations are very close to hydrostatic. The analysis also showed that for low permeability soils the drain does not draw the water level down in the soil adjacent to the drain by very much. The analysis supports the calculations of the wall deformations and forces using the hydrostatic water pressures at EL 785 feet.

6.1.3. Comparison of Results for Cantilever Wall

Displacement Results:

The displacement measurements reached a maximum value of 0.33 inches and were stable for about six months. The time history of displacement measurements is shown in the Instrumentation Data Report in Appendix D.2. Table 6-6 shows a comparison of computed displacement results and measurements.

- Calculated deformations by the Forward 45 design team for the cantilever cut wall were larger than the measured deformations. It should be noted that the design calculations were under drained conditions with no pore pressures.
- Measured displacements are less than all the calculated displacements even if any of the calculated displacements used factored loads. This low displacement measurement at the top of the wall can be attributed to restraining effect of the slab towards the top of the wall.
- Incorporation of *p-y* curves developed from in-situ pressuremeter data improved the deformation calculations as compared to the use of generic *p-y* curves used by PY-Wall or the simplified apparent pressures (Rankine and Coulomb) calculated by SPW-911.

Table 6-6 Comparison of displacement results for Cantilever Wall #6

<table>
<tr><th colspan="8">Displacement Results (in) - Cantilever Wall #6</th></tr>
<tr><th rowspan="2">Software Program</th><th>Analysis Assumptions</th><th colspan="4">Analysis</th><th>Design[2]</th><th rowspan="2">Measured</th></tr>
<tr><th>p-y curves</th><th>Undrained with hydrostatic pore pressures [submerged unit weight]</th><th>Drained with hydrostatic pore pressures [submerged unit weight]</th><th>Drained with measured pore pressures [submerged unit weight]</th><th>Drained with no pore pressures [Total unit weight]</th><th>Drained no pore pressures (Service Level I)</th></tr>
<tr><td>SPW-911</td><td>-</td><td>0.60</td><td>2.60[1]</td><td>2.10</td><td>0.40</td><td rowspan="3">0.81</td><td rowspan="3">0.33</td></tr>
<tr><td rowspan="2">PY-Wall</td><td>Int. generated</td><td>0.99</td><td>13.00[1]</td><td>4.50[1]</td><td>1.10</td></tr>
<tr><td>User-defined</td><td>0.80</td><td>1.10</td><td>0.88</td><td>0.53</td></tr>
</table>

Notes: [1]F.S < 1.0, the wall is not stable [2] Design team considered service limit for displacement calculations, different from research team they considered surcharge loads including live load and wind load due to the noise barrier wall.

Maximum Moment Results:

Bending moment calculations from strain gages and SAA data are proportional to the assumptions of flexural stiffness of the laterally loaded pile. The bending moment calculations from the instrumentation data only considered the flexural stiffness of the steel. The maximum bending moment calculated from the SAA measurements, 100 kip-ft, was compared with the results computed by design team and the research team. Table 6-7 shows a comparison of computed maximum moment results and measurements.

- Calculated bending moments by the Forward 45 design team for the cantilever cut wall were less than the measured bending moments. It should be noted that the design calculations were under drained conditions with no pore pressures, and the loads considered in earth pressures were factored for bending moments (made larger).
- The maximum bending moments computed by incorporating *p-y* curves developed from in-situ pressuremeter data compares better with measured maximum bending moments than bending moments computed by using generic *p-y* curves used in the PY-WALL program. Consideration of the pore water pressure in the ground has a significant effect on the computed maximum moment. Incorporating measured pore pressures resulted in a computed maximum moment more comparable to the measured maximum moment (i.e. computed 118 kip-ft versus 100 kip-ft measured).

Table 6-7 Comparison of maximum bending moment results for Cantilever Wall #6

Maximum Moments (kip-ft) - Cantilever Wall #6							
Software Program	Analysis Assumptions	Analysis				Design	Measured
	p-y curves[1]	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]	Drained no pore pressures (Strength Level I)	
SPW-911	-	109	289[(1)]	235	68	85[(2)]	100
PY-Wall	Int. generated	123	292[(2)]	233[(1)]	122		
	User-defined	108	146	118	71		

Notes: [1]F.S < 1.0, the wall is not stable [(2)] Design team, different from research team, considered strength limit for moment calculations and they considered surcharge loads including live load and wind load due to the noise barrier wall.

Earth Pressure Results:

We attempted to back calculate earth pressures from the computed slope of the bending moment curve. The back calculation of the earth pressures from measured deformations of the wall is very sensitive to small (less than 0.1 inch) changes in the measurements. From the Bending moment curve shown in Table 6-3 there are several inflexion points where the pile curvature changes direction from bending in one direction to bending in the opposite direction. These changes are dependent on relative differences of less than 0.1 inches between two adjacent SAA sensors. As discussed in section 6.1.2 the measured bending moments suggest that the wall has a point of counter flexure where at the excavation level the wall bends toward the excavation while at the top it needs away from the excavation. Without knowing the forces involved that cause the counter flexure the back calculated pressures are unreliable.

- The bending moment data for Pile 6 show multiple inflexion points indicating a change in bending from away from the support soil to towards the support soil. These changes in bending result from additional forces acting on the wall from the footing and possibly the slab at the top of the wall. These unknown forces lead to uncertainty in the back calculation of the earth pressures.
- Earth pressures computed from the SAA measured bending moments are erratic because they are sensitive to very small relative displacement changes between the sensors.
- Even with the uncertainty in the unknown forces an average envelope of pressures is within the range expected using the AASHTO LRFD (Load Resistance Factor Design).

