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ABSTRACT

The National Railroad Passenger Corporation (Amtrak) is replacing the 1907-era two-track bascule bridge over the Niantic River between East Lyme and Waterford, Connecticut, along the heavily traveled Northeast Corridor. Prestressed concrete sheet pile retaining walls were selected to support the new higher approach embankments along both the east and west approaches to the new bridge. Along the west approach a two-tiered wall design was utilized to support a new recreational walkway elevated above the 100-year storm surge elevation for the Niantic Bay, while at the same time keeping the walkway below the level of the adjoining tracks.

The design of the two-tier wall system needed to take into account two simultaneous Cooper E-80 train live loads, the influence of electric traction catenary structure foundations along the wall alignment, and live load surcharge from maintenance vehicles at the walkway level, while at the same time minimizing long-term impacts to the public beach. The concrete sheet pile wall was designed to support the upper prefabricated modular T-WALL® along with all imposed loads, while at the same time protecting the railroad embankment from the scour and wave action of a 100-year storm event in Long Island Sound, and taking into consideration challenging subsurface conditions.

INTRODUCTION AND PROJECT HISTORY

The National Railroad Passenger Corporation (Amtrak) is replacing the 1907-era bascule two-track bridge over the Niantic River between East Lyme and Waterford, Connecticut, along the heavily traveled Northeast Corridor. The existing bridge was built as a replacement for the pre-existing swing-span bridge, built in 1891. The existing bridge, No. 116.74, was constructed parallel to the former swing-span structure, and approximately 49 feet to the north. The bridge carries two tracks, 12 feet 11 inches on center, over the Niantic River and consists of a movable span and four approach spans supported on stone masonry piers. The movable span is a through-girder chain-driven, Scherzer rolling-lift bascule span with overhead counterweights. The horizontal navigational clearance for marine traffic in the river is 45 feet, and vertical clearance is 11.5 feet above mean high water (MHW) with the bridge in closed position.

The proposed movable bridge is a two-track, single-leaf Strauss-type bascule bridge with two approach spans and a central bascule span of 141.5 feet. The horizontal navigational clearance will increase to 100 feet with the bridge in its open position and the vertical clearance will increase to 16 feet above MHW with the bridge in its closed position. The bridge replacement project includes new bridge approach embankments and 2,511 lineal feet of retaining wall along the west approach and approximately 796 lineal feet of retaining wall along the east approach. The paper focuses on the west approach retaining wall, which was designed to minimize the impacts to the existing recreational beach on Niantic Bay and to accommodate a recreational pedestrian walkway along the length of the beach.

The existing tracks west of the river run east-west over a narrow spit of land known as "The Bar". Niantic beach is located on the south side of The Bar, fronting Niantic Bay, which leads into Long Island Sound. Niantic Bay is an arm of Long Island Sound and is occasionally subject to hurricanes.

The overall project limits and west approach retaining wall limits are shown in Figures 1A and 1B, respectively.

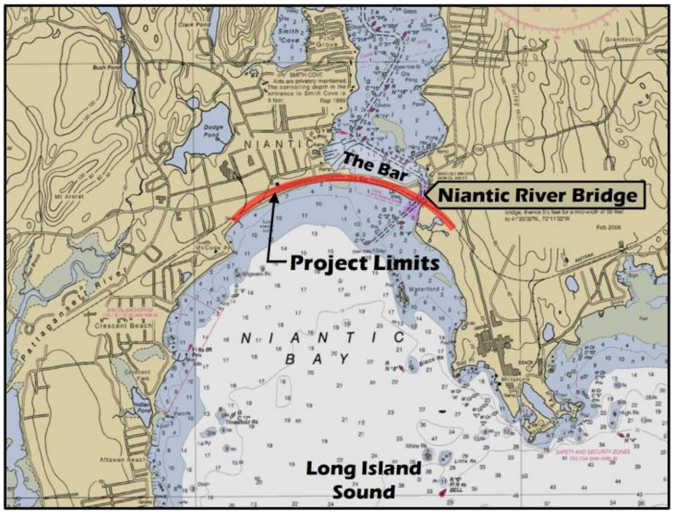

Fig. 1A. Project Limits

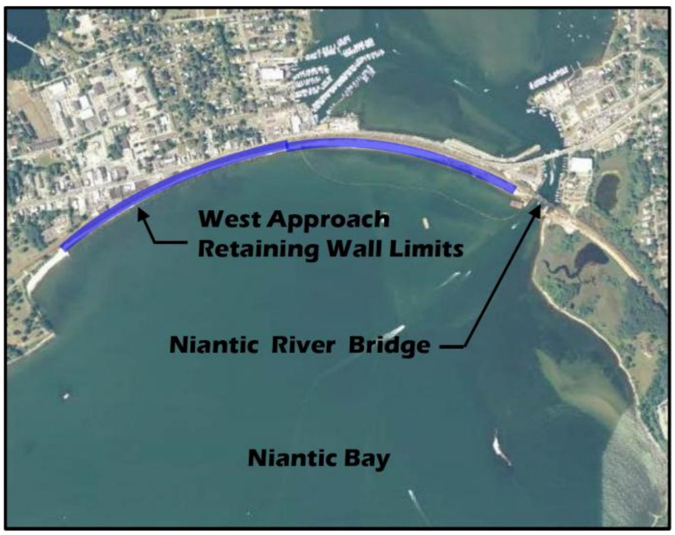

Fig. 1B. Site Aerial View with West Approach Wall Limits

Photo 1. Photo of the Aftermath of Great New England Hurricane (1938, Courtesy of Archives & Special Collections, University of Connecticut Libraries)

During the Great New England Hurricane (also known as the Long Island Express) of 1938, the railroad embankment suffered extensive damage from storm surge and wave action, in spite of the rip rap protection in place at the time. Damage from the Hurricane is seen in Photo 1 below.

SUBSURFACE CONDITIONS

Prior to evaluating wall alternatives, a total of 18 borings were drilled for the west approach retaining wall. The depth of these borings ranged from 55 feet to 145 feet. The average depth of the test borings performed in the first 900 feet west of the existing bridge was about 65 feet which included minimum of 5 to 10 feet of rock coring. Rock was cored at these locations to evaluate the condition of the bedrock and help analyze any deep foundation alternates for the proposed wall.

West of this location the depth to top of rock increased dramatically with presence of subsurface organic silty to clayey soils. The organic soils were encountered at depths ranging from 29 feet to 35 feet below ground surface. The test borings in this area were drilled to depths ranging from 82 feet to 145 feet with no test borings encountering top of rock. In general the borings were drilled beyond the depth of the organic layer and were terminated within the sand layer encountered underneath the organic layer.

The subsurface conditions along the west approach are fairly consistent for 900 feet westward of the new bridge. In this area, the soil consists primarily of loose to medium dense sands and silty sands with occasional gravel to depths between 20 and 48 feet. In addition, borings indicated the presence of scattered cobble and/or boulder-size size rocks at depths ranging from 2 to 10 feet below ground surface. These shallow cobbles and boulders are likely remnants of the historic embankment rip-rap that was buried during the reconstruction of the railroad embankment following the damage done by Great New England Hurricane. Below the sand layer, a dense to very dense layer of schist saprolite, ranging in thickness from 5 to 15 feet, extends to top of rock, which is encountered at depths between 51 and 61 feet below ground surface. The condition of the bedrock in this area is highly variable, with individual core recoveries ranging from 0 to 100 percent and Rock Quality Designation (RQD) ranging from 0 to 93 percent. In general, the hardness of the rock varies from soft to medium hard indicating a fair quality rock mass overall; however, with increasing depth below top of rock, the rock hardness varies from hard to very hard indicating a good quality rock mass.

Beyond this area, the subsurface profile changes significantly going westward. The loose to medium dense sands and silty sands still comprise most of the overburden soils; however, thick layers of soft organic silt and clay are also encountered with increase in depth below ground surface. One of the test borings performed in this area revealed 51 feet of organic silt and clay with Standard Penetration Test (SPT) N-values ranging from weight-of-rods to 6 blows per foot, indicative of very soft to medium consistency material. In addition to the presence of organic silt and clay, the depth to bedrock increases dramatically approaching the west end of the wall. This is evident from a boring near the west end of the wall that was drilled to 145 feet without encountering bedrock. Table 1 below summarizes the typical soil profile encountered along the new alignment of the west approach to the bridge.

