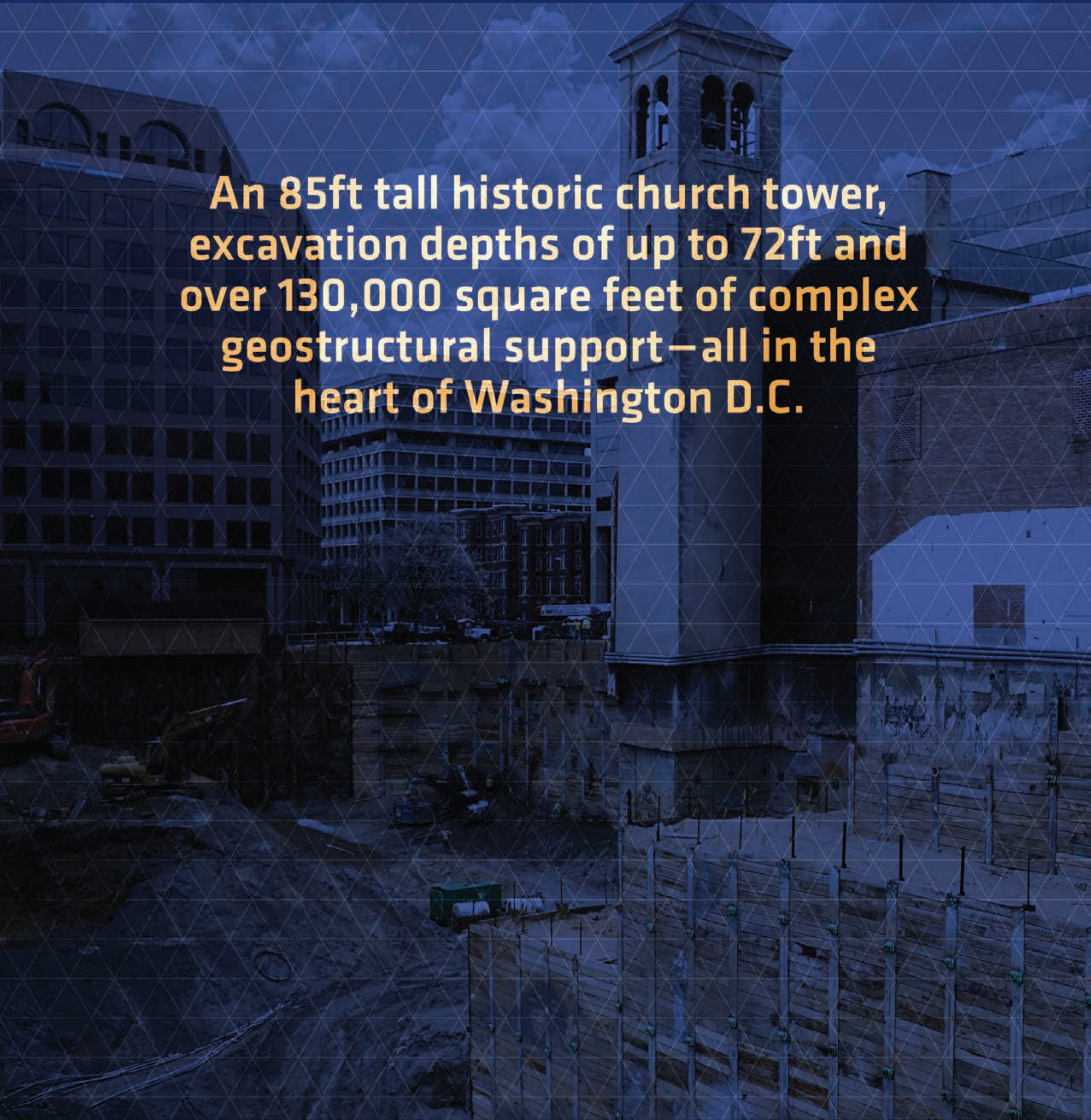

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Deflection-based design for the Capitol Crossing Support of Excavation in Washington D.C.