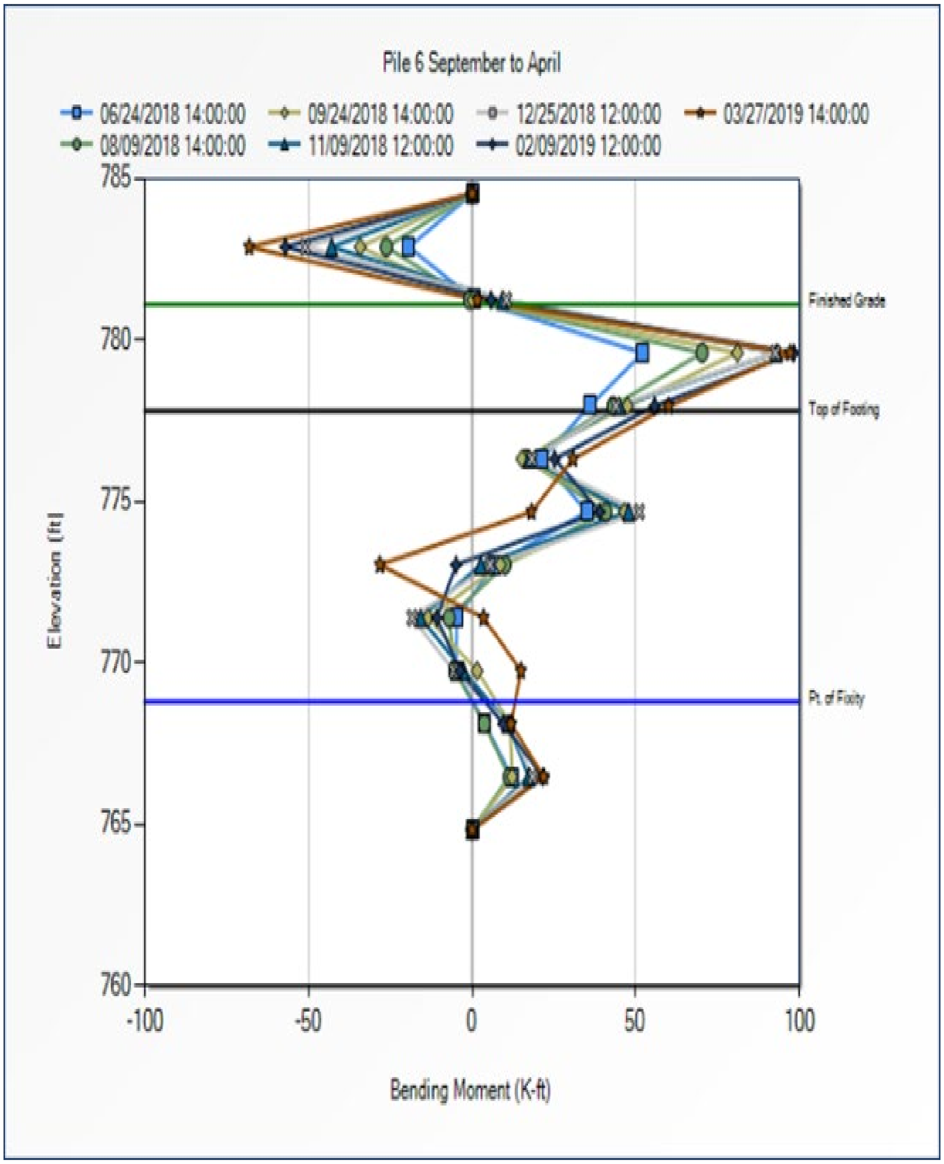


Figure 6-4 Measured bending moments for Cantilever wall #6 computed from SAA

6.2. CALCULATED AND MEASURED RESULTS FOR ANCHORED WALL AT PILE #36

6.2.1. Calculated Results for Anchored Wall

This section presents displacements and moments for the anchored wall at pile #36. It includes results computed by the Forward 45 design team computed under long term drained conditions with no pore pressures on the wall. Design calculations conducted by the Forward 45 design team were provided to Geocomp by WisDOT and are included in Appendix F. Similar to the cantilever wall, this section also presents computed displacements and moment values by the research team using SPW-911 and PY-WALL under four different conditions; undrained with pore pressures, drained with pore pressures, drained with measured pore pressures, and drained with no pressures. Outputs from the analyses performed by the research team are included in Appendix H. For all analyses, the exposed height of the wall was 22 feet and the anchor height were 9.5 feet. Measured anchor locks off load of 98 kips was used as analysis input. Measurement of anchor load is explained in detail in the following measured results section.

Displacement Calculations:

The Forward 45 design team calculated a maximum displacement of 1.25 inches for the Service I Limit State. It should be noted that computed design values are computed with factored loads. Also, the pile length and maximum exposed height used in the design are 2 feet longer than as built. Design calculations conducted by the Forward 45 design team were provided to Geocomp by WisDOT and are included in Appendix F.

Computed displacements for the anchored wall are shown in Table 6-8. Maximum displacement calculated by SPW-911 program was 1.0 feet above the bottom of excavation. Maximum displacements that were calculated by PY-Wall were at the top of the pile.

Table 6-8 Calculated displacement results for Anchored Wall #36

Computed Displacements (in) – Anchored Wall #36							
Software Program	Analysis Assumptions			Analysis Maximum Wall Displacement [Inches]			
	p-y curves	Excavation Depth, ft	Anchor Depth, ft	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]
SPW-911	-			0.80	0.40	-	0.30
PY-Wall	Int. generated	22	9.5	1.10	1.24	1.41	1.73
	User-defined			1.07	1.21	1.34	1.54

Like the cantilever wall, additional analyses with different flexural stiffnesses were done for the anchored wall. Computed displacements for the anchored wall are included in Table 6-9.

Table 6-9 Computed displacements with different flexural stiffness – Anchored Wall #36

Computed Displacements with different flexural stiffness (kip-ft) (Undrained with pore pressures) – Anchored Wall #36				
Analysis Tool	Analysis Assumptions		Excavation Depth, ft	Displacement, inches
	Flexural stiffness	*p-y* curves[2]		
PY-Wall	E_sI_s	Internally generated	22	1.10
		User-defined		1.07
	$E_sI_s + E_cI_c$	Internally generated		1.60
		User-defined		1.09
	$0.5*(E_sI_s + E_cI_c)$	Internally generated		1.54
		User-defined		1.06

Bending Moment Calculations:

The Forward 45 design team calculated bending moments of 220 kip-ft using AASHTO LRFD formulation Strength I Limit State. Design calculations conducted by the Forward 45 design team were provided to Geocomp by WisDOT and are included in Appendix F.

Computed maximum moments for the anchored wall are shown in Table 6-10. Bending moments calculated using the SPW-911 program yielded a maximum moment close to the bottom of excavation whereas for PY-Wall program the maximum moment was computed at the anchor level both for internally generated and user-defined p-y curves. The maximum moment is not as sensitive to the selection of different p-y curves as it is for the cantilever wall.

Table 6-10 Calculated maximum moments results for Anchored Wall #36

Computed Maximum Moments (kip-ft) – Anchored Wall #36							
Software Program	Analysis Assumptions			Analysis Results			
	p-y curves	Excavation Depth, ft	Anchor Depth, ft	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]
SPW-911	-	22	9.5	176	171	-	207
PY-Wall	Int. generated			129	167	143	144
	User-defined			113	141	142	143

Results of maximum moments computed for different flexural stiffnesses are included in Table 6-11. Incorporating flexural stiffness of composite sections had an impact on the location of the maximum bending moment.

Table 6-11 Computed maximum moments with different flexural stiffness – Anchored Wall #36

Computed maximum moments with different flexural stiffness (kip-ft) (Undrained with pore pressures) – Anchored Wall #36				
Analysis Tool	Analysis Assumptions		Analysis Results	
	Flexural stiffness	*p-y* curves	Depth, ft	Maximum Moments, kip-ft
SPW-911	E_sI_s	-	9.5 (anchor level)	104
			18.5	176
PY-Wall	E_sI_s	Internally generated	9.5	113
			17	129
		User-defined	9.5	113
			27	100
	$E_sI_s + E_cI_c$	Internally generated	9.5	157
			28.5	325
		User-defined	9.5	157
			30.5	188
	$0.5*(E_sI_s + E_cI_c)$	Internally generated	9.5	157
			28	258
		User-defined	9.5	157
			30.5	159

Anchor Load Calculations:

The Forward 45 design team calculated 122 kips anchor load using AASHTO LRFD formulations Strength I Limit State. Anchor loads computed under the four different analysis conditions for the anchored wall are shown in Table 6-12.

Table 6-12 Calculated anchor load results for Anchored Wall #36

Computed Anchor load (kips) – Anchored Wall #36							
Software Program	Analysis Assumptions			Analysis Results			
	p-y curves	Excavation Depth, ft	Anchor Depth, ft	Undrained with hydrostatic pressures	Drained with hydrostatic pressures	Drained Measured pressures	Drained w/o pressures
SPW-911	-	22	9.5	74	129	-	125
PY-Wall	Int. generated			97	116	109	86
	User-defined			90	109	104	85

Computed anchor loads for different flexural stiffnesses of the pile are shown in Table 6-13.

Table 6-13 Computed anchor load with different flexural stiffness – Anchored Wall #36

<table>
<tr><th colspan="5">Computed Anchor load with different flexural stiffness (kips) – Anchored Wall #36</th></tr>
<tr><th rowspan="2">Analysis Tool</th><th colspan="2">Analysis Assumptions</th><th rowspan="2">Excavation Depth, ft</th><th rowspan="2">Anchor Load, kips</th></tr>
<tr><th>Flexural stiffness</th><th>p-y curves</th></tr>
<tr><td rowspan="6">PY-Wall</td><td rowspan="2">E_sI_s</td><td>Internally generated</td><td rowspan="6">22</td><td rowspan="2">131</td></tr>
<tr><td>User-defined</td></tr>
<tr><td rowspan="2">$E_sI_s + E_cI_c$</td><td>Internally generated</td><td rowspan="2">121</td></tr>
<tr><td>User-defined</td></tr>
<tr><td rowspan="2">$0.5*(E_sI_s + E_cI_c)$</td><td>Internally generated</td><td rowspan="2">124</td></tr>
<tr><td>User-defined</td></tr>
</table>

6.2.2. Measured Results for Anchored Wall

This section presents measured displacements, moments, and pore pressures from instrumentation data collected throughout construction and the 1-year monitoring period at the instrumented walls. The findings presented in this section are based on instrumentation data presented in the *Instrumentation Data Report*, August 2019 presented in Appendix D.