Table 1. West Approach Typical Soil Profile

Stations Along West Approach (ft)		Stratum	Typical Thickness Range of Stratum (ft)
From	To		
96+15	82+00	Sand	8
		Sand with Silt	30
		Silty clayey sand	8
		Weathered Rock	10
82+00	71+00	Sand	32
		Organic Silt/Clay	42
		Sand	29

The groundwater at the site was encountered at an elevation ranging from 1.8 to -3.0 ft, and is influenced by the tides in the Niantic Bay and Niantic River, which typically fluctuate by about 2.5 feet. The impact of the tidal fluctuations on the groundwater elevations had to be accounted for in the wall design and construction. The 100-year storm elevation was established at 10.1 by FEMA for the Niantic Bay, and was utilized as the design storm surge elevation for the project.

WALL DESIGN CHALLENGES AND CONSIDERATIONS

The new retaining wall needed to be designed to support the two realigned tracks, while at the same time minimizing impacts to the existing passenger and freight rail operations during construction, and to the adjacent recreational beach in the long term. Additionally, the severe storm conditions that can be encountered within Long Island Sound and Niantic Bay required that the new wall system be adequately protected from potential scour.

Near the west end of the project, where the new track alignment ties into the existing alignment, the new retaining wall is very close to the existing tracks. At the river, the new track alignment reaches a maximum offset of 58 feet from the existing alignment. As a result, the proposed retaining wall system also pulls away from the existing alignment as it follows the new track alignment from the west end of the project towards the river. The offset between the wall alignment and the new track alignment was kept to a minimum to reduce environmental impacts and impacts to the adjacent recreational beach as outlined in the Finding of No Significant Impact (FONSI) report previously issued by the Federal Railroad Administration (FRA) for the project.

Wall Constructability

The combination of deep granular soils, high groundwater table, and close proximity of the proposed walls to the existing railroad embankment and tracks presented significant constructability challenges for the new walls.

To provide for long-term defense of the wall from storm surge and wave action, typical retaining wall systems on shallow foundations, such as cast-in-place concrete cantilever walls or prefabricated modular walls, would need to have a bottom of footing elevation at a significant depth below final grade to allow for installation of an appropriate scour protection system. This extended wall depth would then result in an increased overall wall height and width, which in turn would require excavation closer to the existing tracks in order to install the wall.

Any excavation falling within Amtrak's theoretical railroad embankment line, a line representing a theoretical embankment supporting the tracks with 1.5H:1V side slopes, requires temporary sheeting and shoring to maintain stability of the existing tracks. Over its length, the proposed wall alignment is close enough to the existing tracks that temporary excavation support would be required, increasing the overall cost and construction duration of the wall system.

Wall Type and Configuration

To address the design and constructability issues presented by more traditional wall systems with wider footprints, a permanent sheet pile wall system was selected for support of the widened railroad embankment. The use of sheet piling helped to minimize impacts to the existing tracks by moving wall construction operations further away from the active tracks. This also largely eliminated the need for temporary excavation support to protect the existing railroad embankment. Additionally, the use of a sheet pile wall eliminated the need for dewatering, and provided a wall system that could be more easily integrated with a scour protection system.

Prestressed concrete sheet pile panels, 4 feet in width and 1- to 2-feet thick, were selected for their combination of strength and long-term corrosion resistance against the aggressive marine environment present at the job site. The ability to install prestressed concrete sheet piles with a combination of jetting and driving made them a good candidate for the saturated sandy soils encountered at the project site.

The initial design concept for the west approach retaining wall system was a 1,388-foot-long prestressed concrete sheet pile wall, extending west from the bridge abutment location. The front face of the wall was to be offset 15 feet from the centerline of the proposed Track 2 alignment, the closest of the two tracks to the wall, and the top of this wall was to be located at approximately the proposed top of rail elevation for the proposed track realignment. A scour protection blanket would be placed in front of the wall to protect the wall from wave-driven scour action.

The sheet pile wall was originally designed with a final exposed height ranging from approximately 8 feet at the west end of the wall where the new tracks would tie into the existing track alignment, to 20 feet where the wall would tie into the new bridge abutment. This increase in vertical profile was necessary to accommodate the increased underclearance at the new bridge over the Niantic River. At the west end of the wall, where it was closest to the existing tracks and shortest in height, it could be designed as a cantilever section; however, once the wall exceeded approximately 12 feet in final exposed height, it was necessary to convert the wall to an anchored system. Two types of anchors were initially incorporated into the wall design. In the middle section of the wall, where the wall was closer to the existing tracks, the wall was designed with permanent inclined ground anchors. These could be installed while minimizing interference with the nearby rail operations. With the large loads that needed to be supported by the anchors, one ground anchor was required for each four-foot-wide wall panel.

For the eastern section of the wall, where it was furthest away from the existing tracks, it was possible to use piles attached to tie rods for the anchor system. The anchor piles were conceived as prestressed square concrete driven piles, driven at an offset of about 40 feet behind the rear face of the concrete sheet piles, and then attached to the sheet pile wall using horizontal high-strength steel tie rods. The anchor piles had to be placed a sufficient distance from the back of the sheet pile wall to minimize overlap of the passive earth pressure zone of the anchor pile with the active earth pressure zone behind the wall panels. The anchor piles offered a cost advantage over the ground anchors, and were therefore the preferable operation where enough room was available to install them without affecting the existing tracks. Table 2 summarizes the originally-proposed wall system support details for the prestressed concrete sheet pile wall.

Table 2. Original West Approach Retaining Wall Support Summary

Station (ft)		Original Prestressed Concrete Sheet Pile System Support Types *(Going West from the Bridge Abutment)*
From	To	
96+15	90+70	Anchor piles with tie rods
90+70	86+64	Inclined ground anchors
86+64	82+22	Cantilever section

West of the retaining wall, the track realignment was to be supported on a widened embankment section with a 1.5H:1V side slope. The slope and toe of the widened embankment system was to have been protected from storm action by a substantial stone revetment system.

Scour Protection Considerations

The wall design required special considerations for scour protection while at the same time mitigating impacts to the adjacent recreational beach. The 100-year storm surge level established by FEMA for Niantic Bay is 10.1 feet above the National Geodetic Vertical Datum of 1929, the vertical design datum for the project. In contrast, the mean high water at the new bridge is at approximately EL. 2.0. The design considered a breaking wave height equal to 78 percent of the prevailing near-shore water depth during the 100-year storm, and up to 4.6 feet of scour was estimated at the wall as a result of the 100-year storm.

To minimize the impacts of the scour protection system on the beach in front of the wall, it was desirable to have most of the system buried beneath the restored beach during normal conditions. This would maximize the amount of post-construction usable beach space available to the public.

The scour protection system was designed as a layered system of natural stone projecting 25 feet from the front face of the wall, where it could protect the passive earth pressure zone that the sheet pile wall relies on for its stability. The uppermost layer of the system consisted of a single layer 1,900-pound armor stones on top of a double underlayer of 190-pound stone, over a 1.2-foot-thick bedding layer of 10-pound stone. To maintain separation between the bedding stone and the underlying sand present at the beach, a heavy-duty nonwoven geotextile was specified. In addition to separation, the geotextile also would help keep the bottom two layers of stone from raveling should storm action erode the sand on the bayside of the scour blanket. The total thickness of this scour protection system is approximately 6 feet, and was designed to be covered by a minimum of 1 foot of beach sand cover, thereby maintaining some usable beach area in front of the wall.

Figure 2 shows a typical cross-section of the originally-proposed cantilever section of the sheet pile retaining wall with the scour protection system at its face. Since up to 7 feet of excavation would be required in front of the in-place concrete sheet pile wall panels to install the scour protection system, it was necessary to analyze a construction case taking this intermediate wall configuration into effect. This case was especially important for the anchored sections of the wall, because the scour protection system was to be installed prior to anchoring the wall panels. Adequate factors of safety for the wall stability had to be maintained at all times during construction.