The Capitol Crossing project had over 130,000 square feet of support of excavation (SOE), with 1,300 tiebacks providing lateral support for excavation depths of up to 72 feet. This paper will focus on the design and performance of the SOE adjacent to the Holy Rosary Church and the Bell Tower. Deflection-based design was used to estimate the movements of and limit damage to these structures.

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CAPITOL CROSSING PROJECT

Washington D.C.

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ABSTRACT

The support of excavation (SOE) for the Capitol Crossing project in Washington, DC consisted of a combination of soldier piles, lagging, tiebacks, braces, underpinning, single auger soil mixed (SASM) walls, micropiles, and a tiedback slurry wall.

Over 130,000 square feet of SOE was installed with more than 1,300 tiebacks providing lateral support for excavation depths of up to 72 feet. Several structures including the Holy Rosary Church, the Bell Tower, the Casa Italiana, and a modern office building were supported by a combination of conventional hand dug underpinning pits, bracket piles, and stiff SASM walls. This paper will focus on the design and performance of the SOE adjacent to the Holy Rosary Church and the Bell Tower.

Deflection-based design was used to estimate the movements of and limit damage to these structures. An extensive monitoring program was used to provide real time movement data. The results of this monitoring program will be discussed in terms of estimated movements versus actual movements.

INTRODUCTION

Schnabel was retained by Balfour Beatty Construction (BBC) in 2014 as the support of excavation (SOE) subcontractor for the Capitol Crossing project. The project, which consisted of a below grade garage, a vehicular tunnel and a modified street ramp, is located in northwest Washington, D.C. The excavation for the garage is bounded by Massachusetts Avenue on the north side, I-395 on the east side, E Street on the south side, and 3rd Street on the west side (See Fig. 1). In its final configuration, the new superstructures span over I-395 and are supported by a slurry wall on the west side of I-395 and large diameter caissons located in the median and east sides of I-395.

The temporary SOE for the Capitol Crossing project consisted of a combination of drilled/driven soldier piles, lagging, tiebacks, braces, underpinning, single auger soil mixed (SASM) walls, micropiles, and a tiedback slurry wall. Schnabel designed and installed all the SOE except for the slurry wall. Over 130,000 square feet of SOE was installed with more than 1300 tiebacks providing lateral support for excavation depths of up to 72 feet. Several structures including the Holy Rosary Church (HRC), the 85ft. tall Bell Tower, the 2-story Casa Italiana, and a modern 8-story office building were supported by a combination of conventional hand dug underpinning pits, bracket piles, and stiff SASM walls. The photo in Fig. 2 shows the existing buildings.

