Design of A-Walls for Stabilization of Slopes and Embankments in Soft Soils

Jesús E. Gómez
Schnabel Engineering, West Chester, PA

Carlos Englert
Schnabel Engineering, Rockville, MD

Helen D. Robinson
Schnabel Engineering, West Chester, PA

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Dr. Jesús E. Gómez, P.E., D.GE
Schnabel Engineering
West Chester, PA-USA

Carlos Englert, M.S.
Schnabel Engineering
Rockville, MD-USA

Helen D. Robinson, P.E.
Schnabel Engineering
West Chester, PA-USA

ABSTRACT

A-Wall systems are a combination of deep foundations and, in some cases, tiebacks used to provide lateral support to an unstable ground mass. Determination of the lateral and vertical forces acting on an A-Wall system can be a complex endeavor. As the unstable soil mass tends to move past and through the A-Wall system, forces are generated between the A-Wall elements and the soil. These forces provide support to the ground mass. If the A-Wall is correctly designed, the forces will increase as the soil moves until a maximum is attained at which ground movement ceases and the system reaches equilibrium.

Design of an A-Wall thus requires Soil-Structure Interaction (SSI) analyses that provide a solution that meets force and moment equilibrium as well as compatibility of displacements. The individual elements of the A-Wall are designed based on a structural analysis utilizing the estimated forces.

This paper describes the philosophy of design of A-Walls. It also contains a detailed description of design steps based on the use of a commercially available computer program that allows determination of soil forces against a deep foundation element installed through a mass of moving soil. An iterative method is presented to find a solution to the A-Wall problem that meets equilibrium and compatibility and considers material nonlinearity of soils and A-Wall components, as well as geometric nonlinearity of the deep foundation elements. Two case histories are presented.

INTRODUCTION

One of the more exciting applications for deep foundations is the stabilization of slopes. Micropiles, caissons, drilled or driven piles, and even tiebacks may be utilized in an A-Wall system to provide lateral support to an unstable ground mass. Determination of the lateral and vertical forces acting on an A-Wall system can be a complex endeavor. As the unstable soil mass tends to move past and through the A-Wall system, forces are generated between the A-Wall elements and the soil. These forces provide support to the ground mass and may be a combination of shear, bending, tension and compression.

This paper presents two case histories where A-Walls were used successfully for stabilization of slopes or embankments. The paper contains a summary of the procedure for practical design of A-Walls, and provides a list of useful published references available in the literature. It is important to note that there are no published guidelines for design of A-Walls. Therefore, the designer must apply judgment and the experience gained from previous projects.

HIGH STREET WALL REHABILITATION, PORT DEPOSIT, MARYLAND

Port Deposit is a small historic town in the state of Maryland, located on the Susquehanna River bank. It was once a hub of trade between New York State and Washington D.C., and was very famous for its granite quarries.

High Street rides along a steep granite slope. The road was built with fill retained by several masonry walls along the street. These walls displayed several signs of mass soil movement, and one of them partially collapsed, leaving the road to several homes blocked. The partial wall collapse can be seen in Figure 1.

Fig. *1. Collapsed Stone Wall at Port Deposit, Maryland*

Inclinometer and tiltmeter readings showed that fill material was sliding on top of the granite bedrock. Slope stability analyses confirmed this mode of failure as the most probable. Slope stability analyses were performed to determine the soil thrust and the required stabilizing force to increase the factor of safety to an acceptable level.

The lack of space and the presence of houses along the slope were decisive factors in selecting a micropile A-Wall to stabilize a portion of the road. Figure 2 depicts the key features of the A-Wall.

Design of the wall consisted of the following steps:

1. Determine the required stabilizing force for the desired factor of safety against sliding of soils over their contact with bedrock.
2. Layout preliminary dimensioning of the micropiles and spacing following Pearlman et al. (1992).
3. Create a structural frame model of the A-Wall in structural design computer program such as SAP2000.
4. Apply the required stabilizing force as a distributed load over the micropiles.
5. Verify and adjust micropile design based on bending moments and axial loads determined from frame analysis.
6. Iterate using structural software as needed with new micropile dimensions.
7. Design cap beam according to axial and shear loads from micropiles.