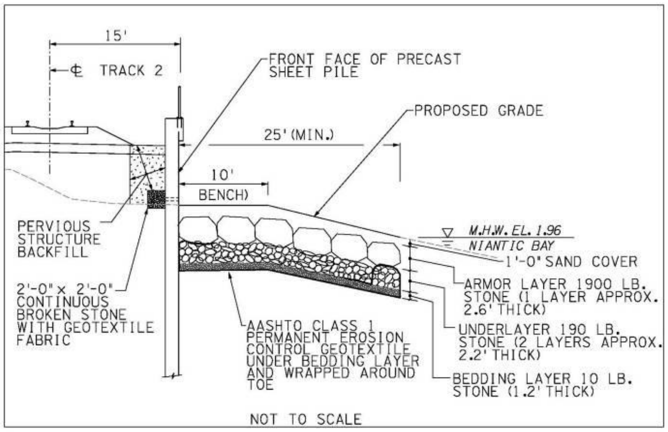

Fig. 2. Typical Cantilever Wall Section and Scour Protection System for Original Wall Design

Niantic Bay Overlook

General. During the preliminary design effort for the wall, the Town of East Lyme was in the process of constructing the Niantic Bay Overlook project along the beach between the railroad and the bay. Construction of the Overlook began at the end of October 2003 and was completed in May 2005. The purpose of this project was to build a continuous recreational walkway, roughly paralleling the railroad and adjacent beach, from Amtrak's Niantic River Bridge westward along the shore to the Hole-in-the-Wall Beach at McCook's Point Park. This walkway extended over a total length of approximately 5,340 feet, and included three different sections. The first section was an at-grade 5-foot-wide walkway of concrete and dirt sections, about 740 feet in length. To the west of that was an elevated timber boardwalk, 10 feet in width, extending for another 1,860 feet. Beyond this, the last section of the Overlook project was a 14-foot-wide stabilized stone dust walkway at grade. This final section extended about 2,740 feet to the west of the boardwalk.

Photos 2 and 3 below show the original elevated boardwalk section of the Overlook project.

Photo 2. Original Boardwalk (looking east towards bridge)

Photo 3. Original Boardwalk (looking east towards bridge)

Realignment and Reconstruction of Overlook. The original Amtrak project design included realignment and reconstruction of different sections of the Overlook walkway subsequent to construction of the bridge and west approach retaining wall. The design included the following changes to the Overlook:

- The 5-foot-wide at-grade walkway at the east end of the Overlook, with one section of concrete walk and the remainder of stone dust, was to be realigned parallel to the new wall and reconstructed as a 10-foot-wide at-grade concrete walkway. This portion of the existing Overlook fell entirely within the footprint of the new approach embankment.

- Approximately 1,100 feet of the elevated boardwalk, at its western end, was beyond the limits of the new retaining wall and would maintain its existing alignment; however, it was to be removed during construction and reconstructed at the end of the project to facilitate contractor access to the project area. The eastern end of the boardwalk required realignment and reconstruction to maintain a minimum offset of 10 feet from the face of the new approach wall.

Figures 3, 4, and 5 below show typical cross-sections of the original wall design and scour protection system, with the relocated elevated boardwalk structure or at-grade concrete walkway shown in front of the wall, depending on the location. It should be noted that on this project the stations increase going eastward, towards the bridge.

As shown in the figures, the relocated elevated boardwalk and at-grade concrete walkway were to be situated within the limits of the proposed scour protection system at the face of the wall. This presented significant challenges to the design, reconstruction, and long-term performance of these sections of the Overlook.

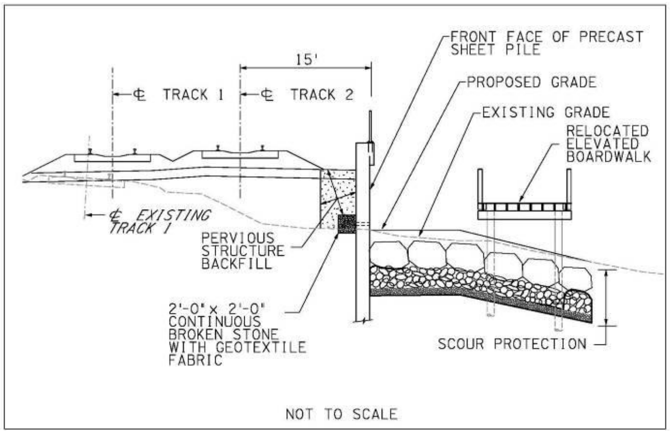

Fig. 3. Typical Cantilever Wall Section with Relocated Overlook Boardwalk, Sta. 82+22 to 86+64

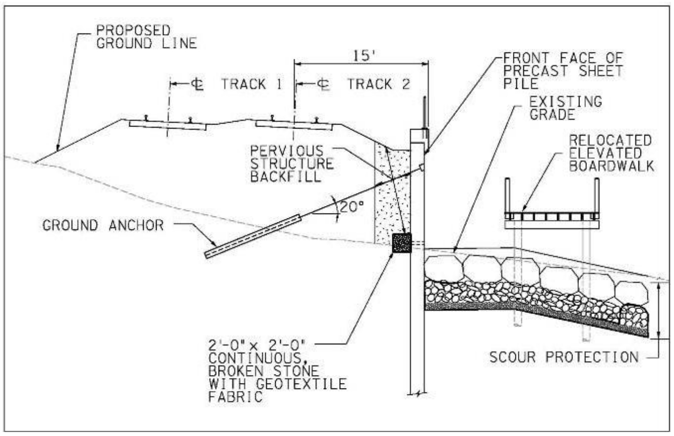

Fig. 4. Typical Anchored Wall Section with Relocated Overlook Boardwalk, Sta. 86+64 to 90+70

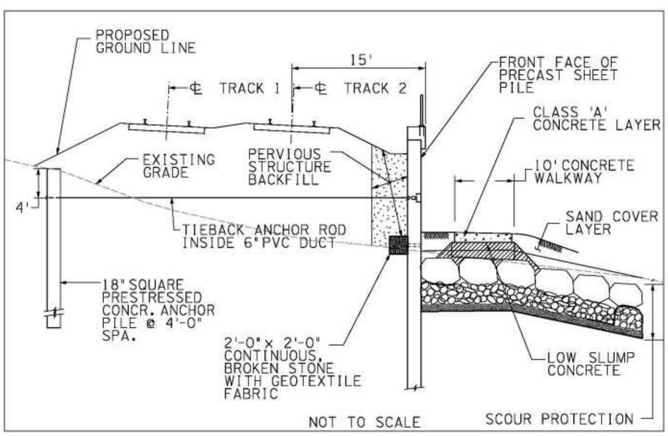

Fig. 5. Typical Pile Anchor Wall Section with At-Grade Concrete Walkway, Sta. 90+70 to 96+15

<u>Overlook Reconstruction Challenges</u>. To facilitate the retaining wall construction, sections of the existing at-grade walkways and elevated boardwalk would have be removed prior to wall construction. Subsequent to the completion of the wall construction, these displaced Overlook sections would need to be reconstructed in the same or new configurations, depending on the location along the length of the project. This would be further complicated by the fact that the boardwalk and at-grade walkways incorporated several interpretive educational signs for the Overlook users, as well as numerous benches with commemorative name inscriptions.

The removal of the boardwalk would entail partially disassembling and storing sections of the elevated superstructure for reuse and storing for reuse the benches along its length. These components would likely have to be moved offsite during construction, as onsite storage space would be at a premium due to the long, narrow work zone. Full disassembly and subsequent reassembly of the boardwalk superstructure would be prohibitively time-consuming and expensive. The timber piling supporting the boardwalk would also have to be removed so as to not interfere with construction operations, particularly the installation of the scour protection system at the face of the wall.

The reassembly of the boardwalk superstructure sections would be challenging in that the mounting locations for the sections would need to line back up with the newly-reinstalled piles. Otherwise, modifications to the pile support bents or superstructure assemblies might be required.

As shown in the previous figures, the reconstructed boardwalk would fall in the midst of the scour protection system for the wall (and to the west of the wall, in the revetment system for the embankment). Installing the boardwalk within the limits of the scour protection system would be a challenge because the of the large size of the scour protection stones in the 6-foot thick system. The 1,900-pound armor stones in front of the wall would be in the range of 2.5 feet or more in diameter. West of the proposed wall, the revetment system for the widened embankment was to use a layered stone approach similar to that of the scour protection system for the wall; however, the stone sizes required for the revetment were much larger than those required for the wall. At 7,000 pounds, the revetment armor stones were more than three times the weight of the armor stones for the wall, and more than a foot larger in overall diameter. Figure 6 shows a typical section of the widened embankment with revetment system, illustrating the reconstructed boardwalk.

