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George Aristorenas
Weidlinger Associates, Inc., Cambridge, Massachusetts

Minhaj Kirmani
Weidlinger Associates, Inc., Cambridge, Massachusetts

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George Aristorenas, Ph.D., P.E.
Weidlinger Associates, Inc.
Cambridge, MA-USA-02142

Minhaj Kirmani, Ph.D., P.E.
Weidlinger Associates, Inc.
Cambridge, MA-USA-02142

ABSTRACT

One Lincoln Street is one of the most recent buildings constructed in downtown Boston. Its structure consists of a 36-story high-rise building directly connected to a 7-story low-rise building. Its substructure has five levels of underground parking garage to accommodate 900 cars in a city where prime real estate is becoming scarce. The excavation for the underground parking garage was supported by reinforced concrete slurry walls, which also serve as the substructure's permanent walls. The stiffness of the slurry walls, together with the strut and tieback bracing system, minimized movement during excavation, which occurred in close proximity to existing buildings.

Of particular interest is the northwest corner of the excavation, which was supported by an *arched slurry wall*, possessing a shape in plan of a quadrant of a circle with a radius of 50 feet. This paper presents key aspects of the analysis, design, construction and performance of the arched slurry wall. While the other slurry walls in the project were designed to support the 59 foot deep excavation with two levels of bracing, a remarkable feat by itself, the 3 foot thick arched slurry wall was analyzed, designed and constructed to support the excavation with *no* bracing. The analysis consisted of two-dimensional finite element models, modified to include the effects of three-dimensional arch action.

Predicted lateral movement of the wall was minimal, having minor impact to adjacent structures, and measured inclinometer readings favorably support the predicted movements.

INTRODUCTION

A 3-foot thick slurry wall, the shape of which is shown on Fig. 1, envelopes the substructure of One Lincoln Street. The Bedford and Kingston buildings bound the footprint on its northeast and southwest, respectively, and are located as close as five feet from the slurry walls. With the ground surface at El. +20'-0" and a final excavation level at El. –39'-0", the five-level underground parking garage requires an excavation with an average depth of 59 feet. The slurry wall, which is an integral part of the parking garage, concurrently serves as the support of excavation. Two levels of bracing laterally support the slurry walls, located at El. +8'-0" (Level 1) and El. –12'-0" (Level 2). Fig. 2 shows the bracing layout for Level 1, consisting mainly of struts and external waler beams. Level 2 bracing, not shown here, also contained tiebacks†.

† Other discussion about the construction of this project may be found in Kirmani et. al. (2003).

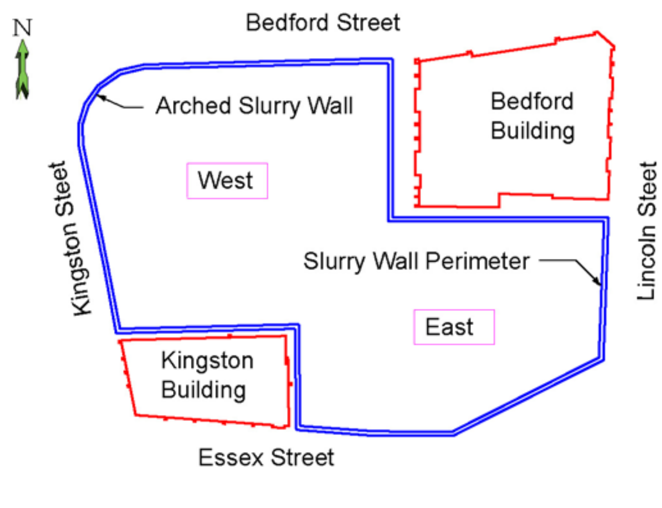

Fig. 1. Perimeter Wall Plan

Located northwest of the footprint, where Bedford Street and Kingston Street intersect, is a rounded corner, geometrically defined by a segment of a circular arc having a radius of 50 feet and an included angle of 83°. While preliminary bracing schemes developed by the contractor involved waler beams and struts laterally supporting the *arched slurry wall* at two bracing levels, it was proposed that the self-supporting geometrical characteristics of the arched wall at the northwest corner be utilized to evaluate its capacity and behavior as a temporary support of excavation system.