It is important to note that this case was relatively simple because there was one potential sliding surface, and because the soil would likely tend to move over its contact with bedrock with little distortion as suggested by the inclinometer data. Therefore, only one sliding surface needed to be considered and the required stabilizing force could be assumed to act as a uniform load over the micropile length. In reality, a triangular distribution could have been more appropriate but a uniform distribution was a more conservative assumption.

Development of the structural model for use in structural analysis software required some assumptions about the behavior of the system. The piles were assumed fixed at their contact with bedrock. This is a reasonable assumption as the micropiles were embedded several feet into bedrock. They were also assumed to rotate rigidly at the top thus considering their embedment into the pile cap. The pile cap was not modeled explicitly.

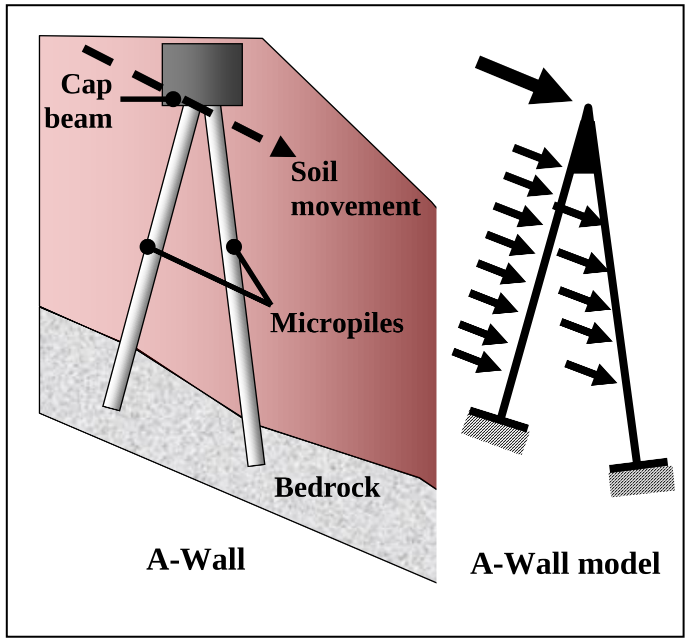

Fig. 2. Port Deposit A-Wall and structural model for analysis

In the analysis, a portion of the stabilizing force was applied directly to the cap beam. This acknowledges the fact that the continuous beam receives direct loading from the soil as it tends to move. The effect of this load is mostly axial compression and tension in the leading and trailing micropiles, respectively. In a case such as Port Deposit, where the micropiles are embedded into rock, the available axial capacity of the micropiles is significant. It is important not overestimate the load on the cap beam and to make sure it does not exceed a conservative estimate of passive resistance of the soil.

Finally, the spacing of the micropiles must be such that arching of the soil develops. Otherwise, the stabilizing force would not be realized and the soil movement could still occur between the micropiles. The procedure given by Pearlman et al. (1992) includes a determination of the maximum spacing between supporting elements. In general, the authors have found that the spacing between the micropiles should not exceed approximately three times the micropile diameter. This spacing should be measured between consecutive micropiles, i.e. between one leading micropile and the adjacent trailing micropile, and not between micropiles of the same row. At the present time and as described subsequently in this paper, the available computer software for analysis of deep foundations subject to soil movement allows implicit consideration of arching during design without the need for a separate check on the spacing.

Figures 3 through 5 are views during construction of the A-Wall. The cap beam was constructed by first leaving block-outs within the beam for subsequent micropile installation. The reinforcement of the cap beam is often minimal as bending moments, shear, and torsion are not significant for the typical beam section dimensions. To date, no significant movement of the street and/or slope above the A-wall have been observed (Englert, et al. 2007).