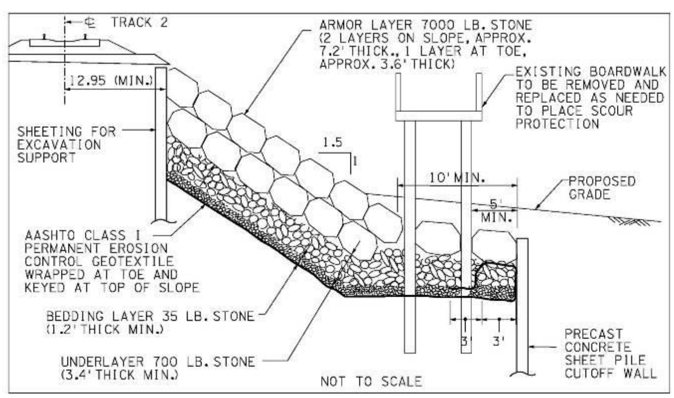

Fig. 6. Typical Embankment Section and Revetment System

If the piles were to be installed prior to placing the scour protection system, they would temporarily have significantly-reduced embedment of the pile tips, meaning they would be more susceptible to unintended displacement during installation of the scour protection system. The Contractor would have to be very careful not to damage the piles with the equipment or stone, and to not push the piles out of alignment.

It would be practically impossible to drive the timber piles through the scour protection system once it was already in place without making special provisions to do so ahead of time. One concept for this installation approach was to preinstall vertical sleeves of metal or plastic pipe in the scour protection system as it was being installed. The sleeves would be installed in the locations where the piles would be installed later on, allowing the piles to pass through the scour protection system without being damaged. A challenge of this approach would be to maintain the sleeves in the proper location and vertical alignment while installing the stone around them and not crushing or otherwise damaging the sleeves. Another potential drawback to this approach was the possibility that cobble or boulder-size obstructions could be encountered in the sand below the scour protection system as the timber piles were being installed. Since the pile could not be offset and re-driven due to the fixed location dictated by the preplaced sleeve, it would be necessary to try to pre-drill a hole through the sleeve to remove, break-up or displace the obstruction.

Long-term Performance of Boardwalk Structure. Once the elevated boardwalk was reconstructed in front of the proposed wall system, there were concerns about how it would fare during a major storm event. A benefit of the layered stone scour protection system proposed for the wall is that this type of system can flex and reconfigure itself should sand start to wash away at the toe of the system during a storm event, or if wave action where to shift individual stones in the armor layer. This effectively prevents the system from being undermined and enhances its long-term performance. However, the shifting of stones in the scour protection system could place large stresses on the timber piles supporting the boardwalk, causing potential damage to the structure.

Another concern was the effect that waves reflecting off the face of the retaining wall would have on the boardwalk, located just 10 feet in front of the wall face. Large reflected waves riding a storm surge could create significant simultaneous uplift and lateral forces on the boardwalk superstructure, potentially pulling it off of its pile bent supports, or otherwise damaging the structure. The boardwalk was designed to withstand only a 25-year storm event, so it was unclear how the reconstructed boardwalk would perform under the concentrated wave action in front of the wall during larger storm events. The design team performed analyses of the boardwalk to see if it could withstand the hydrodynamic forces at the face of the wall. As a result of the analyses, it was determined that the pile bents for the boardwalk should be augmented with additional diagonal bracing during the replacement of the boardwalk to withstand the lateral hydrodynamic forces of the 100-year design storm.

Long-term Performance of At-Grade Walkways. In addition to the challenges associated with reconstructing the elevated boardwalk, there were also concerns regarding the long-term performance of the at-grade sections of the Overlook affected by the bridge replacement project.

To the east of the boardwalk, the at-grade stone dust walkway was to be reconstructed as a 10-foot-wide concrete walkway, which would have to be built over the scour protection system to be placed in front of the wall. An aggregate base material was not considered appropriate for the walkway, because it could be eroded away in a severe storm event. To provide a firm and durable subgrade for the concrete walkway, a layer of low-slump concrete was to be poured over the finished armor stone to serve as a base layer for the sidewalk. A drawback of this approach is that the concrete base and sidewalk would be a rigid system with little tolerance for the differential movements that could occur if individual armor stones would settle or shift over time. This could cause cracking of the walkway and lead to accelerated deterioration of the system.

Relocation of the eastern stone dust walkway also presented another issue. The original walkway alignment was up on the side of the railroad embankment, with all but the eastern and western ends of the walkway between EL. 6 and El. 15. In addition, relocating the walkway to the front of the wall, would place the entire walkway at around EL. 4, thereby subjecting the walkway to more frequent flooding during moderate storm events producing higher-than-normal tides (mean higher high water in the river is at EL. 2.2). Eventually, the decision was made to replace this portion of at-grade concrete walkway with a new section of elevated boardwalk, which would keep this section of walkway from being flooded on a regular basis. This decision extended the eastern end of the boardwalk another 500 feet, with a ramp transitioning from the boardwalk level to a short at-grade concrete walkway running under the bridge near the face of the abutment.

EVOLUTION TO A TWO-TIER WALL SYSTEM

During the course of the design process, several status meetings were held to keep the various stakeholders apprised of the project progress. Once such meeting was held in November of 2008, to address concerns raised by the Town of East Lyme regarding the relocation and reconstruction of the Overlook. This meeting included representatives from the design team, Amtrak, the Connecticut Department of Environmental Protection (CT DEP), the Town of East Lyme, and the Town's design consultant, Applied Coastal Research and Engineering. The Town's main concerns were regarding the installation of the timber piles within the scour protection system; the effect storm waves being reflected off the wall and impacting the boardwalk; and accommodating the reconstruction of the western stone dust walkway over the large armor stones. An additional concern was raised by the Town regarding the long-term stability of the one-foot beach sand cover layer over the scour protection system.

As part of the meeting, the Town proposed a concept of offsetting the new retaining wall 12 feet further south towards the bay, to accommodate an at-grade walkway behind the wall, thereby eliminating the constructability and long-term performance concerns of having the Overlook in front of the wall. Subsequent to the November meeting, the Town provided sketches to illustrate their proposed wall and walkway configuration for review by Amtrak and the design team. One concept presented was to keep a full-height retaining wall but with a further offset from the track to accommodate the at-grade walkway. This would result in having the walkway at approximately the same elevation as the adjacent track. In this case, a fence would be required to keep pedestrian traffic on the walkway away from the active tracks. The second concept proposed using a two-tier retaining wall system to provide a grade separation between the tracks and the walkway. This would result in a shorter concrete sheet pile retaining wall adjacent to the beach with the walkway behind the sheet pile wall, and a prefabricated modular retaining wall providing the grade separation between the walkway and track. A separation fence would be required along the shorter sections of the upper wall, and along the remainder of the upper wall a railing would be needed for the safety Amtrak employees working at track level.

To eliminate issues with reconstructing the boardwalk along the toe of the widened embankment sections west of the wall, where the new revetment system would be installed, the Town also suggested extending the wall system approximately 1,100 feet further to the west, to the end of the beach, where it would meet the end of the at-grade stone dust walkway on the side of the railroad embankment. This would eliminate the entire elevated boardwalk and would result in the entire eastern half of the Overlook being protected from future storm action by the new wall system. As a result, the service life of this portion of the Overlook would be increased considerably.

With these concepts in hand, the design team performed a brief feasibility and cost analysis to evaluate the two options. Both options were considered technically feasible. To evaluate the relative costs of the options, it was necessary to select a potential secondary (upper) wall type for the two-tier wall design concept. To minimize costs and construction time associated with the secondary wall, prefabricated modular concrete wall systems were investigated. Mechanically stabilized earth options were not considered, as they are not typically accepted by Amtrak for support of their tracks. The T-WALL® system was ultimately selected to evaluate the two-tier wall because it is a gravity-type wall system with a favorable track record on railroad construction projects carrying freight rail loading (AREMA Cooper E-80 loading), and with its large precast concrete units it can be constructed much quicker than traditional cast-in-place concrete retaining walls.

The cost estimate revealed that the concept of using a single wall further from the tracks would slightly reduce the cost of the affected work along the west approach. Increases to the construction cost would result from extending the length of the wall approximately 1,100 feet westward, and from increasing the volume of backfill behind the wall to accommodate the walkway. Cost savings would be realized by not having to relocate the boardwalk and extend it further eastward towards the bridge, not having to construct the stone revetment system on the widened embankment west of the wall, and reducing fill volumes associated with widening the reconfigured embankment west of the originally proposed wall.