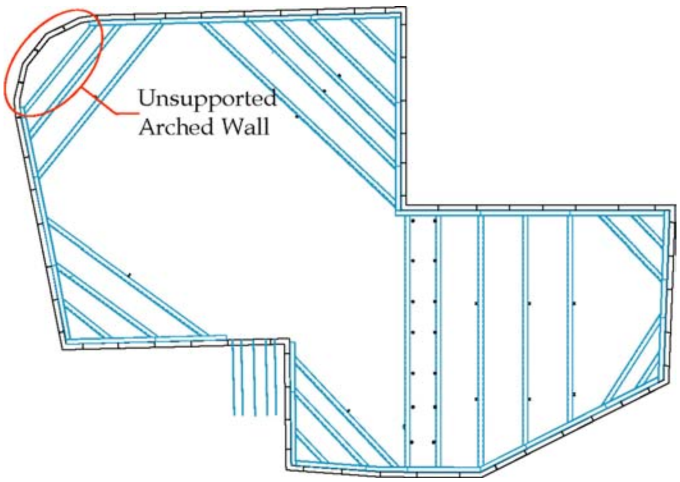

Fig. 2. Bracing Layout at Level 1 (El.+8'-0")

ANALYSIS

Figure 3 shows the layout of the slurry wall panels for the arched slurry wall. While the corner is geometrically defined by a smooth curve, practical construction considerations dictate that the wall be built using four *corner panels*. Each corner panel consists of two straight legs that are skewed to each other, such that the assembly of these panels defines the general shape of an arch.

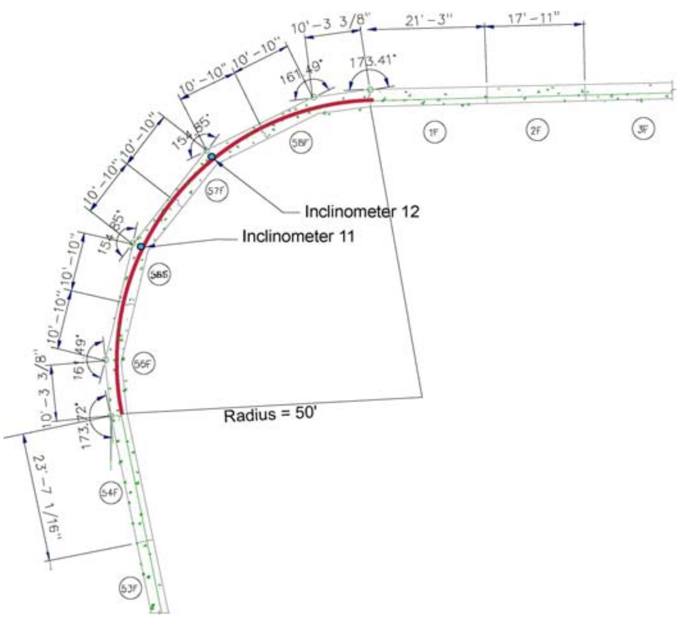

Fig. 3. Panel Layout of Arched Wall

It is anticipated that the arched wall will be subjected to lateral loads from soil pressure, hydrostatic pressure, traffic and construction surcharge pressures during excavation.

<u>Perfect Arch Characteristics</u>

In theory, a smooth circular arch subjected to radial loads behaves as a membrane in which only in-plane axial loads are generated (Fig. 4), if the end supports of a circular arch are pinned. The pressure on the arch is statically transformed into in-plane compressive stresses that are transferred to the end supports. This is ideal in this particular configuration because the reactions at the end supports are co-planar with the stiff axis of the straight slurry walls. The lateral deformation resulting from such a condition is only due to the in-plane (axial) elastic shortening of the arch. Thus, in principle, an arch is a self-supporting structure.

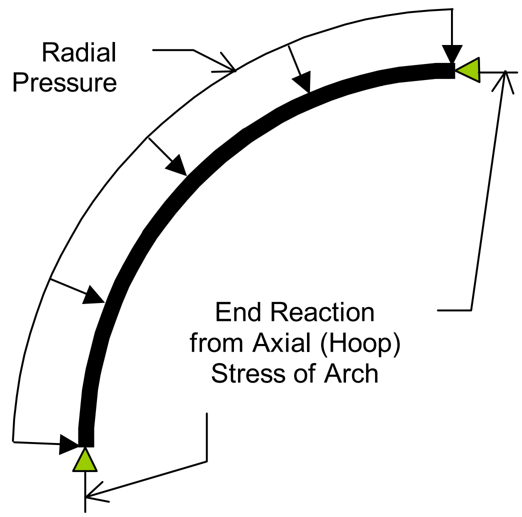

Fig. 4. Perfect Arch Loading and Reaction

<u>Behavior of Arched Wall With Corner Panels</u>

While an arched wall, consisting of a series of corner slurry wall panels, will still predominantly transfer loads axially to the end supports, deviation from a perfectly smooth curve generates bending moments and shear locally within each individual panel.