Fig. 3. Cap beam and block-outs for installation of micropiles

Fig. 4. Installation of micropiles through cap beam

Fig. 5. Cap beam reinforcement at Port Deposit

THE JEFFERSON MEMORIAL SEAWALL

The Jefferson Memorial is located in the West Potomac Park Historic District and is part of the National Mall & Memorial Parks (NAMA). It was constructed from 1939 to 1943 as a monument to the third President of the United States, Thomas Jefferson. Figure 6 shows the location of the Jefferson Memorial. Figure 7 is an aerial view of the Jefferson Memorial building and surrounding grounds

At the project site, Pleistocene Age terrace soils were extensively eroded by the Potomac River down to bedrock, and were replaced with recent, soft alluvial deposits. Significant filling of this area took place early in the 20th Century during reclamation of the West Potomac Park. The planned location for the Memorial within the park required reconfiguration of the existing shoreline along the Tidal Basin. Figure 8 shows the original and modified shoreline. Material was dredged from the area labeled as "Cut" in the figure on the northeast side of the site and used as backfill in the northwest side. While the Jefferson Memorial building and a portion of the surrounding ring walls were constructed on steel piles extending to bedrock, the Ashlar Seawall along the reconfigured shoreline was built on timber piles bearing on relatively soft soils, possibly due to wartime scarcity of steel toward the end of construction.

Fills up to 30 to 40 feet deep were placed over the soft, highly compressible alluvial soils extending down to a depth of 87 to 102 ft below the North Plaza, where bedrock is encountered (EYP 1992). Since its construction, and as expected by its designers, the Jefferson Memorial grounds have sustained noticeable ground settlement. The plaza settled and showed considerable damage in the years following the Memorial's construction. It is estimated that the North Plaza may have settled 3 to 3.5 ft. The main structure of the Memorial, however, did not sustain significant damage due to its foundation elements extending to rock.

Fig. 6. Location of Jefferson Memorial in Washington, D.C.

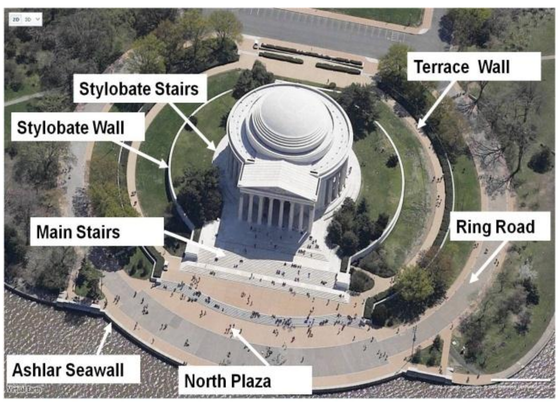

Fig. 7. Aerial view of the Jefferson Memorial

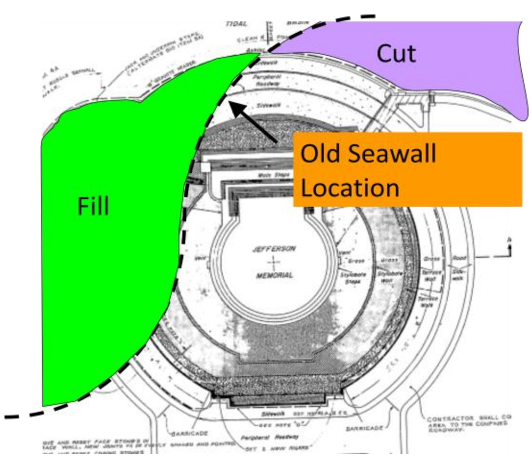

Fig. 8. Reconfiguration of the shoreline during construction of the Jefferson Memorial

Lateral movement of the North Plaza also occurred following construction of the Memorial. The Memorial stairs were buttressed as part of the North Plaza reconstruction project to correct lateral displacement that had occurred up to that date (Storch 1965).

The settlement of the North Plaza was a result of the compression of the soft alluvial soils under the weight of the additional fill placed on the western half of the Plaza. Lateral movement of the North Plaza was likely due to distortion of the soil mass as it compressed near the edge of the embankment.