Data was collected from the time period starting in April 2018 and ending in August of 2019. Automated instrumentation was installed and consisted of Shape Accelerometer Arrays (SAA), strain gages, and vibrating wire piezometers. These instruments were connected to Geocomp's *i*SiteCentral automated data collection and management system and were read hourly throughout the monitoring period. Data was also collected from the General Mitchell Airport detailing a time history of rainfall and snowfall amounts during the time period.

1. Displacement Measurements

- Deformation profiles obtained at the anchored wall indicate that the system exhibits fixed-earth conditions with negligible deformations within about 10 feet above the toe of the pile.
- A change in the slope of the deformation profiles is observed at about the elevation of the top of the concrete footing (El. 771 feet).
- Changes in the slope of the deformation profiles are observed above and below the elevation of the anchor head (El. 782 feet).

- After approximately 16 months of monitoring, the maximum measured deformation was approximately 1.50 inches.
- Approximately 0.60 inches of horizontal deformation was measured between the start of excavation and the anchor installation (maximum excavation depth was approximately 12 feet from the top of the wall).
- After performance testing and lock-off was achieved the deformations reduced to approximately 0.5 inches.
- 0.9 inches of horizontal deformation was measured at the maximum excavation depth which was approximately 22 feet from the top of the wall. In other words, an additional 0.40 inches of horizontal deformation was measured between anchor lock-off and final excavation.
- After installation of the CIP face, an additional 0.10 inches of deformation was measured and the total deformation at the end of construction was approximately 1.0 inch.
- In the winter months between the end of January 2019 and the end of March 2019, 0.5 inches of additional post-construction deformations were measured. This additional deformation registered in the winter months rebounded in the spring of 2019 to the total deformation magnitudes recorded prior to the winter, indicating elastic deformation induced by either temperature effects or frost action.
- Some of the deformations measured within the construction period appear to be related to the development of pore water pressure within the retained soil

2. Bending Moment Measurements

- The bending moment distribution obtained from strain gages and SAA displacement profiles indicate that the maximum moment develops near the tieback location.
- Due to the locations of the strain gages, the measurements did not capture the maximum moments at the location of the anchor where the maximum magnitudes were expected.
- The inferred maximum bending moments from SAA displacement profiles were calculated at several stages both during and after construction and presented below sequentially:
    - i. 80 kip-ft at level of excavation (approximately. 12 foot depth) during cantilever stage prior to anchor installation
    - ii. 46 kip-ft at the level of the anchor location after anchor installation
    - iii. 64 kip-ft at the level of the anchor during excavation to maximum depth (22 feet)
    - iv. 82 kip-ft at the level of the anchor after CIP installation
    - v. 140 kip-ft at the level of the anchor during the 2019 winter (post-construction)

vi. 80 kip-ft at finish grade level during the 2019 winter (post-construction)

- The maximum bending moments were measured by strain gages at several stages both during and after construction:
  i. 33 kip-ft at approx. depth of 4 feet below excavation level during cantilever stage
  ii. 36 kip-ft below the level of the anchor after anchor installation
  iii. 13 kip-ft above level of the anchor during excavation to maximum depth (22 feet)
  iv. 33 kip-ft above level of the anchor during after CIP installation
  v. 33 kip-ft above level of the anchor during the 2019 winter (post-construction)
  vi. 66 kip-ft below finish grade near the top of concrete footing during the 2019 winter (post-construction)
- Calculation of moments from strain gage measurements are more direct than the ones calculated from curvature from the SAA measurements. However, the calculated moments from strain gages will be limited to the location of the strain gages whereas SAA provide a continuous profile of calculated moments. Therefore, the difference in maximum moment magnitude measured with the two methods is due to the more refined spacing available from the SAA data as compared to the widely spaced strain gages along the length of the pile.
- During winter, fluctuations in bending moment diagram is observed as shown in below Figure 6-5. Possible explanation for these fluctuations can be freezing and thawing of soil/water behind the wall or changing water pressures from freezing.

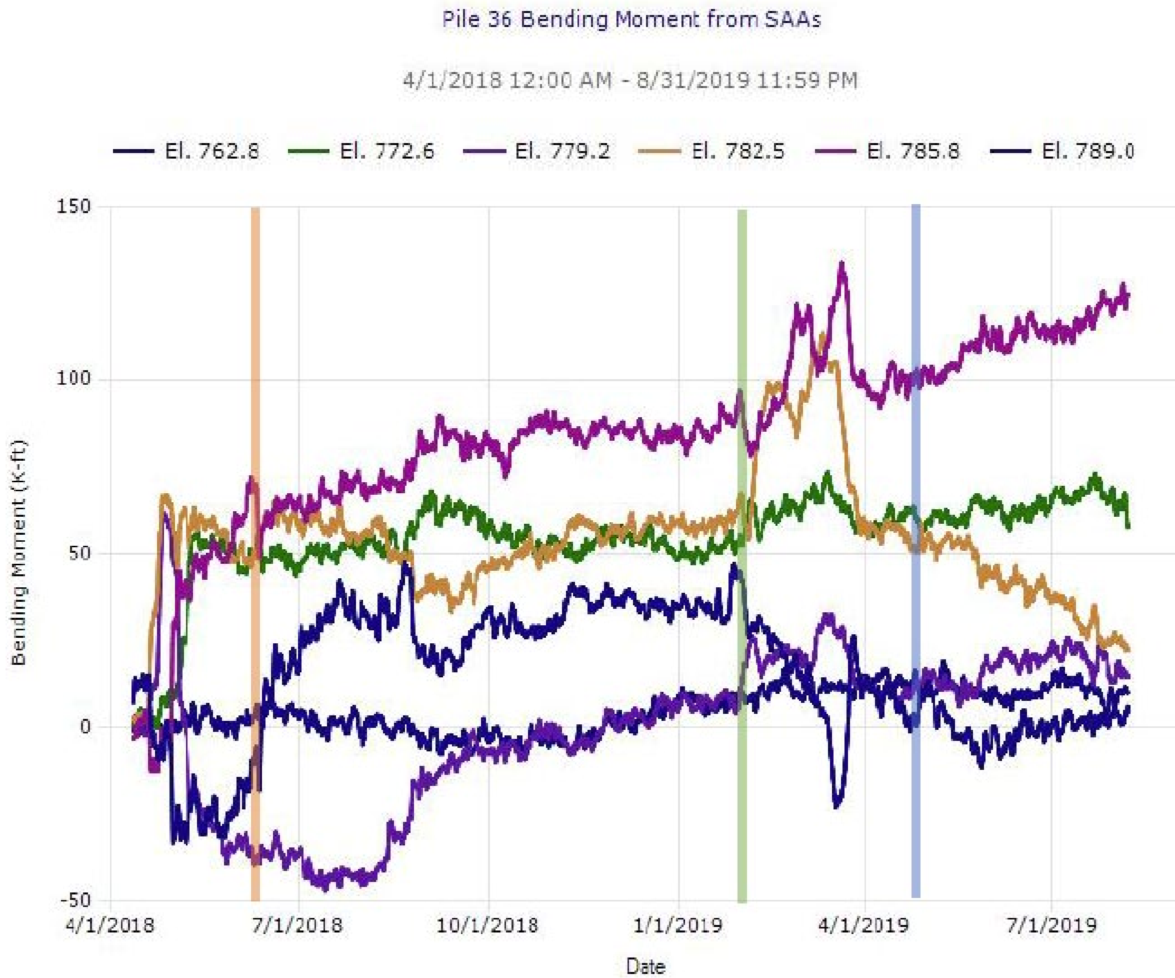


Figure 6-5 Time history of bending moments from start of construction (April 2018) till end of observation period (August 2019) for anchored wall #36

3. Anchor Load Measurement

- The anchor load measurements obtained from the instrumented lock nut at the anchor head indicate that the load varied from 49.5 kips after lock-off and increased to a maximum measured load of 133 kips during CIP placement (83.5 kips load change).
- The instrument at the anchor head appears to have been damaged during casting of the CIP face. It is not clear if the maximum load of 133 kips noted above occurred when this instrument was damaged. Detailed construction activities were not provided to the research team during this time period.
- The increase in anchor load measurements correspond to the subsequent excavation stages from 12 feet to 22 feet.