The two-tier retaining wall system was estimated to slightly increase the overall construction cost. Compared to the single wall option, the height of the wall at the beach would decrease, and the overall fill volume would be decreased, but the cost of adding the secondary wall would more than offset these savings and the other cost savings identified for the single wall versus the original wall and boardwalk concept. The estimated changes in construction cost for the two new wall alternatives were within about five percent of the cost for the originally proposed work.

Aside from the technical feasibility and cost of the proposed options, other considerations were how these potential changes would impact the CT DEP permit for the project, and if the change would affect the FONSI previously issued by the FRA for the project. The FONSI, issued in June of 2002, had identified the impacts in relation to public access to the beach via the Niantic Bay Overlook Structure, which had yet to be constructed. It was determined where impacts could not be entirely avoided mitigation or compensation would be proposed. The FONSI further stated that Amtrak would need to comply with the Connecticut DEP's request for an in-kind or better replacement of any impacted Overlook structure components. This general requirement in the FONSI allowed for flexibility of a replacement structure without the need to alter the document.

Ultimately, the two-tier wall alternate was selected by Amtrak as the best way to address East Lyme's concerns regarding the Overlook, while providing the greatest separation between the public and the railroad once the project was completed.

Since the CTDEP was an integral participant with the Town of East Lyme in the evolution of the structural alternatives for the replacement Overlook structure, it was a simple matter for Amtrak to resubmit the DEP permit with the appropriate modifications documenting the new two-tier wall system.

The resolution of this design issue demonstrated that communication, cooperation and coordination among the stakeholders lead to a successful implementation of a solution best addressing the needs of the project

TWO-TIER WALL DESIGN

Wall System Configuration

To accommodate a walkway behind the new retaining wall, and to incorporate a vertical grade separation between the walkway users and adjacent railroad traffic, a two-tier wall system was designed. To implement the new design concept, the west end of the approach retaining wall was extended west of the originally-proposed wall location over 1,100 feet, with a total wall length of 2,577 feet along the front face of the wall panels.

The two-tier system utilized the original concept of a prestressed concrete sheet pile wall along the beach as the primary retaining wall, with a secondary prefabricated concrete wall offset 10 feet behind it, and a new 10-foot-wide concrete walkway on the bench between the two walls. The front face of the prestressed concrete sheet pile wall was now located at 25 feet from the centerline of the realigned Track 2, an increase of 10 feet over the original design. This allowed incorporation of a 10-foot-wide walkway behind the wall, which matched the width of the walkway on the existing elevated timber boardwalk.

To maintain the top of the concrete sheet pile wall, and concrete walkway behind it, at an adequate elevation to protect the secondary wall system and walkway from a 100-year storm event, the elevation of the walkway was fixed at EL. 13.36 at the bayside edge of the walkway. Beyond the western limits of the originally proposed retaining wall, the secondary retaining wall was not required, as the proposed walkway grade was at a similar elevation to the proposed track embankment grades adjacent to the walkway. Thus, for the last 1,118 feet of the western approach wall, a single wall system was utilized while maintaining the walkway behind the wall. In this area, a concrete barrier wall with a security fence mounted on top was incorporated to separate the walkway users from the adjacent railroad.

The lowest overall sections of the wall system, at the western end of the west approach wall (Station 71+04 to 88+01), were designed with a cantilever concrete sheet pile wall section. This included the portion of the wall utilizing a single wall system, with the walkway close to adjacent track level, and several hundred feet of the two-tier wall system. Once the secondary wall reached an exposed height of about 7.5 ft, the resultant loadings on the supporting concrete sheet pile wall were great enough that an anchored system was required to control wall deflections and keep the moments in the prestressed concrete panels within allowable levels. From this point eastward, towards the river, the concrete sheet pile wall was designed as an anchored section. Table 3 summarizes the various configurations of the west approach wall.

Table 3. West Approach Retaining Wall Support Summary – Revised Design

Station (ft)		Wall Support Types (*Going West from the Bridge Abutment*)	
From	To	Wall Near Beach	Wall Near Tracks
96+15	95+87	*Not Required*	Ground or Pile-Anchored Prestressed Concrete Sheet Pile Wall
95+87	94+45	Cantilevered Prestressed Concrete Sheet Pile Wall (to Support Ramp)	
94+45	88+01	Deadman-Anchored Prestressed Concrete Sheet Pile Wall	T-WALL
88+01	82+22	Cantilevered Prestressed Concrete Sheet Pile Wall	T-WALL
82+22	71+04		*Not Required*

Typical wall sections illustrating the revised design concept are shown in Figures 7 through 10 below.

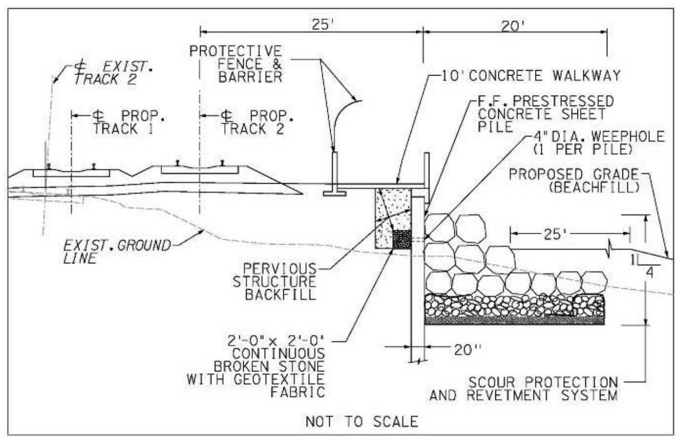

Fig. 7. Typical Single Wall Cantilever Section, Sta. 71+04 to 82+22

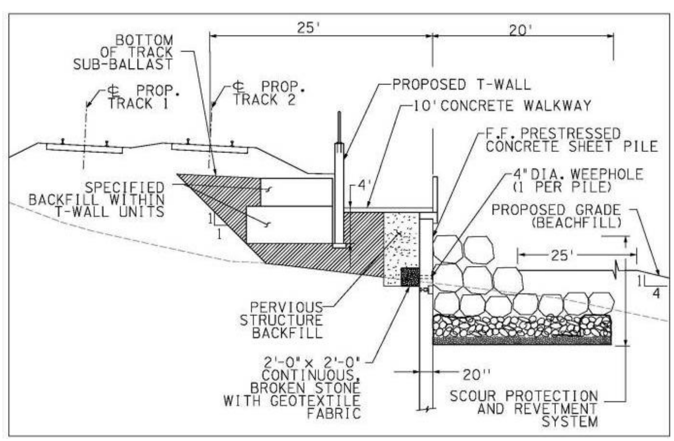

Fig. 8. Typical Two-Tier Cantilever Wall Section, Sta. 82+22 to 88+01

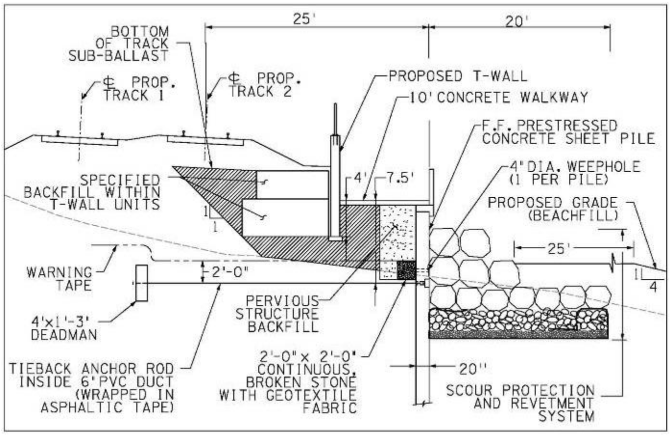

Fig. 9. Typical Two-Tier Wall Section with Deadman Anchor, Sta. 88+01 to 94+45

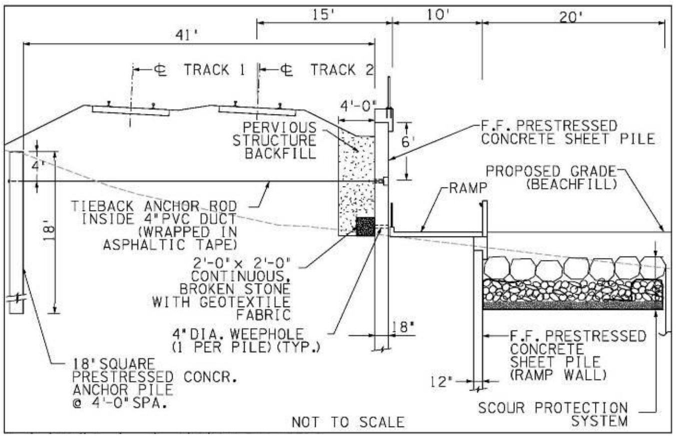

Fig. 10. Typical Two-Tier Wall Section with Anchor Pile, Sta. 94+45 to 95+87

The original wall design utilized a combination of ground anchors and pile anchors to provide lateral restraint for the anchored portion of the prestressed concrete sheet pile wall. The new alignment and configuration for the concrete sheet pile wall, now further away from the existing active tracks, allowed more flexibility in choosing an anchor system. A continuous concrete deadman system was selected to support the anchored portion of the wall along almost its entire length, from Station 88+01 to 94+45. A deadman system was not feasible with the original design, because with the anchors at a shallower depth relative to the final embankment grade, the system could not develop sufficient passive resistance to resist the required anchor loads. Additionally, much of the deadman alignment would have been too close to the active tracks to construct without using temporary sheeting to support the tracks, which would have made installation of this anchor system cost prohibitive.