Figure 9 contains the results of optical surveys of the Ashlar Seawall since its construction. The data shows that settlement of the seawall started immediately after construction and that it reached approximately 6 inches on its westernmost end. The data also shows that the settlement increased consistently along the wall starting at the original shoreline and increasing toward the west, which is consistent with the larger thickness of the most recent fill placed in the western half of the North Plaza area.

The rate of settlement of the seawall gradually decreased until it became almost zero after the 1960s. The North Plaza was reconstructed in 1969-1970 as a structural slab on grade beams and HP piles extending to bedrock as depicted in Figure 10.

In February 2006, settlement of the Ashlar Seawall accelerated reaching a rate of approximately 1 inch/year during the 2006-2008 period. Monitoring data confirmed that the main structure of the Jefferson Memorial and the North Plaza on piles were not undergoing appreciable vertical movement, while surrounding areas were undergoing settlement at a rate consistent with that of the seawall. The monitoring data also showed that the North Plaza was undergoing lateral movement toward the Tidal Basin. Lateral movement was registered to a depth of approximately 60 to 70 ft below the North Plaza according to inclinometers installed soon after movements were noticed (see Figure 11).

Piezometer readings revealed that pore pressures within the deep alluvium were significantly less than those expected in a hydrostatic condition. Furthermore, piezometric measurements indicated that the interface with bedrock acted as a drainage boundary. This suggested that a drop in the piezometric head at the rock boundary had occurred recently and that it may have induced consolidation and associated settlement of the soils as well as lateral movement of the North Plaza.

After careful analyses of various alternatives, the project team decided to demolish and reconstruct the Ashlar seawall on an A-Wall consisting of vertical caissons and battered pipe piles connected together by the new seawall. The scheme provides resistance to future vertical and lateral movement of the North Plaza and the new seawall (Gómez, et al. 2011).

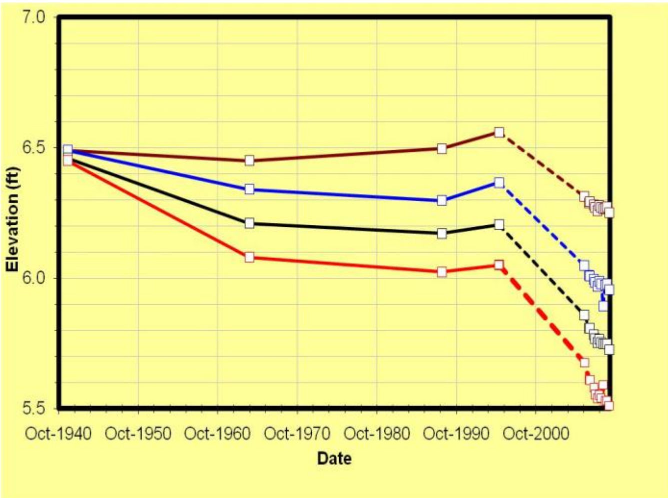

Fig. 9. Historical settlement of points along the top of the Ashlar Seawall. The red and brown data correspond to the westernmost and easternmost ends of the seawall, respectively

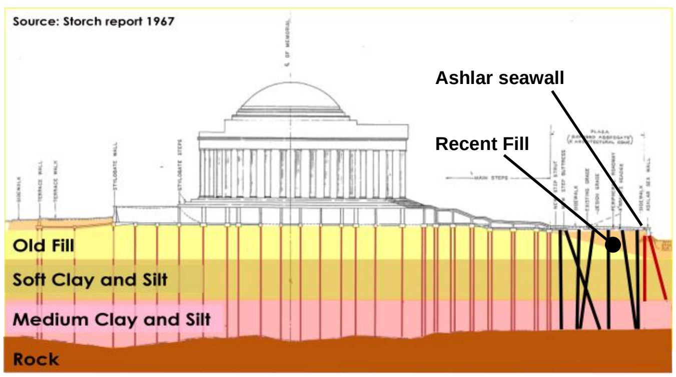

Fig. 10. North-South cross section depicting foundation depths and stratigraphy (adapted from Storch 1965)

Figure 12 is a depiction of the adopted stabilization solution. It also shows the forces acting on the A-Wall. Immediately after construction, the system is only subject to the weight of the seawall, which is absorbed by the vertical caissons. Over time, lateral and vertical movement of the surrounding soils develops. This generates downdrag as well as lateral forces on the caissons and battered piles. Due to the presence of the caissons, it is anticipated that the lateral loads on the piles would be relatively small due to a shadowing effect.