- A lift-off test was not performed during performance testing to verify the specified lock-off load of 98 kips.
- It is likely that the measured anchor load during the winter of 2019 increased based on the additional displacements that were measured.

Table 6-14 summarizes the measured displacements and bending moments for the anchored wall at Pile #36.

Table 6-14 Displacement and bending moment from instrumentation data for Anchored Wall at Pile #36

Instrumentation (Research Team)				
Time of the measurement	Displacement in	Bending moment kip-ft (SAA)	Bending moment, kip-ft (strain gauges)	Anchor Load, kips
Start to 12 ft of excavation	0.60	80 at excavation level (12 ft depth)	33 below excavation level (16 ft depth)	-
Anchor installation and lock off	0.50	46 at the level of anchor (9.5 ft depth)	36 at the level of anchor (9.5 ft depth)	-
Final excavation (max. depth of 22 ft)	0.90	64 at the level of anchor (9.5 ft depth)	13 at the level of anchor (9.5 ft depth)	49.5
Installation of CIP installation	1.00	82 at the level of anchor (9.5 ft depth)	33 at the level of anchor (9.5 ft depth)	133*
Maximum during 16 months of monitoring (including the pore pressure rise in 2019 winter)	1.50	140 at the level of anchor (9.5 ft depth)	33 at the level of anchor (9.5 ft depth)	-
		80 at finish grade (17 ft depth)	66 at finish grade (17 ft depth)	-

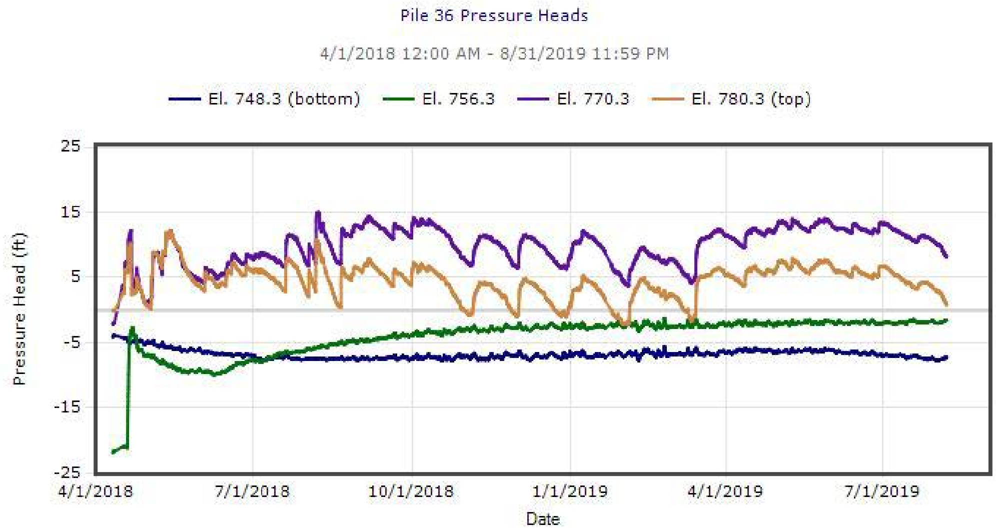
*The equipment appears to have been damaged during casting of the CIP.

4. Pore Water Pressure Measurements

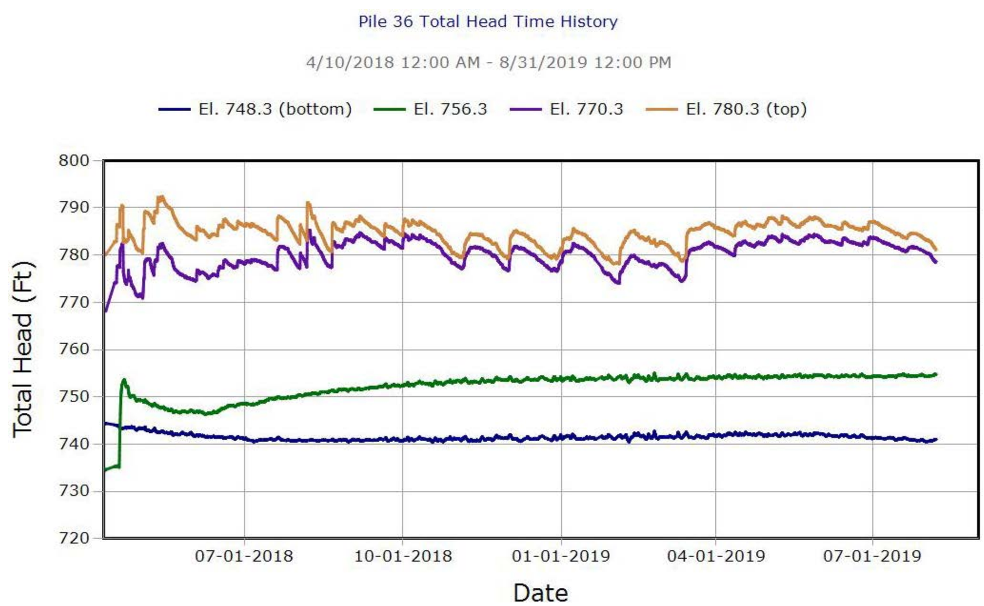
The Forward 45 design team assumed the retained soil had no water pressures occurring in the system. Figure 6-4 shows a time history of the pressure heads and Figure 6-5 shows the total head profile from start of construction (April 2018) till the end of observation period (August 2019).

- Pore water pressure measurements indicate the presence of positive pressure heads in the foundation and retained soils.
- The top 2 piezometers registered positive heads during and after construction.

- The bottom 2 piezometers located below the toe of the pile elevation registered negative pressure heads for the entire monitoring period.



(a)



(b)

Figure 6-6 Time history of pressure head (a) and total head (b) measurements from start of construction (April 2018) till end of observation period (August 2019) for anchored wall #36

- The piezometer installed at approx. El 770 feet (4 feet below finished grade) registered an average 10 feet (El. 780 – 12 feet below top of pile El.) of pressure head and a maximum of 15 feet (El. 785 – 7 feet below top of pile El.).

- The top two piezometers installed at approx. El 780 feet (12 feet below top of pile El.) were influenced by rainfall. The piezometers registered an average 4.5 feet (El. 784.5 – 7.5 feet below top of pile El.) of pressure head and a maximum of 12 feet (El. 792 – at top of pile El.).
- None of the piezometers registered temperatures below freezing during the winter season.
- Active construction of the wall took place from April 2018 – June 2018. Detailed construction activities were not provided to the research team during this time period.

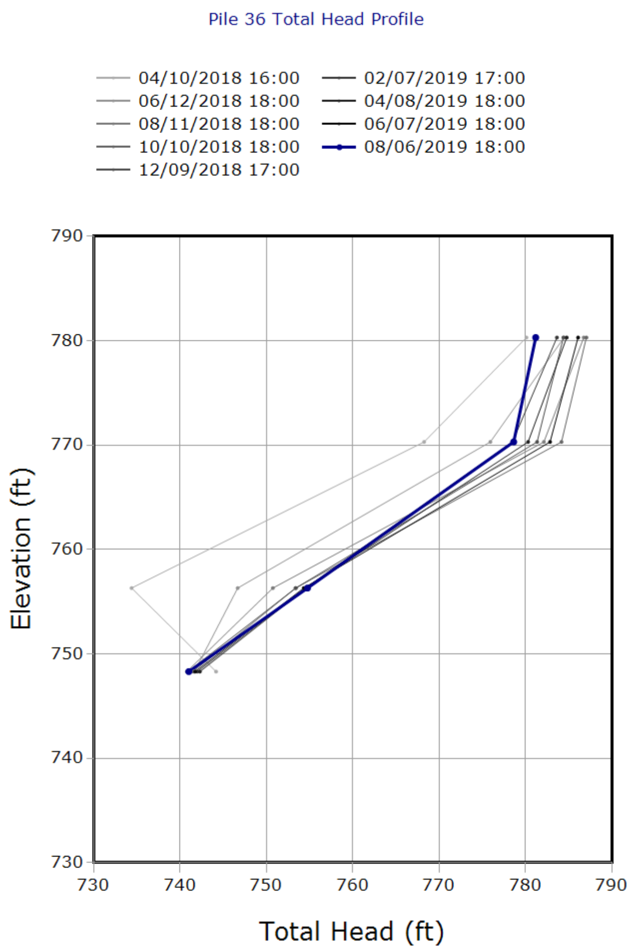


Figure 6-7 Total head profile from start of construction (April 2018) till end of observation period (August 2019) for cantilever wall #36

6.2.3. Comparison of Results for Anchored Wall

Displacement Results:

Table 6-15 shows comparison of computed displacement results and measurements for Anchored Wall #36.