With the reduced height and corresponding top elevation of the concrete sheet pile retaining wall, the anchors were now at a greater depth below final grade than the original design. This increased embedment depth allowed the deadman system to attain higher design capacities, making it technically feasible for support of the wall. Also, by moving the wall further away from the existing tracks, the concerns about needing temporary excavation support for installation of the deadman system were eliminated, making the deadman system the most economical of the anchor systems evaluated.

As shown in Figure 10, west of Station 94+45, the T-WALL® ends and the concrete walkway ramp down to the beach level begins. To accommodate the ramp, the anchored concrete sheet pile wall alignment is stepped 10 feet towards the tracks, where it becomes a full-height wall, to allow it to provide primary support of the tracks. The ramp for the walkway is then supported by a cantilever concrete sheet pile wall at a 10-foot offset in front of the anchored wall, placing it in line with the anchored concrete sheet pile wall to the west. The cantilever sheet pile wall supporting the ramp decreases in height as the ramp transitions from walkway level down to beach level.

The section of anchored wall adjacent to the walkway ramp is higher in overall height than the anchored wall section within the two-tier wall system. As a result, the anchors at this east end of the wall are closer to finished grade at the top of the embankment, and it is no longer feasible to use a concrete deadman anchor system in this area, because the deadman cannot develop sufficient capacity with the shallower embedment. For this section of the wall the concrete sheet piles were supported using a combination of driven pile anchors and inclined ground anchors. The pile anchors consist of 18-inch square prestressed concrete piles, 20 feet in length, offset 41 feet from the rear face of the sheet pile wall. Ground anchors were used for panels at, and directly adjacent to, where two new catenary pole foundations fall on top of this section of concrete sheet pile wall. Supporting the catenary poles on the wall panels puts additional loading on the wall, resulting in larger required anchor forces which exceed the deflection-limited capacity of the anchor piles. As a result, ground anchors were utilized to obtain greater allowable capacities and to avoid interference with the opposing catenary pole foundations on the north side of the tracks.

The last five concrete sheet pile wall panels in the wall (between Station 95+95 and 96+15), at the end of the walkway ramp, are anchored directly to the northern wingwall of the west bridge abutment. To accommodate the final grading on the north side of the abutment, the northern wingwall is longer than the southern wingwall. The length of the southern wingwall was minimized, because the adjoining concrete sheet pile wall is much cheaper to construct on a lineal foot basis than the cast-in-place wingwall. With the northern wingwall creating as an obstruction for installation of other anchor types, the simplest approach to anchoring the remaining wall panels was to tie them directly to the northern wingwall.

To provide the Overlook users on the new elevated walkway section to access the beach, similar to that at the existing timber boardwalk, a total of three stairways and one handicapped-accessible ramp (in addition to the ramp at the east end of the wall) were incorporated along the length of the new wall. The stairways and ramp were located at the front face of, and parallel to, the front face of the concrete sheet pile wall, tying into the walkway above at overlook points created by bumping the wall out another 10 feet.

In an effort to preserve some aspects of the original elevated boardwalk structure, the railing system, benches and commemorative plaques from the original structure were saved for and reuse along the length of the new elevated concrete walkway.

Revised Scour Protection Configuration

As shown on the typical sections, the final two-tier wall system incorporated a slightly revised configuration of the scour protection system at the front face of the wall. With the lower overall concrete sheet pile height, and corresponding reduction in required embedment depth, the width of the scour protection system in front of the wall was reduced from 25 feet to 20 feet. Additionally, two layers of large, 6,800-pound revetment stones were added just in front of the wall, above the armor stones for the scour protection system. These revetment stones serve to dissipate the energy of waves breaking at the front face of the wall, and help to prevent overtopping of the sheet pile wall by breaking waves during extreme storm events. These breaking waves could otherwise create significant hydrodynamic impact loads on the walkway, railing, and face of the T-WALL®, leading to potentially accelerated deterioration of these structures. The effects of the scour protection system on the design of the retaining wall are discussed in the following sections.

Photo 4. Installation of Scour Protection System

Beach Replenishment

By offsetting the railroad alignment as much as 58 feet closer to the Bay, up to 27 feet of the existing beach was being displaced by the new embankment and the west approach retaining wall system supporting it. As a result, very little usable beach area would remain along some portions of the wall at high tide. Additionally, there was some risk that the sand layer blanketing the scour protection system in front of the wall could be washed away during storm events, essentially eliminating the sand beach altogether in some areas during high tides.

To address these issues, a beach replenishment system was incorporated into the project, including approximately 76,000 cubic yards of imported sand placed along the roughly 2,500-foot-long beach. This will provide 3 feet of sand cover over the top of the scour protection system armor layer in front of the wall, and is designed to result in a final target beach width of 25 feet after equilibrium is reached. To complement the beach replenishment effort, a terminal groin is being constructed at the east end of the beach, close to the river channel, that will prevent eastward longshore transport mechanisms from washing sand into the river channel. The terminal groin will be a rubble-mound structure with a layer of armor stone protecting it, and will project approximately 180 feet out into the bay from the shoreline. Additional details regarding the beach replenishment can be found in Weggel et al (2011).

Photo 5. Placement of Sand for Beach Replenishment

Advance Probing and Removal of Obstructions

In an effort to avoid complications from subsurface obstructions during installation of the concrete sheet pile wall panels, the contractor was required to drill a probe hole at each sheet pile wall panel location prior to starting installation of the wall. In this manner, the presence of boulders or cobbles which could affect installation of the piles could be detected ahead of time. Where potential shallow obstructions, less than 10 feet below existing grade, were detected, they were specified for overexcavation using conventional excavation methods. Where potential deep obstructions were encountered, predrilling was specified. Installation of the concrete sheet pile panels, which varied in thickness from 12 to 24 inches depending on their location within the wall system, was then performed primarily by jetting the panels into place.

Wall Design Approach

As shown in Figure 9, the two-tier anchored wall system includes a prefabricated modular concrete T-WALL® as the secondary wall, with an anchored prestressed concrete sheet pile wall as the primary wall. The secondary wall was designed to directly carry the dead and live loads from the realigned Tracks 1 and 2, while the anchored sheet pile wall was designed to support the concrete walkway and resultant loads from the secondary wall. As a result, the secondary wall was designed first, and once the forces on this wall were determined, the design of the primary prestressed concrete sheet pile wall structure was advanced taking into account the loads applied by the secondary wall.

Figure 11 shows the anchored two-tier wall system layout towards the eastern end of the west approach wall, between approximately Station 90+50 and 91+50. This section of the wall will be discussed to illustrate the design procedures used elsewhere for the western approach wall.