As a quick analysis of a vertical section through the arch wall, one is tempted to develop a two-dimensional axisymmetric finite element model. However, such a model will be suited for a perfectly smooth circular arched wall which will only develop horizontal axial (hoop) stresses. An axisymmetric model will not contain the extra flexural deformations that are inherent in an arched wall consisting of corner panels.

Another option, of course, would be to develop a three-dimensional model. However, such an undertaking is cumbersome and requires significant resources which, the authors believe, are not commensurate to the increase in accuracy gained over an approximate two-dimensional analysis.

Thus, to include the flexural effects resulting from the use of corner panels without necessarily performing very sophisticated analyses, a two-pronged approach was adopted:

1. Using a structural beam stick element, analyze an individual corner panel, subjected to a unit lateral uniform load, and obtain (a) its lateral stiffness due to flexural deflection, and (b) a unit load bending moment and shear relationship. This model provides a preliminary phase for a finite element model, and provides a method to determine horizontal bending moments as a function of the lateral pressure.

2. Develop a two-dimensional plane strain finite element model, incorporating the lateral stiffness determined from the previous step. This model serves to predict lateral wall movement and vertical bending moments.

As a final step in the analysis, lateral pressure obtained from the finite element analysis is directly used in conjunction with the unit load bending moment and shear relationships obtained from the first step.

Behavior of Single Corner Panel

To implement step 1, consider the typical corner panel shown on Fig. 5a. Since the ends of the panel have a thickness of 3 ft., as opposed to a point support, then there is some degree of fixity at the ends. In fact, if the entire thickness of the slurry wall at the ends is in compression, full fixity exists.

Thus, the panel may be modeled as a two-legged beam element with fixed ends, but free to translate perpendicular to the plane of the panels. With a unit distributed load applied as shown, the deflection, shear, and bending moment generated are shown on Figs. 5b, 5c, and 5d, respectively. The deflection at the midspan (apex) is 0.0183", while the deflection at the ends of the panel is 0.0229". These values, considering a unit distributed load, correspond to a lateral stiffness of 656 kcf and 524 kcf at the midspan and ends, respectively. The maximum horizontal shear per unit distributed load is 10.5 kips and occurs at midspan. The bending moment per unit distributed load at midspan is 36.75 kip-ft, while the ends generate a bending moment of –18.38 kip-ft, or half the midspan bending moment. Also, the end compressive reaction is 48.85 kips per unit distributed load.

Finite Element Analysis

Two-dimensional plane strain finite element analysis of a cross-section along the arched wall was performed using the program ANSYS. Formulation of the model follows the general procedure described in SEI/ASCE [2000], except that additional horizontal spring elements on the wall are activated. The horizontal spring elements represent the accumulated stiffness from arch action. For this project, a lateral stiffness of 400 kcf to simulate arch action was conservatively chosen.