The lateral loads on the system induce bending of the caissons as well as axial loads in the caissons and piles. It is estimated that, over time, if the tendency for lateral soil movement continues, significant tension will develop in the caissons and compression in the battered pipe piles.

In addition, the existing North Plaza piles are subjected to lateral loads and downdrag as well. The lateral loads are transferred through the North Plaza to the new seawall and generate additional axial loads on the caisson and pipe piles without significant bending. Earth pressures from the backfill of the seawall would also develop and generate additional bending and axial loads that are relatively minor.

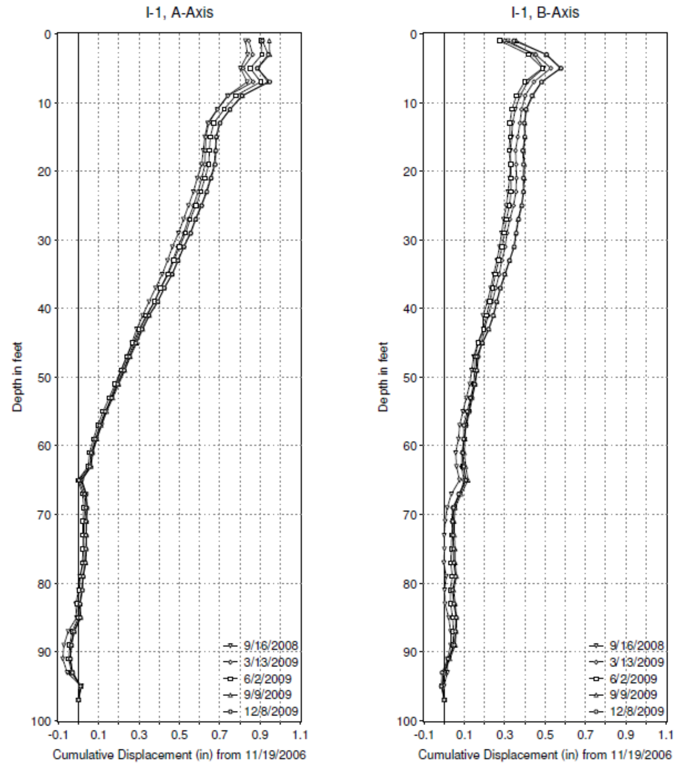

Fig. 11. Inclinometer data collected near the northwest corner of the North Plaza (Gómez, et al. 2011)

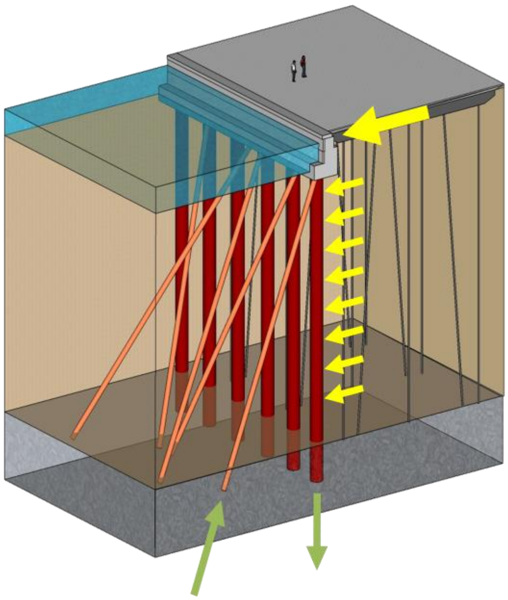

Fig. 12. Depiction of Jefferson Memorial A-Wall and system forces (Gómez, et al. 2011)

Figure 13 is a detail of the newly constructed Ashlar seawall, which also acts as the cap beam connecting drilled shafts and battered pipe piles. Post-construction survey readings of the North Plaza and seawall show that vertical and lateral movements have been arrested.