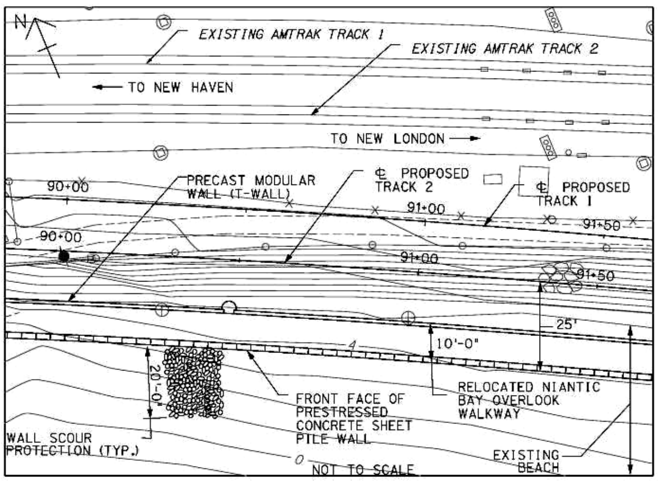

Fig. 11. Wall Overview – Anchored Wall Section, Sta. 90+50 to 91+50

The exposed height of the precast concrete sheet pile for this section averaged about 10.6 feet, measured from the top of armor stone in the scour protection system, to the walkway level at the top of the wall. This wall height remained constant over the length of the wall from Station 71+04 to 94+45, as the concrete walkway elevation and top of scour protection system remained constant throughout this range. This design height assumed that any sand cover over the top of the scour protection system could eventually be washed away during storm events.

The exposed height of the prefabricated modular T-WALL® at in this area was about 9.3 feet, with an additional embedment depth of 4 feet below the finished concrete walkway elevation to protect the toe of the wall from frost heave, while still maintaining and adequate clearance over the deadman tie rod running beneath it. The exposed height of the T-WALL® varied between approximately 4.5 feet at its west end (Station 82+22) up to 10.2 feet at its east end (Station 94+45), paralleling the change in vertical alignment of the new Track 2.

During the wall design, different construction scenarios were considered to determine the most critical design condition for the concrete sheet pile wall, which was then used to set the embedment depth for the wall panels. A total of three design cases were considered; two construction cases, and one post-construction (final) case. The design cases are discussed below.

Construction Case 1. This case considers the excavation taking place in front of the wall to install the scour protection system prior to any embankment fill being placed behind the wall. Thus, the effective exposed height of the wall is from the bottom of the excavation for the scour support to the existing grade level at the back of the wall. In this case, the wall has to support the existing embankment behind the wall, along with an additional 250 psf of construction live load surcharge for equipment operating behind the wall and some live load surcharge from the existing tracks. No anchors have been installed at this point in the wall construction, so it acts as a cantilever wall. A typical sketch showing details of this design case is shown in Figure 12.

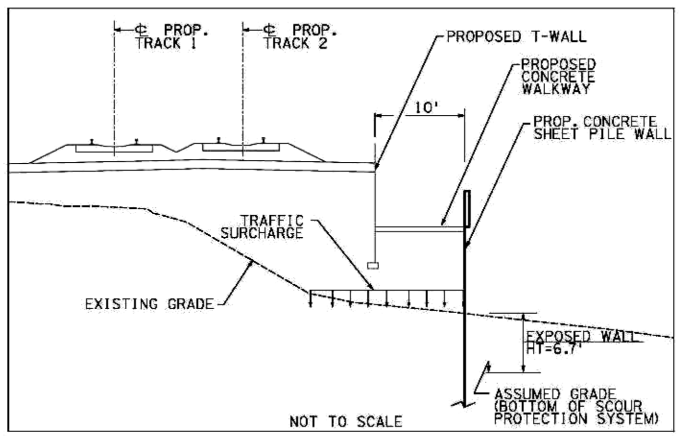

Fig. 12. Wall Construction Case 1, Sta. 90+50 to 91+50

Construction Case 2. In this case, the scour protection system has been installed in front of the wall, and now fill is being placed behind the wall up to the anchor tie rod level for the wall (7.5 feet below the top of wall).

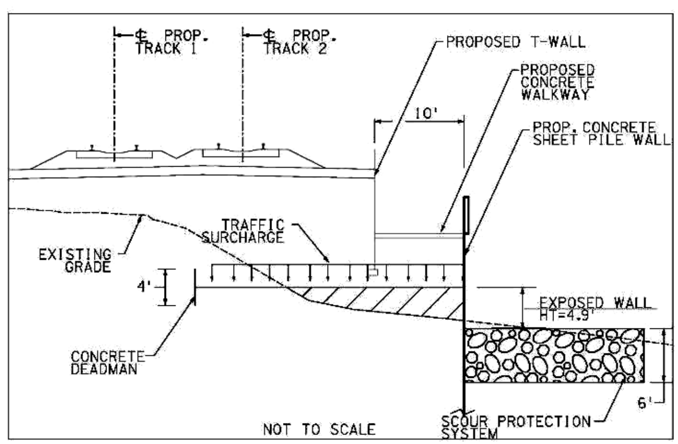

Fig. 13. Wall Construction Case 2, Sta. 90+50 to 91+50

A typical section illustrating this case is shown in Figure 13. Again, a construction live load surcharge of 250 psf was assumed. The wall also acts as a cantilever for this case. The calculation of passive resistance in front of the wall ignored any sand that might be in place over the top of the scour protection armor stone.

Post-Construction (Final) Case. This design case considered the final two-tier wall system configuration, with the anchors and scour protection in place, and all appropriate live and dead loads applied. The loads included train live loads from both new tracks, resultant loads from the base of the T-WALL®, live loads on the Overlook walkway, earth pressures from the wall backfill, and 3 feet of unbalanced hydrostatic pressure above the weep hole level in the sheet pile panels. A typical sketch showing details of the design case analyzed is shown in Figure 14 below.

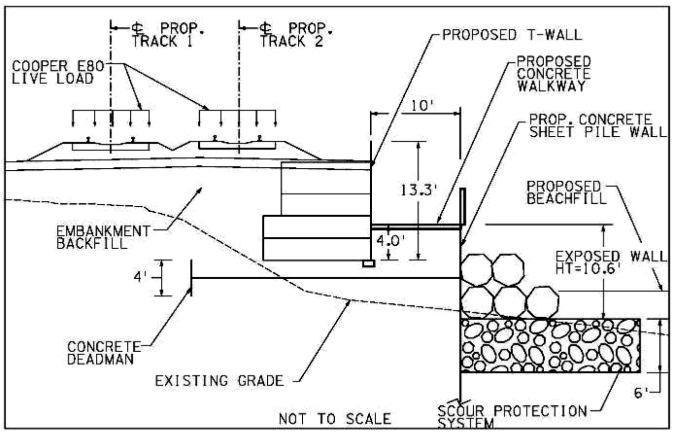

Fig. 14. Final Wall Configuration, Sta. 90+50 to 91+50

Where appropriate, loading from the new catenary structures also had to be considered. The centerline of the catenary structure foundations fell slightly behind the facing panels of the T-WALL® modules. Since it would be difficult to design the wall modules to directly accommodate the catenary structure loadings, the catenary poles were founded on drilled shaft foundations located so that the outer edge of foundation would fall in line with the front face of the wall modules. At the catenary pole locations, a gap was left between two adjacent sets of T-WALL® modules to make room for the catenary foundation. The exposed portion of the catenary pole foundation, extending from the walkway level up to the top of the secondary wall, would be cast as a rectangular section so that it would blend in with the front face of the T-WALL®.


Photo 6. Wall Construction with Catenary Pole Foundation

Wall Analyses for Construction Cases 1 & 2. As shown in Figures 12 and 13, different exposed wall heights and backfill levels were analyzed to determine which case would result in the largest required wall embedment depth. Rankine's earth pressure theory was used to determine the active earth pressures behind the wall and passive earth pressures in front of the wall. In determining the passive earth pressures at the face of the wall, all sand cover over the top of the armor stone was ignored, and no contribution from the revetment stones was included either. Horizontal pressures on the back of the wall resulting from construction live load were determined using the Boussinesq elastic solution for a rigid wall condition. The required wall embedment depth was determined using a horizontal static equilibrium analysis with a factor of safety of 1.5 applied to reduce the passive earth pressure coefficients.

Wall Analyses for Final (Post-Construction) Case. The prefabricated modular T-WALL® was analyzed for external stability, including checks of sliding, overturning, and bearing capacity. (Internal stability of the T-WALL® system is performed by The Neel Company when the final shop drawings are prepared for the wall.) The wall design was performed using the allowable stress design (ASD) method in accordance with the AREMA and AASHTO design standards. ASD factors of safety of 1.5, 2.0, and 3.0, were used for sliding, overturning, and bearing capacity, respectively. Coulomb's earth pressure theory was used to determine the active earth pressures behind the wall. Passive pressures at the face of the wall were ignored for the sliding analysis. Horizontal pressures from the twin Cooper E80 train live loads were estimated at the back the T-WALL stems using Boussinesq's solution for a strip load parallel to a rigid wall. Each Cooper E80 train load was modeled as an 8.5-foot-wide strip load with a uniform intensity of 1,882 psf. Based on the results of external stability analyses, sliding controlled the T-WALL® design.