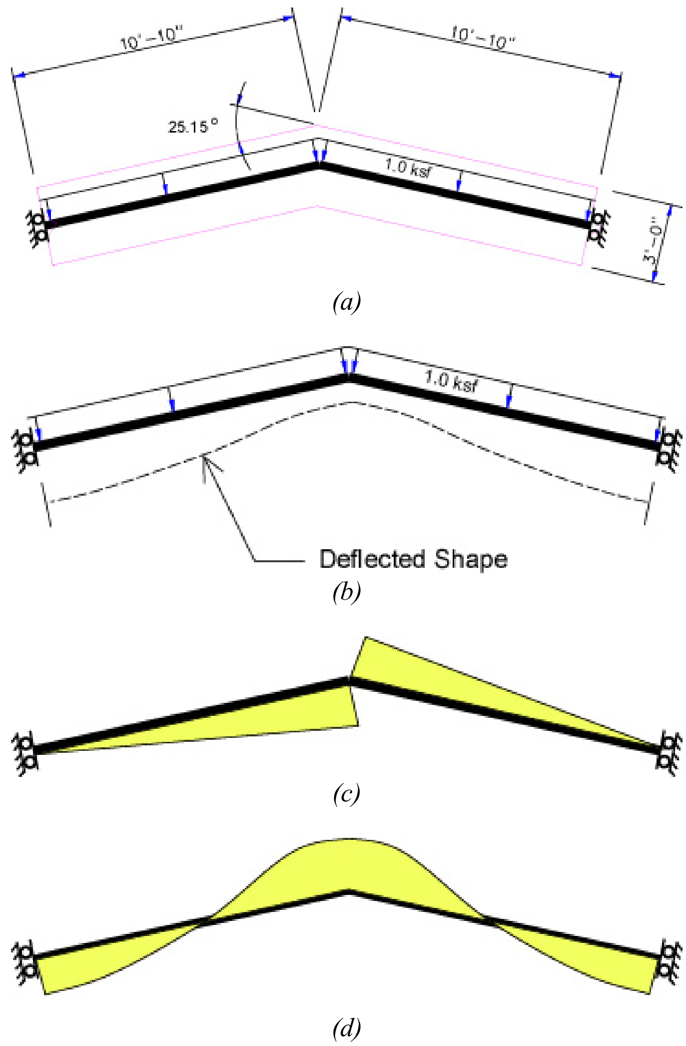

Fig. 5. (a) Typical Corner Panel, (b) Deflected Shape, (c) Shear Diagram, (d) Horizontal Bending Moment

At the site, particularly where the arched wall was located, the soil consists of a 12-ft thick layer of miscellaneous fill, underlain by a 12-ft thick silty clay layer known as the "Boston Blue Clay," a 5-ft thick very dense clayey silt Glaciomarine layer, a 35-ft thick very dense sand Glacial Till layer, and moderately weathered Cambridge Argillite bedrock. In general, the soils encountered at the site are more competent than what typically exists in the Boston area where the soft silty clay can be up to 80 feet thick.

Among the key features of the finite element analysis are as follows:

1. Cohesionless soils (Fill, Glacial Till) are modeled by a bilinear stress-strain curve with a Drucker-Prager strength criterion. The key strength parameter is the friction angle.

2. Cohesive soils (Clay, Glaciomarine) are modeled using the hyperbolic stress-strain relationship developed by

Filz, et. al. [1990]. The key strength parameter is the undrained shear strength.

3. A staged excavation analysis is performed by sequentially deactivating excavated soil elements at every bracing installation step. In this case, since no lateral bracing is present, excavation stages are chosen primarily for numerical stability and accuracy.

Figure 6 shows the finite element model for the arched wall. Four-node quadrilateral plane strain elements were used for the soil, while an elastic beam element was used for the wall. As mentioned previously, the arch action was modeled using spring (link) elements laterally attached to the wall with a stiffness of 400 kcf. No other lateral support for the wall was present. A construction surcharge of 600 psf was applied next to the wall with a width of 12 feet, while a traffic surcharge of 250 psf over a width of 40 feet was also applied.

Note that the model was used to investigate several stages of excavation, including the final (permanent) configuration.

Vertical bending moments in units of kip-ft/ft of wall are presented in Fig. 7. These are direct values obtained from the finite element analysis. Using the net total lateral pressure from various excavation stages in the analyses, horizontal bending moments for each panel were generated using the unit load relationships obtained previously. Fig. 8, for instance, shows the variation with depth of the horizontal bending moment at the apex of each panel. One can observe that, in contrast to conventional straight panels laterally supported by a number of levels of bracing, the horizontal bending moment of a corner panel of an arched wall is significantly higher than the vertical bending moment. That is, the vertical curvature is smaller than the horizontal curvature in a corner panel, implying that there are more horizontal reinforcing bars in a corner panel of an arched wall.