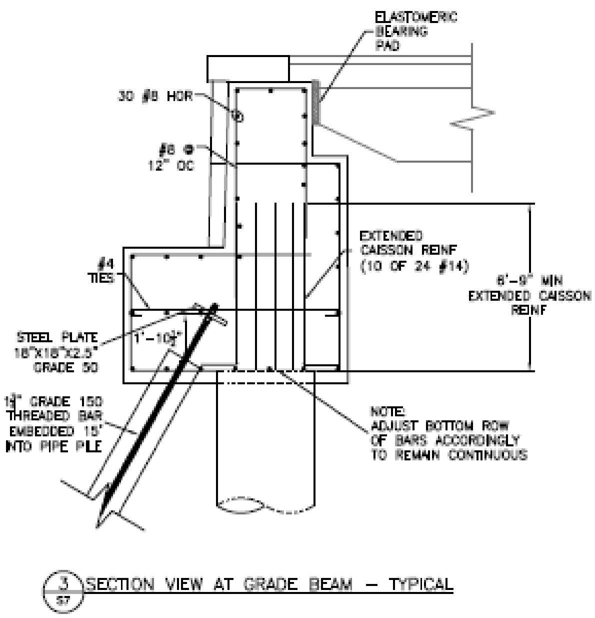

Fig. 13. Detail of new Ashlar seawall also acting as cap beam for A-Wall system (Gómez, et al. 2011)

JEFFERSON MEMORIAL A-WALL DESIGN PROCESS

Vertical loads on the Jefferson Memorial seawall foundation are due to the weight of the wall and downdrag; and are relatively easy to estimate. However, estimation of the loads induced by the tendency for lateral movement of the soils requires soil structure interaction analyses.

The computer program LPILE Plus Version 6.0 was used extensively for this purpose. The program allows the user to impose a profile of horizontal displacements with depth to the soils surrounding the deep foundation element. Thus, it is possible to determine the deflections and bending moments that develop on a deep foundation element of known fixity conditions at the head as the soil moves horizontally past it. Three dimensional effects and arching are automatically considered by P-Y curves that are selected for the analysis.

However, LPILE only allows analysis of a single foundation element. Therefore, it is not possible to analyze the A-Wall system without an iterative process to ensure compatibility of displacements, bending moments, and forces.

The design was thus performed according to the following steps:

1. Estimate soil loading by hand and develop preliminary design for analysis. This estimation consisted of calculating the passive resistance of the soils surrounding the caissons.
2. Define a scaled horizontal soil movement profile based on the inclinometer data.
3. Model the caisson and pipe pile using LPILE with zero-moment condition at the head.
4. Apply various scaled profiles of soil displacement separately to the caisson and battered pipe pile. The maximum (near-surface) soil displacement of each displacement profile ranged between 3.5 and 10 inches.
5. For each maximum displacement magnitude, caisson or pile head displacement was permitted ranging between 0.25 and 2 inches.
6. Determine the shear force at the head of the caisson and pile for each of the displacement combinations analyzed.
7. The solution was that which satisfied equilibrium of forces at the head of the piles and caissons and compatibility of horizontal displacements.
8. Dimension caissons and piles and establish caisson reinforcement to resist loading.
9. Repeat steps 4 through 8 iteratively.

The analysis was further complicated by the interaction of the A-Wall with the North Plaza. The foundation piles of the North Plaza were also subject to lateral thrust from the soil that would ultimately be transferred to the A-Wall. The lateral displacement that had occurred was estimated based on the openings of the North Plaza joints and was considered in the estimation of forces. The shear force at the head of the piles was estimated for a variety of pile head displacements also using LPILE. The total horizontal load exerted by the North Plaza on the new seawall was estimated as the sum of the shear forces at the head of each of the Plaza piles for each magnitude of head displacement.