The design of the anchored walls for the two-tier wall system was performed using the methodology provided in FHWA's *Geotechnical Engineering Circular No. 4, Ground Anchors* *and Anchored Systems* (1999). An apparent earth pressure diagram was developed for the wall considering a final exposed wall height of 10.6 feet from the top of the scour protection armor stone to the top of the concrete walkway, plus an additional 2 feet in case the upper layer of armor stone was not in intimate contact with the front face of the wall. Below the bottom of the apparent earth pressure diagram at the back of the wall, active and passive earth pressure loads on the wall were taken into account. To help optimize the moments in the concrete sheet piles, the anchor tie rods were located at a depth of 7.5 feet below the top of the sheet pile wall, which placed them at about 3.5 feet below the bottom of the lowest T-WALL® modules in the secondary wall. Other loadings included the twin Cooper E80 train loads, loads from the secondary wall, walkway live loads, and unbalanced hydrostatic pressure. The wall embedment depth was determined by calculating a reaction force from the upper portion of the wall at the assumed point of wall fixity, and then performing a static equilibrium analysis of forces below that point using a minimum factor of safety of 1.5 applied to the passive earth pressure coefficients. As with the construction cases, no passive earth pressure contribution was considered from either the sand cover or revetment stones over the top layer of armor stones in the scour protection system.

The required anchor forces were determined from the apparent earth pressure diagrams established for analysis. One anchor tie rod was provided for each 4-foot-wide concrete sheet pile wall panel, and so the anchor force calculated on a per-foot basis along the wall was multiplied by four to obtain the total force to be resisted by each tie rod. In addition to this, the calculated maximum anchor rod force was increased by a factor of 1.2 for design as required by AREMA.

The results of the anchored sheet pile wall embedment analyses are summarized in Table 4 shown below.

Table 4. Summary of Wall Embedment Analyses,
Sta. 90+50 to 91+50

Design Case	Grade Differential, Wall Back to Front (ft)	Min. Wall Embedment Depth (ft)	Max. Pile Tip Elev. (ft)
Construction Case 1	6.7	13.1	-16.1
Construction Case 2	4.9	8.5	-5.5
Final Case	10.6	16.0	-13.0

As shown in the table above, analysis of Construction Case 1 resulted in the most critical wall embedment depth. Based on the controlling pile tip elevation of -16.1 feet determined from the analyses, a design pile tip elevation of -17.0 feet was ultimately selected for this section of the wall. To minimize small variations in the length and tip elevations of the concrete sheet piles, the same design pile tip elevation was used for adjacent wall sections of similar overall height.

Deadman Anchor and Tie Rod Design. The deadman anchor system had to be sized to resist the design anchor forces calculated in the analyses described above. To maximize the capacity available from the deadman system, the deadman had to be placed far enough away from the back of the sheet pile wall that any overlap between the passive earth pressure zone of the deadman and the active earth pressure zone behind the sheet pile wall would be minimized. An offset of 30 feet between the front face of the deadman and back face of the sheet pile wall was selected to minimize this overlap. With anchor tie rods closely spaced at 4-foot centers along the length of the wall, the deadman system was designed as a continuous reinforced concrete panel to maximize its effectiveness and to simplify its construction.

The vertical dimension of the deadman system was selected by choosing an adequate size to provide the required anchor capacity, while minimizing the size of the passive earth pressure zone to keep the anchor tie rod length as short as possible. This would keep the deadman as far as possible away from the existing tracks, minimizing construction impacts to the tracks. Tie rod lengths and impacts to the existing embankment and tracks could be minimized by placing the deadman at higher elevation; however, by reducing the overburden stress at the deadman level, the capacity of the deadman is also reduced. Additionally, the deadman had to be placed deep enough so that a reasonable buffer could be maintained between the bottom of the T-WALL® system and the anchor tie rods passing beneath it.

The concrete deadman design was performed using the general design methodology presented by Dismuke (1991). Based on this methodology, the ultimate deadman capacity was determined from the difference between the estimated passive and active earth pressure resultants at the front and back faces of the concrete deadman block, respectively. A deadman block height of 4 feet was selected to provide the necessary allowable anchor capacity utilizing a factor of safety of 2.0. The allowable deadman capacity per anchor tie rod using this configuration was approximately 66 kips, while the required anchor force was approximately 59 kips.

Photo 7. Continuous Deadman Construction

The anchor tie rod was designed as a 2-inch diameter Grade 55 steel rod. During construction, an alternate of a 1.75-inch diameter Grade 75 rod of high strength steel was approved. To provide long-term corrosion protection to the anchor rod, a hot-dipped galvanized rod and anchorage hardware was specified. Additionally, the length of the anchor rod between the deadman and wall was wrapped in asphaltic tape, and placed inside a 4-inch diameter Schedule 40 PVC pipe. Each end of the PVC pipe was then sealed with expanding spray foam sealant to prevent soil and water intrusion into the PVC pipe.

Photo 8. Concrete Deadman with Anchor Rod Protection Cap

While aiding in corrosion protection of the anchor rod, the primary purpose of the PVC pipe is to isolate the anchor rods from embankment settlements occurring after the installation of the anchor rods had been completed. Including construction of the T-WALL®, as much as 17 feet of fill would be placed above the anchor rod level in some locations. Any short-term or long-term settlements resulting from this fill placement can be accommodated by allowing the pipe to move downward with the overlying fill material, while the anchor rod remains in the same location. The concept is to place the PVC pipe on a compacted lift of material at the proposed anchor rod elevation, and then inserting the anchor rod into PVC pipe and letting it rest on the bottom of the pipe. Filling then proceeds over the pipe and enclosed anchor rod. If settlement occurs, no additional stresses are placed on the anchor rod until the PVC pipe moves downward enough that the crown of the pipe reaches the top of the anchor rod. The PVC pipe can be sized such that the amount of relative movement that the pipe can accommodate is greater than the settlement expected subsequent to the anchor installation.

<u>Global Stability</u>. The global stability of the two-tier wall configuration was checked using Bishop's Method in the SLOPE/W computer program. The analyses assumed simultaneous Cooper E-80 train live loads at both new track locations behind the wall. The analyses indicated a minimum factor of safety of 1.73 against a global stability failure under static loading. For seismic loading, a horizontal seismic coefficient of 0.08 was used in the analysis which resulted in a minimum factor of safety of 1.5. Considering 100-year storm surge conditions, the static and seismic analysis cases result in estimated minimum factors of safety of 1.7 and 1.48, respectively.

CONSTRUCTION

The construction contract was awarded in January of 2010, and construction of the west approach retaining wall began during the summer of 2010.

Photo 9. T-WALL Construction and Walkway Subgrade

While the construction of the wall and the installation of the scour protection system was still in progress, the remnants of Hurricane Irene made landfall on the northern shore of Long Island Sound on August 28, 2011. During the peak of the storm, the tops of the breaking waves from Niantic Bay were just above the top of the prestressed concrete wall panels, at about EL. 11.9. Inspection of the site following the storm revealed that the wall system weathered the storm very well, despite not having the sheet pile wall coping completed and not having the revetment stones for the scour protection system in place.

Photo 10. Completed Two-Tier Wall and Elevated Walkway

The construction of the west approach wall was completed during the summer of 2012, and the new bridge and approaches were opened to train traffic on September 8, 2012.

The entire project is expected to be completed in the spring of 2013.

Photo 11. Wall with Close-up of Revetment Stones

Photo 12. West Approach Wall, Ramp and Terminal Groin

CONCLUSION

This project provides an example of how transportation projects can evolve throughout the course of their design phase, particularly when stakeholders are actively engaged in the process. The two-tier wall system provided a creative solution to support Amtrak's realigned western approach tracks leading up to the new Niantic River Bridge, while at the same time incorporating the replacement of an elevated recreational walkway and a scour protection system designed to withstand 100-year storm events .

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- Town of East Lyme
- Connecticut Department of Environmental Protection
- URS Corporation, construction manager for the project
- Cianbro/Middlesex Joint Venture Team, general contractor for the project

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