- Calculated deformations by the Forward 45 design team for the cantilever cut wall were larger than the measured deformations. It should be noted that the design calculations were under drained conditions with no pore pressures.
- The calculated deformation profile obtained from the SPW-911 analyses does not agree with the measured deformation profile. In SPW-911, fixity was not achieved, and the higher deformations were calculated in the lower portion of the pile while the deformation measurements indicate fixity within the lower 10 feet of the pile and maximum deformations at the top of the pile. The deformation profiles obtained from PY-Wall are in good agreement with the measured deformation profiles, considering that both result in the maximum deformations occurring at the top of the pile.
- Incorporation of *p-y* curves developed from in-situ pressuremeter data improves the deformation calculations as compared to the use of generic *p-y* curves used by the PY-Wall program or the simplified trapezoidal apparent pressures (Terzaghi and Peck) calculated by SPW-911.
- We considered variations in the flexural stiffness of the pile due to the influence of concrete in the embedded section Changes in flexural stiffness of the laterally loaded pile elements has a significant impact on deformation calculations. We did not consider variations in the stiffness of the tie-back.
- The effect of water pressures was clearly observed in computed values. Consideration of loading conditions related to the presence of positive pore water pressure in the system and freeze/thaw conditions can provide better estimations of service performance.

Table 6-15 Comparison of displacement results for Anchored Wall #36

Displacement Results (in) - Anchor Wall #36							
Software Program	Analysis Assumptions	Analysis				Design (1)	Maximum Measured Displacement
	p-y curves	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]	Drained w/o pore pressures (Service Level I)	
SPW-911	-	0.80	0.40	-	0.30	1.25	1.10
PY-Wall	Int. generated	1.10	1.24	1.41	1.73		
	User-defined	1.07	1.21	1.34	1.54		

Notes: (1) Design team considered service limit for displacement calculations, different from research team they considered surcharge loads including live load and wind load due to the noise barrier wall.

Maximum Moment Results:

Like the results for the cantilever wall, the bending moment calculations for the anchored wall from strain gages and SAA data are proportional to the assumptions of flexural stiffness of the laterally loaded pile. The bending moment calculations from instrumentation data only considered the flexural stiffness of the steel. In addition, it should be noted that the Forward 45 design team did not include water pressures in calculations for displacement and bending moments. Table 6-16 shows a comparison of computed maximum moment results and measurements.

Table 6-16 Comparison of maximum bending moment results for Anchor Wall #36

Maximum Moments (kip-ft) - Anchor Wall #36							
Software Program	Analysis Assumptions	Analysis				Design (1)	Maximum Measured Moment
	p-y curves	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]	Drained no pore pressures (Strength Level I)	
SPW-911	-	176	171	-	207	210	140
PY-Wall	Int. generated	129	167	144	143		
	User-defined	113	142	142	144		

Notes: (1) Design team, different from research team, considered strength limit for moment calculations and they considered surcharge loads including live load and wind load due to the noise barrier wall.

- Design Service I bending moment calculations for the anchored cut wall were higher than the measured bending moments.
- The calculated maximum bending moment from SPW-911 is in good agreement with the measured maximum bending moment.
- Consideration of the presence of positive pore water pressures in the system has significant effect on the computed maximum moments indicating the importance of determining the actual pore pressure conditions in front of, and behind the wall.
- Incorporation of *p-y* curves developed from in-situ pressuremeter data improves the calculation of maximum bending moment as compared to the use of generic *p-y* curves used by PY-Wall.
- The bending moment distribution obtained from PY-WALL results is in better agreement with the measured bending moment profiles. Bending moment distributions are included in Appendix H.2. The moment reversal indicated by the measured profiles was validated by the PY-WALL results. However, the magnitude and location of the maximum moments are highly dependent on the assumptions of flexural stiffness for the concrete encased section of the pile. When the composite stiffness of the concrete encased section of the pile is considered, the bending moments developed in the lower portion of the pile can exceed the moments developed at the anchor level.

Anchor Load Results:

Table 6-17 shows a comparison between computed anchor loads and measurements. The measured change in the anchor loads caused by the additional wall movements in the winter months was not captured due to damage of the strain gages during the CIP face installation.

- Calculated anchor loads by the design team with Service Limit I state were lower than the measured anchor loads, whereas calculated anchor loads with Strength I state were in good agreement with the measured anchor loads.
- SPW-911 calculated the anchor loads lower than the measured anchor loads.
- PY-WALL Service I anchor load calculations are in good agreement with the anchor load magnitudes measured during construction.

Table 6-17 Comparison of anchor load results for Anchored wall #36

Anchor Load (kip) - Anchor Wall #36							
Software Program	Analysis Assumptions	Analysis				Design[1]	Maximum Measured Anchor Load[2]
	p-y curves	Undrained with hydrostatic pore pressures [submerged unit weight]	Drained with hydrostatic pore pressures [submerged unit weight]	Drained with measured pore pressures [submerged unit weight]	Drained with no pore pressures [Total unit weight]	Drained no pore pressures (Strength Level I)	
SPW-911	-	74	129	-	125	122	133
PY-Wall	Int. generated	97	116	109	86		
	User-defined	90	109	104	85		

Notes: [1] Design team, different from research team, considered strength limit for anchor load calculations and they considered surcharge loads including live load and wind load due to the noise barrier wall. [2] The equipment measuring anchor load appears to have been damaged during casting of the CIP.

Earth Pressure Results:

The earth pressures were back calculated from the computed slope of the bending moment curve. Back calculated earth pressures are shown in Figure 6-9. The back calculation of the earth pressures from measured deformations of the wall is very sensitive to small (less than 0.1 inch) changes in the measurements. The back calculated bending moments depend on relative differences of less than 0.1 inches between two adjacent SAA sensors. As shown in Figure 6-9 the back calculated earth pressures are erratic. From the bending moment curve shown in Figure 6-8, there are several inflexion points where the pile curvature changes direction from bending in one direction to bending in the opposite direction. These changes are expected at the tieback location and at the footing location. It’s especially difficult to back calculate the earth pressure for the tieback wall without measurements of the tieback loads.