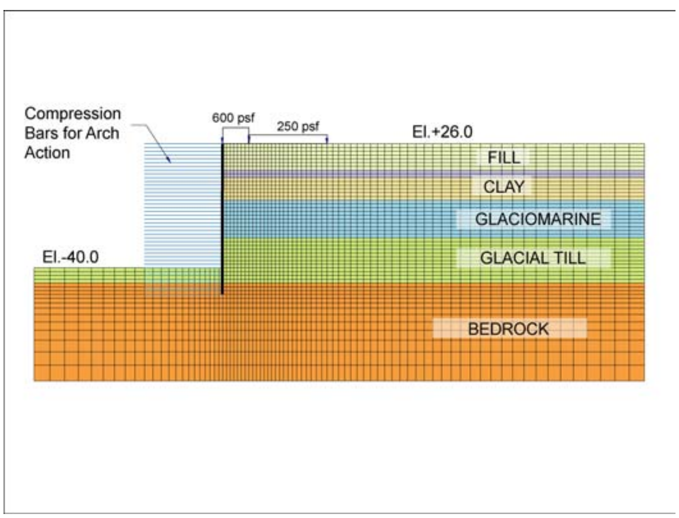

Fig. 6. Finite Element Model

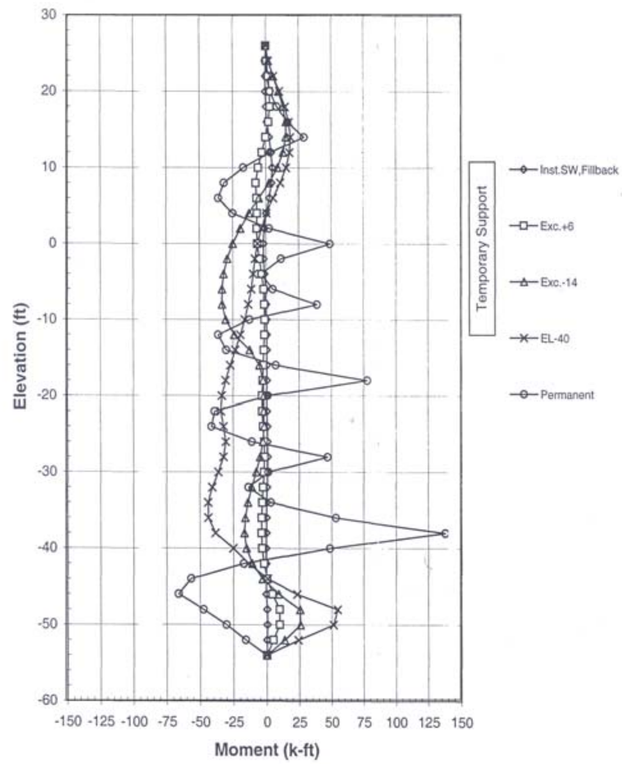

Fig. 7. Vertical Bending Moments

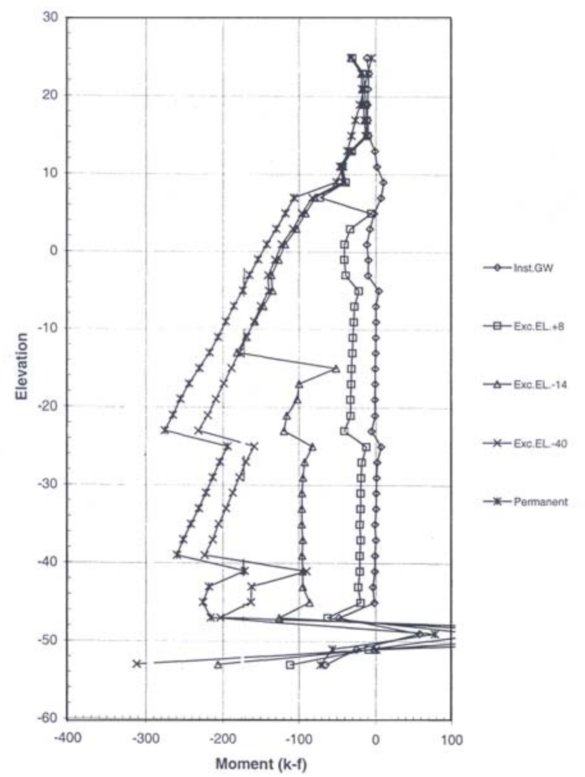

Fig. 8. Horizontal Bending Moment at Panel Apex

CONSTRUCTION

Construction of the arched slurry wall first entailed the installation of individual corner panels. Each panel was slurry-trench excavated to the desired bottom elevation, the trench being stabilized by heavy bentonite slurry. After the trench was cleaned, the reinforcing bar cage was lowered into the trench. With the cage in place, concrete was tremied into the trench, gradually displacing the slurry, until the desired top of wall elevation was reached. This procedure was repeated for each panel. Mass excavation then proceeded after the concrete had sufficiently set, methodically exposing the interior face of the arched wall.