The total force exerted by the North Plaza was then added to the A-Wall model for the correct level of displacement to determine additional axial loads on the piles and caissons. The design of the caissons and pipe piles was adjusted to meet the loading determined from this analysis and was subjected to one more numerical analysis iteration.

The final design considered a maximum deflection at the top of the piles and caissons of 1 inch, which resulted in 525 kips of tension in the caissons, and 354 kips of compression in each pipe pile once the maximum anticipated soil movements develop.

DESIGN OF A-WALLS FOR GLOBAL STABILITY

The Jefferson Memorial A-Wall was conceived to control deformations of a structural system subject to movement of the foundation soils. It is a special case in that continuing movement of the soils past the A-Wall is not an issue for the A-Wall itself. However, most stabilization projects of slopes and embankments using A-Walls require that there is no potential for soil movement.

The process to design an A-Wall for slope or embankment stabilization is very similar to the process illustrated in the previous section:

1. Determine the required stabilizing force for a minimum factor of safety against global instability, and determine the maximum slope or embankment movement allowable based on serviceability requirements.
2. Define a scaled horizontal soil movement profile. This profile can be determined using judgment if actual inclinometer data is not available.
3. Model the A-Wall foundation elements using LPILE with zero-moment condition at the head.
4. Apply various scaled profiles of soil displacement separately to the caisson and battered pipe pile.
5. Determine the total soil force on the foundation elements for each magnitude of displacement.
6. Iterate.
7. The A-Wall must safely resist soil movement that induces a total force on the A-Wall equal to the force required for the minimum factor of safety without the soil movement exceeding the serviceability limits imposed

Loehr (2008) describes the process for design of A-Walls for slope stabilization. It is important to note that this process may be complex, especially if there are multiple potential failure surfaces, existing low factors of safety, and tight serviceability limits.

LIMITATIONS OF FINITE ELEMENT ANALYSES

The main reason for not performing finite element analyses of the Jefferson Memorial A-Wall as the primary design tool was that, at the time, it was only possible to perform a two-dimensional analysis that would negate consideration of three-dimensional effects around the caissons and piles as consequence of soil movement. The A-Wall would be analyzed as a continuous system that would be subject to loads that would be unrealistically large. Although a three-dimensional analysis of each foundation would have been possible, it would result in a similar or larger number of iterations.

However, two-dimensional finite element analyses were indeed performed to validate certain aspects of the design. It is possible that with the new analysis elements introduced in certain finite element computer programs, these analyses could be nowadays completed without manual iterations. If so, the design of A-Wall systems, and of combined foundations in general would be simplified significantly.

CLOSURE

A-Walls formed by deep foundation elements are widely used in the United States and abroad for stabilization of slopes and embankments. Even though the A-Wall concept is not recent, there are still no established modern guidelines for their design. The engineer must rely on experience and judgment, as well as on the limited amount of previous published work on the subject.

Design of A-Wall systems poses several difficulties. One is the iterative nature of the computations necessary to obtain force and moment equilibrium as well as compatibility of displacements of the foundation elements. Another difficulty is that the pattern of soil displacement is not known a priori, especially in those cases where instability has not yet developed. Furthermore, the introduction of an A-Wall in an unstable soil mass will modify the pattern of displacement in ways that cannot be predicted accurately using current design procedures.

The relatively recent availability of computer software that allows analysis of deep foundations subject to soil movement is a significant advance for A-Wall design. However, this capability is still not available for analysis of pile groups, which would likely eliminate the need for complex manual iterations that are still necessary.

Two-dimensional finite element analyses are not greatly useful for design of A-Walls because they are implicitly assumed to be continuous. Three-dimensional analyses are too cumbersome and may not capture essential elements of the interaction between the soil and the A-Wall.

The recent introduction of embedded pile elements in finite element software widely used in geotechnical design may become very useful for modeling A-Walls, especially in cases such as the Jefferson Memorial. It would be possible to develop a more comprehensive model of the soil and structure that accounts for the change in the movement and deformation pattern of the soil due to the presence of the A-Wall itself.

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