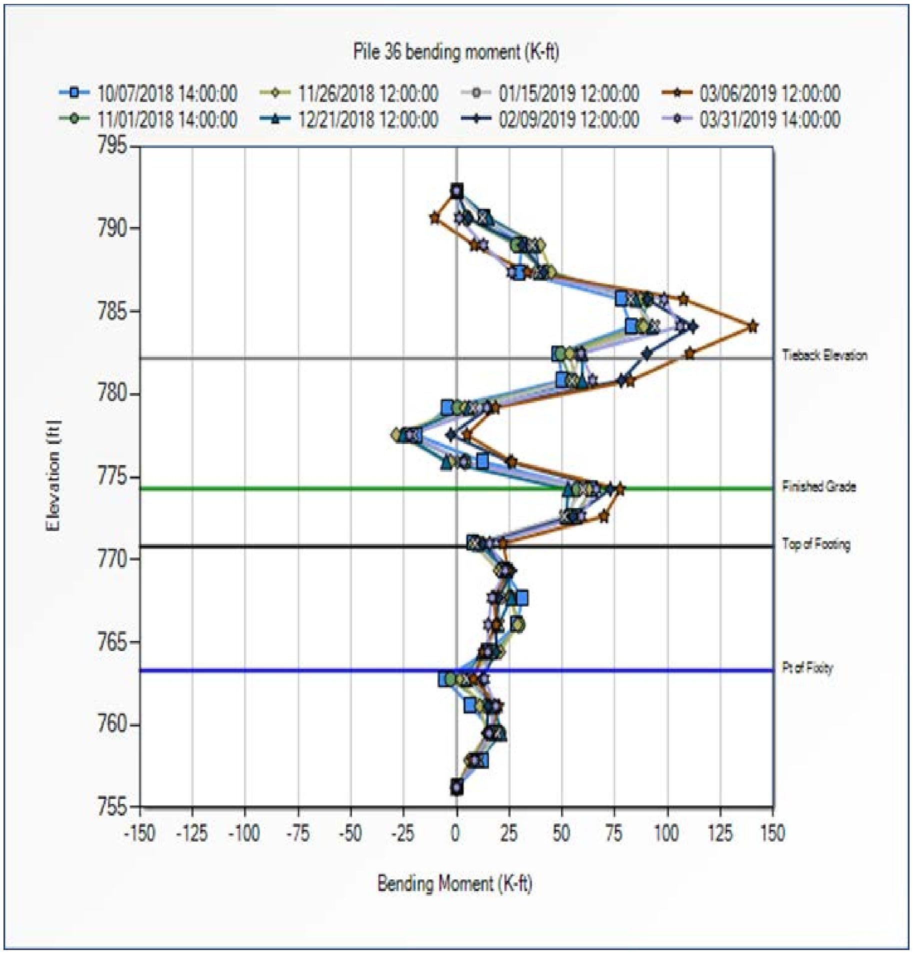
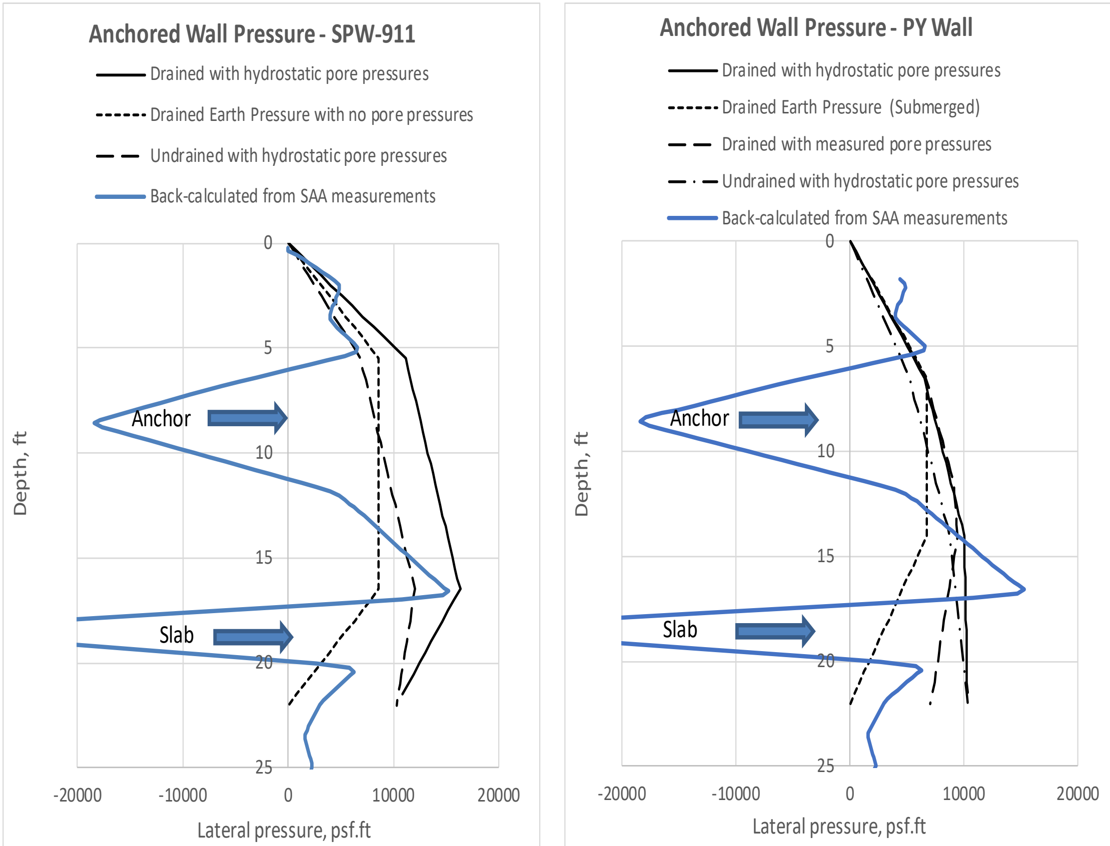


Figure 6-8 Measured bending moments at Anchored wall #36

- Unknown forces at the tieback and footing lead to uncertainty in the back calculation of the earth pressures.
- Earth pressures computed from the SAA computed bending moments are erratic because of they are sensitive to very small relative displacement changes between the sensor.
- The maximum envelope of pressures computed from the SAA is within the range expected using the AASHTO methods.



(a) (b)

Figure 6-9 Back-calculated earth pressures compared with earth pressures used in analyses of Anchored Wall # 36 (a) PY-Wall (b) SPW-911

7. CONCLUSIONS AND IMPLEMENTABLE RESULTS

A key objective of this research was to instrument and monitor the performance of a cantilevered retaining wall and an anchored retaining wall and to obtain data with which to evaluate methods used to design other similar walls in Wisconsin. This objective was achieved with the successful measurement of wall displacements, strains in the walls' structural elements and pore pressures in the retained soil over a period of 15 months that included construction of the walls and almost a year of in-service performance measurements. Measurements were obtained every hour continuously over the full period of monitoring.

The measurements demonstrated the following:

- The monitoring approach of using electronic sensors, data loggers, remote power and wireless data transmission to a central data storage system was very successful. Data were continuously and reliably collected except for a few instances where communications were lost. These instances required brief site visits by the research team to restore communications. In addition to the performance monitoring, more than 11,000 readings were obtained over the monitoring period to demonstrate the effects of environmental conditions (precipitation and temperature) on wall performance. This monitoring provided almost continuous data during construction and the post construction period.

- In the fall just after construction was completed, positive pore water pressures developed in the retained soil to almost hydrostatic conditions with the water surface near the top of the retained soil. These elevated pore water pressures continued until the monitoring was stopped 15 months later. We hypothesize that the walls might be preventing lateral seepage from occurring hence deterring seepage that would have allowed surface water that entered the slope from rain and snow melt to drain off. The original design assumed zero pore water pressures behind the wall.
- Temperature measurements showed a wide range of temperatures from a high of about 45 C to a low of -10 C on the wall elements measured by the strain gages. There was a high variability of temperature measurements on the wall elements prior to CIP installation. Only the uppermost strain gauges (0.5 feet from top of pile) experienced freezing temperatures during the winter. The displacement measurements on the Pile 36 (Tieback) wall showed outward movement during freezing conditions. This displacement was recovered once the ground thawed.
- The wall movements measured with the SAA inclinometer were very repeatable and consistent over time. The maximum wall movement was measured at the top of the wall and was 1.5 inches.
- The moments deduced from the wall movements measured with the SAA were continuous along the profile of the pile whereas moments calculated from strain gage measurements were limited to the locations of strain gages. The moments calculated from the SAA measurements were larger than the moments calculated from the strain gage measurements. This is attributed to the much closer spacing of the measurement points (1.6 feet spacing) with the SAA method compared to that of the strain gages (5 feet spacing) therefore providing much better determination of moment and its distribution.
- Plans to measure settlement of the ground surface behind the wall during and after construction were not successful due to unexpected interferences with the measurement method and construction activities.
- The back calculation of the earth pressures from measured deformations of the wall is very sensitive to small (less than 0.1 inch) changes in the measurements. These changes depend on relative differences of less than 0.1 inches between two adjacent SAA sensors. We found that the back calculated earth pressures are erratic and sensitive to inflexion points in the piles which are

influenced by the slabs at the top and bottom of the wall and the tie back. It's especially difficult to back calculate the earth pressure for the tieback wall without measurements of the tieback loads.