Figure 9 shows the exposed arched slurry wall, with the bottom slab already in place. Note that walers and diagonal struts laterally support the adjacent straight slurry walls at the upper bracing level, while tiebacks are present at the lower bracing level.

Fig. 9. The Arched Slurry Wall

PREDICTED AND MEASURED WALL DEFORMATION

Figure 10 shows the wall displacement at the end of excavation, as predicted in the finite element analysis and as measured from Inclinometers 11 and 12 (see Fig. 3). The numerical analysis produces a maximum displacement of 0.64”, while the maximum wall displacement measured, from Inclinometer 12, is only 0.22”.

All three curves of Fig. 10 show negligible movement at the bottom of the wall, indicative of a satisfactory wall toe embedment into the bedrock. The predicted wall displacement has a similar shape to that of Inclinometer 11. However, an apparent deviation in behavior is observed with Inclinometer 12, in which the wall even moved back at the top and a more pronounced belly exists. An explanation for this variation in behavior is remains elusive; however, local variations in excavation and loading sequences may partially be responsible.

In any event, the observed wall deformation is smaller than predicted. Explanations for this observation may be one or a combination of the following reasons:

1. Actual soil properties at the site are much better than those recommended for design. This is a common occurrence, since a safe design is necessarily desirable.

2. Conservative values of construction and traffic surcharge were used in the finite element analysis as compared to actual construction equipment and vehicular loads present. Furthermore, these loads are considered to be short duration live loads, as opposed to sustained loads, that are likely to produce temporary deformational impacts.

3. As mentioned previously, a conservative arch action lateral stiffness of 400 kcf was used in the finite element analysis, compared to 656 kcf and 524 kcf obtained in the analysis of the corner panel. As a quick estimate, the finite element analysis displacements can be scaled down by 0.67, producing a maximum deflection of about 0.4”.

4. Actual excavation staging adopted has less detrimental impact on wall deformation, as opposed to an instantaneous soil removal assumed in the finite element analysis.

5. The actual material stiffness (e.g., concrete’s elastic modulus) of the panels is higher than conventional design assumptions.

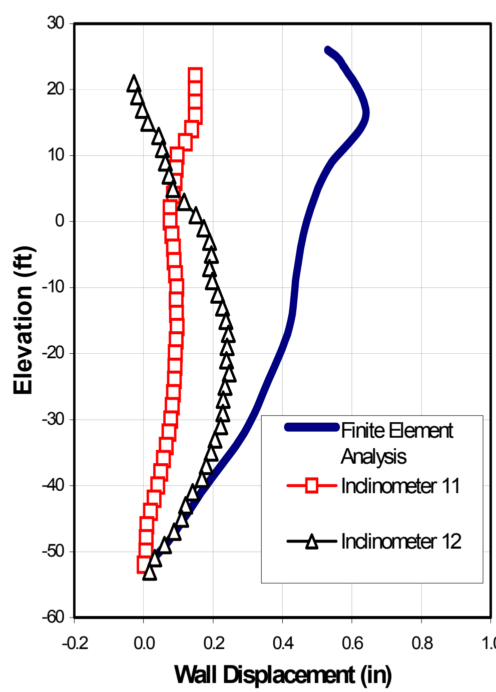

Fig. 10. Predicted and Measured Wall Displacement At End of Excavation

CONCLUSION

In the light of initial concerns regarding a laterally *un*supported wall during excavation, it was demonstrated by theory and numerical finite element analysis that the arched slurry wall located at the northwest corner of the project is a self-supporting structure that is capable of serving, in the absence of external bracing, as a support of excavation. This is possible through known arch-action principles, in which radial loads can be transformed into in-plane axial forces that are eventually transferred as axial loads at the end supports. This behavior was corroborated during construction when, not only did one inclinometer exhibit a similar wall deflected shape as the predicted shape, but that two inclinometers in the arched wall produced wall deflections which were less than half of the maximum predicted deflection. Overall, the observed smaller wall deflections demonstrated the adequacy of the support of excavation system, particularly an unbraced arched wall supporting a 59-foot excavation.

ACKNOWLEDGMENT

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