A second objective of the research was to compare measured performance of the two wall types to the performance calculated with the design methods in common use in Wisconsin to assess the adequacy of these methods. Two methods commonly used in Wisconsin were considered, SPW-911 and PY-WALL. SPW-911 accepts input earth pressures and hydrostatic pore pressures to compute wall forces from equilibrium and bending theory. PY-WALL uses a generalized beam-column formulation with earth pressures input by the user or computed from non-linear springs (p-y resistance curves). Soil resistance is defined with nonlinear p-y resistance curves. Water pressures are not explicitly included but may be added as user-specified pressures. Both methods provide calculations of maximum moment and horizontal displacement of the wall. Neither method gives calculations of settlement of the retained ground behind the wall. The original design by Forward 45 used the PY-Wall method for both wall types. The design calculations were performed for drained conditions without pore water pressure for a Stability Limit State and for a Service Limit State. The strength limit state was used to determine the maximum bending moments, and the service state was used to determine the maximum wall deflections.

Comparing the calculated maximum moment in the design to measured maximum moment we found the following:

- For the cantilever wall, the maximum moment computed by the design team is 85 kip-ft and the measured maximum moment is 100 kip-ft. For the anchored wall, the maximum moment computed by the design team is 210 kip-ft and the measured maximum moment is 140 kip-ft.
- We tried to reproduce the calculated maximum displacement and bending moment for both walls calculated by the design team. We used their original dimensions and earth pressure diagrams supplied in their report. However, we were unable to reproduce their results. The maximum moment and displacement numbers reported by the Forward 45 design team for the cantilever wall was suspect.

Comparing the calculated maximum displacement in the design to the measured maximum displacement shows that:

- For the cantilever wall, the maximum displacement computed by the design team is 0.81 inches and the measured displacement is 0.33 inches. For the anchored wall, the maximum displacement computed by the design team is 1.25 inches and the measured maximum displacement is 1.1 inches.
- There was a geometry difference for the cantilever wall – the design exposed height was 12.5 feet, but the actual constructed exposed height (highest during construction) was 10.5 feet.

There was a geometry difference for the anchored wall- the design exposed height was 21 feet, but the actual constructed exposed height (highest during construction) was 22 feet.

- Since we could not reproduce the Forward 45 design result of displacement and moment, we did not succeed in adjusting their results for the dimensional differences. The reported results for the cantilever wall are for an exposed height of 12.5 feet. For the anchored wall, the reported results are for an exposed height of 21 feet where the actual height is 22 feet. The reported displacement and moment would have been slightly higher with the actual dimension.
- Computed displacements with a composite stiffness are significantly less than displacements computed with the soldier pile stiffness alone.

We made calculations for the two walls using SPW-911 with the material parameters used by Forward 45 for the original design. Comparing the calculated maximum moment to the measured maximum moment we found:

- For the cantilever wall the calculated moment from the drained analyses with no pore pressures was 68 kip-ft which was lower than measured moment of 100 kip-ft.
- For the anchored wall the calculated moment from the drained analyses with no pore pressures was 207 kip-ft which was very high compared to the measured moment of 140 kip-ft.

Comparing the calculated maximum displacement to the measured maximum displacement we found:

- For the cantilever wall the calculated displacement from the drained analyses with no pore pressures was 0.4 inches which is quite close to measured displacement of 0.33 in.
- For the anchored wall the calculated displacement from the drained analyses with no pore pressures was 0.3 inches which is low compared to the measured displacement of 1.1 inches.

These results show that: a drained analysis with no pore pressures in SPW-911 compares relatively well with measurements especially for cantilever wall.

We made calculations for the two walls using PY-WALL with the material parameters used by Forward 45 for the original design. Comparing the calculated maximum moment to measured maximum moment we found:

- For the cantilever wall the calculated moment from the drained analysis with no pore pressures was 122 kip-ft when internally generated p-y curves were used, and 71 kip-ft when user-defined p-y curves were used.

- For the anchored wall the calculated moment from the drained analysis with no pore pressures was 143 and 144 kip-ft when internally generated and user-defined p-y curves were used respectively. Both design moments were slightly higher than the measured moment of 140 kip-ft.

Comparing the calculated maximum displacement to the measured maximum displacement we found:

- For the cantilever wall the calculated displacement from the drained analysis with no pore pressures was 1.10 inches when internally generated p-y curves were used and 0.53 inches when user-defined p-y curves were used. The latter calculated displacement was closer to the measured displacement of 0.33 inches.
- For the anchored wall the calculated displacement from the drained analysis with no pore pressures when user-defined p-y curves were used was 1.54, and 1.73 inches when internally generated and user-defined p-y curves were used respectively. Both calculated displacements were higher than the measured displacement of 1.1 inches.

These results show that:

- The use of p-y curves generated from in-situ pressuremeter data results in more realistic deformation and bending moment design estimates especially for cantilever wall.
- The use of composite stiffness for concrete encased laterally loaded piles can improve deformation and bending moment design estimates but requires careful consideration of cracking moments of the concrete section to determine appropriate flexural stiffness.
- The PY-Wall program gave results that better compare with the measured values of moment and displacement than SPW 911.
- Design values which considered factored loads in earth pressure distribution for the worst-case scenario should be more than the measured moment and displacements. However, this was not the case for the cantilever wall because the measured deflections were likely affected by the slab. The moment reported by the design team for cantilever wall was exceeded in calculated bending moments from SAA measurements. The development of significant positive pore water pressures behind the wall likely contributed to this difference as no pore water pressures were included in the design especially for cantilever wall.
- The maximum moment did not change in magnitude and location for different loading conditions. In general, the maximum moment was located at the bottom of the excavation for the cantilever wall and at the anchor level for the anchored wall.

We also examined the effects of adding water pressures into the analyses since significant pore water pressures were measured after wall construction in the retained ground. Performing the analyses with full hydrostatic pore pressures in the retained soil behind the wall (which is similar to what was measured on both walls) produced the following results:

- Maximum Moments results for the cantilever wall:

  The calculated maximum moments from the drained analyses with hydrostatic and measured pore pressures were 146 and 118 kip-ft when user-defined p-y curves are used respectively. These results are much higher compared to calculated moment from drained analyses with no pore pressures which was 71 kip-ft. The measured maximum moment was 100 kip-ft and thus the results incorporating pore water pressures predicted better with actual conditions.

- Maximum Moments results for the anchored wall:

  The calculated maximum moments from the drained analyses with hydrostatic and measured pore pressures was 142 and 143 kip-ft, respectively, when user-defined p-y curves were used. These values compared well with the measured moment of 140 kip-ft. The calculated moment from the drained analyses with no pore pressures was 144 kip-ft.

- Maximum displacements for the cantilever wall:

  The calculated maximum displacement from the drained analyses with hydrostatic and measured pore pressures were 1.1 and 0.88 inches, respectively, when user-defined p-y curves were used whereas, calculated displacement from the drained analyses with no pore pressures was 0.53 inches which shows the effects of pore pressures on the displacement results. The maximum measured displacement was 0.33 inches.

- Maximum displacements for the anchored wall:

  The calculated maximum displacement from the drained analyses with hydrostatic and measured pore pressures were 1.21 and 1.34 inches respectively when user-defined p-y curves were used. The results with user-defined p-y curves compare very well with the actual measured displacement of 1.1 inches. Calculated displacements from the drained analyses with no pore pressures when user-defined p-y curves were used was 1.54 inches. Horizontal displacement of the anchored wall seems to be less sensitive to the nature of the pressure distribution behind the wall. The maximum measured displacement was 1.1 inches.

We back calculated earth pressures using the displacement measurements from the SAA and we found:

- There is uncertainty in the calculation of the earth pressures due to additional forces acting on the wall. These forces include the tiebacks, support forces from the footing, and support forces from the slab at the top of the wall.

- Despite the uncertainty in the calculation, the envelope of average back calculated earth pressures is within the range expected using the AASHTO LRFD and Terzaghi and Peck methods with pore pressures and drained conditions (see Figure 6.3 and 6.6)
- With hindsight, back calculating earth pressures from measured horizontal movements of the wall might be improved by fitting a smooth mathematical function to the measurements and performing the calculations using the mathematical function rather than the individual measurements. This would more closely follow the continuous nature of the beam comprising the pile than of the discrete points used to measure the deformations from which the apparent earth pressures were computed.

These results show the importance of adequately considering pore water pressures in the retained backfill for the design of the wall since they can affect both the maximum moment for wall design. Buildup of positive pore pressures behind a wall designed with drained conditions and no pore water pressure can result in excessive moments in the wall and larger than anticipated lateral displacements. AASHTO LRFD states that if retained earth is not allowed to drain, the effect of hydrostatic water pressures shall be added to that of earth pressures. Furthermore, AASHTO states that cohesive backfill walls should be designed assuming the most unfavorable conditions with consideration for the development of pore pressures. In addition, FHWA manual on soils and foundations (NHI-06-089) recommends that for non free draining retained soils, in addition to the chimney drain against the wall a second drain be placed at the back of the retained backfill.

In this study we found that the AASHTO LRFD formulations for estimation of earth pressures are adequate for design when all applicable loading conditions are considered. AASHTO LRFD gave forces larger than measured in these two walls, but this is to be expected since AASHTO is an envelope for design whereas these measurements are for the smaller in-service conditions.

The measurements on the cantilevered wall showed that the wall moved outward during freezing weather. It is possible that the anchor in the anchored wall experienced an increased load during this time but the instrumented lock off nut failed so we have no measurements to prove this. Neither of the two design methods have a way to determine the effects of ground freezing on wall performance. There is little to no guidance that we could find in literature to deal with freezing ground. Industry practice is to ignore it or take steps to minimize ground freezing by adding insulation to the wall to prevent freezing directly behind the wall.

This research has demonstrated the following significant results:

- Pore pressures may develop in the retained soils behind walls in ways that designers do not anticipate. Designers should consider the various possible loading cases that could develop behind a wall, including earth pressures and water pressures, and design the wall to resist each of these in an explicit manner so as not to miss some important case that could create unacceptable performance. The designer must be careful to correctly use the applicable soil unit weights and pore water pressures to calculate forces on the wall following the AASHTO design guidelines.
- Temperature changes can affect the stresses in a wall and its lateral support system significantly, especially where the ground behind the wall can freeze. Until the potential effects of ground freezing behind the wall can be better understood and quantified, the recommended approach is to protect any exposed wall from freezing of the soil behind the wall.
- Designers should consider all possible load cases, including pore pressures, when developing lateral stresses that a wall must withstand throughout its life. These load cases need to consider how pore water pressures may develop behind, beneath and in front of the wall and change over the life of the wall. They should consider other sources of load as well, including ground freezing pressures, temperature changes, construction loads and loads from adjacent and ancillary structures. Most of the commonly employed methods for wall design such as PY-WALL and SPW-911 can be difficult to use to examine these other cases. Other methods such as the finite element method may be helpful.

Based on the results of this work we offer the following recommendations to WisDOT for its future wall design practice:

- Designers should include all applicable load cases to ensure that worst-case loading or combination loading is addressed. In this case the added loads from pore water pressure build up in the retained soil and potential ground freezing were not addressed by the original designers. These load cases should be considered in design or provisions should be made during construction to prevent the buildup of pore pressures behind the wall including drainage and surface water controls and prevent freezing of the ground behind the wall with insulation. WisDOT should consider developing standard details for protection against pore water pressure buildup and ground freezing behind the wall. It is not an easy task to quantify freezing pressures because they are dependent on the confinement and stiffness of the wall.
- Designers should consider both undrained and drained cases for each wall design to cover various possibilities that can develop in the field during construction and long-term post construction. Realistic

worst-case pore pressures acting against the wall and in the retained soil should be considered as well. These considerations will help provide additional protection so that a critical load case is not missed. In the case of the walls analyzed in this study, the undrained case does not govern the design. In these stiffer clays, the drained condition will be the worst case for design, but this would not be the case if soft clays were involved in the soil profile. As a precaution, we think it prudent for designers to explicitly consider both cases, so a critical case is not missed.

- Based on the results of this work, it appears that designers using the PY-WALL method should obtain soil parameters for actual site conditions using a pressuremeter or laboratory strength testing on undisturbed samples rather than using the p-y curves internally generated by the software.
- We recommend WisDOT consider requiring performance testing of a representative number of anchors in its specifications to reduce uncertainty in the actual anchor lock-off loads since these loads have a direct impact on the performance of the wall system. (The one measurement we had in this work was significantly less than the design lock-off load, not an unusual occurrence in our experience.)
- We recommend WisDOT consider requiring more careful documentation by contractors of the sequence of work and dates the work was performed for retaining wall construction (when each important step at each wall section is performed). In performing this research study, we had difficulty determining when specific actions were performed in the field so we could better examine cause and effect. This information is valuable not only for research like that done here, but also for evaluating cause and effect when problems arise in the field and result in disputes.
- Modern instrumentation and monitoring systems can provide near continuous data on the performance of retaining wall systems. For unusual cases, cases where poor performance could create significant risks and costs, and situations where there is value to be gained from improved understanding, strong consideration should be given to instrumenting and monitoring representative sections with near-real-time data collection methods. This is especially the case where measured results from early construction phases could be used to modify designs for later work. We also recommend additional monitoring for situations where substantial pore water pressures might develop behind walls not designed to withstand those pressures and for cases where the ground behind the wall might freeze and create unknown pressures on the wall that it is not designed to resist.
- For cut walls where the zone of influence of the construction might include existing utilities and/or buildings that could be impacted by ground settlement, methods other than SPW911 and PW-WALL should be used to predict wall and ground movements as neither of these programs calculate

movement behind the wall. These situations might require a more comprehensive method of analysis such as the finite element method available in several commercial software programs.

Worthwhile follow on research from this work might include investigation of the following:

- Instrument more walls of different types to determine if potential load cases can be identified and quantified in ways that the envelop of design pressures can be reduced for specific situations in ways that result in cost savings without incurring unacceptable performance.
- Further study of the effects of ground freezing behind walls to determine if the additional pressures from ground freezing can be accommodated in a design for less cost than insulating the wall to prevent ground freezing.
- Investigate ways to effectively and permanently reduce and control water pressures in the retained soils behind a wall to significantly reduce the loads the wall must resist.

8. REFERENCES

American Association of State Highway and Transportation Officials (2014) "AASHTO LRFD Bridge Design Specifications 8th Edition" Washington, D.C.

Forward 45, (2015) "Geotechnical Exploration and Foundation Evaluation Report Zoo Interchange Corridor Study"

GTSoft Ltd (2003) Soldier Pile Design Using SPW-911, "Sheet Pile Wall Design Software, SPW-911 User Manual"

United State Steel, (1984), "Steel Sheet Piling Design Manual", Updated and reprinted by U. S. Department of Transportation /FHWA with permission

WisDOT Bridge Manual (2017) "Chapter 14 – Retaining Walls"

Williams B.P. and Waite, D. (1993) "The Design and Construction of Sheet-Piled Cofferdams", Special Publication, Construction Industry Research and Information Association

Federal Highway Administration (2006) FHWA-NHI-06-089, "Soils and Foundations Reference Manual-Vol II"

9. APPENDICES

Appendix A – Kick Off Meeting PowerPoint and Meeting Minutes

Appendix B – Literature Review Report

Appendix C – Instrumentation Installation Report

Appendix D – Instrumentation Data Reports

Appendix E – Laboratory Index Test Results

Appendix F – Forward 45 Design Calculations

Appendix G – Forward 45 In-Situ Pressuremeter Data Report

Appendix H – Research Team Analysis Output