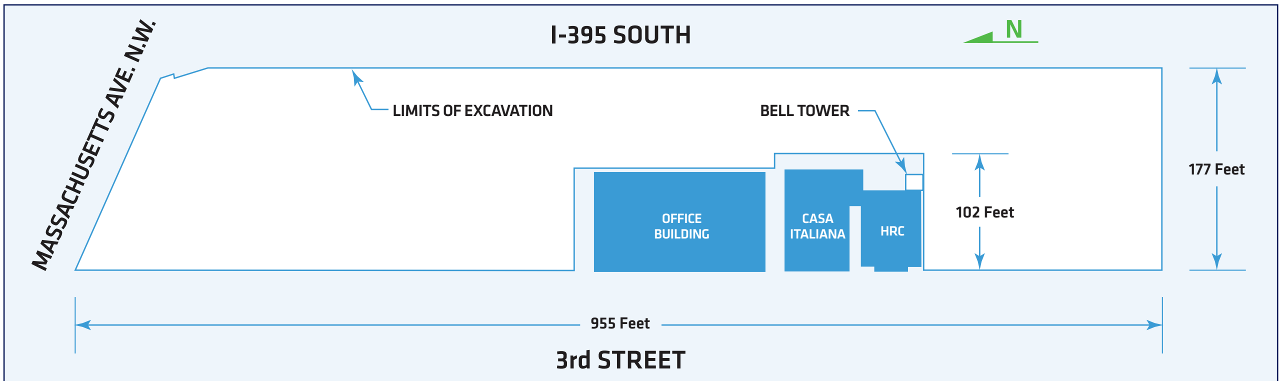

Fig. 1 Plan view of site

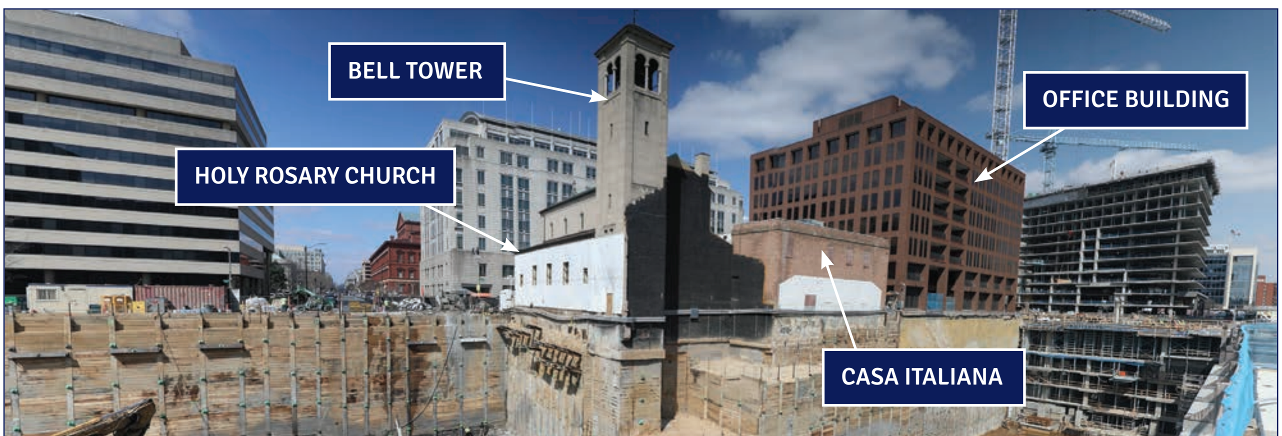

Fig. 2 Existing buildings adjacent to excavation

Deflection-based design was used to estimate movements and limit damage to the adjacent buildings. An extensive monitoring program was used to provide real time movement data, the results of which will be discussed in terms of estimated movements versus actual movements. The design and performance of the SOE adjacent to the HRC and Bell Tower will be the primary focus of this paper.

DEFLECTION-BASED DESIGN

Deflection-based design is used to estimate the movements of the SOE. These estimated movements are used to estimate the ground movements behind the SOE which in turn are converted to angular distortion and horizontal strains in the adjacent structures. The estimated strains are then used to predict potential building damage. The goal is to be able to reliably estimate movements of the SOE and therefore ground movements behind the SOE that result in limited damage to buildings within the influence zone of an excavation. Ground movements consist of vertical and horizontal movements Vertical movement causes angular distortion while horizontal movement causes lateral strain (see Fig. 3). Boscardin and Cording (1989) developed damage criterion considering angular distortion and lateral strain. The damage criterion is based on the state of strain at a point (see Fig. 3d from Son et al 2005). The average state of strain in a building unit is evaluated using this criterion.

A building unit can be a section between two columns or cross walls, two different building geometries, or building stiffnesses, or two different ground displacement gradients. It is usually the portion of the building closest to the excavation and subject to the largest distortions. Per Boscardin and Cording (1989), the damage criterion "is based on the concept that a structure is deformed by the combination of angular distortion and lateral strain, and the maximum strain on the structure can be determined by a principal strain formed by both the angular distortion and the lateral strain". Potential building damage levels are estimated by comparing the maximum principal strain with the critical strains for each different damage category shown in Fig. 3d. The results of field observations are the basis for the critical tensile strains shown in Fig. 3d for different damage levels. Each of the boundaries between damage categories represents a constant principal extension strain.

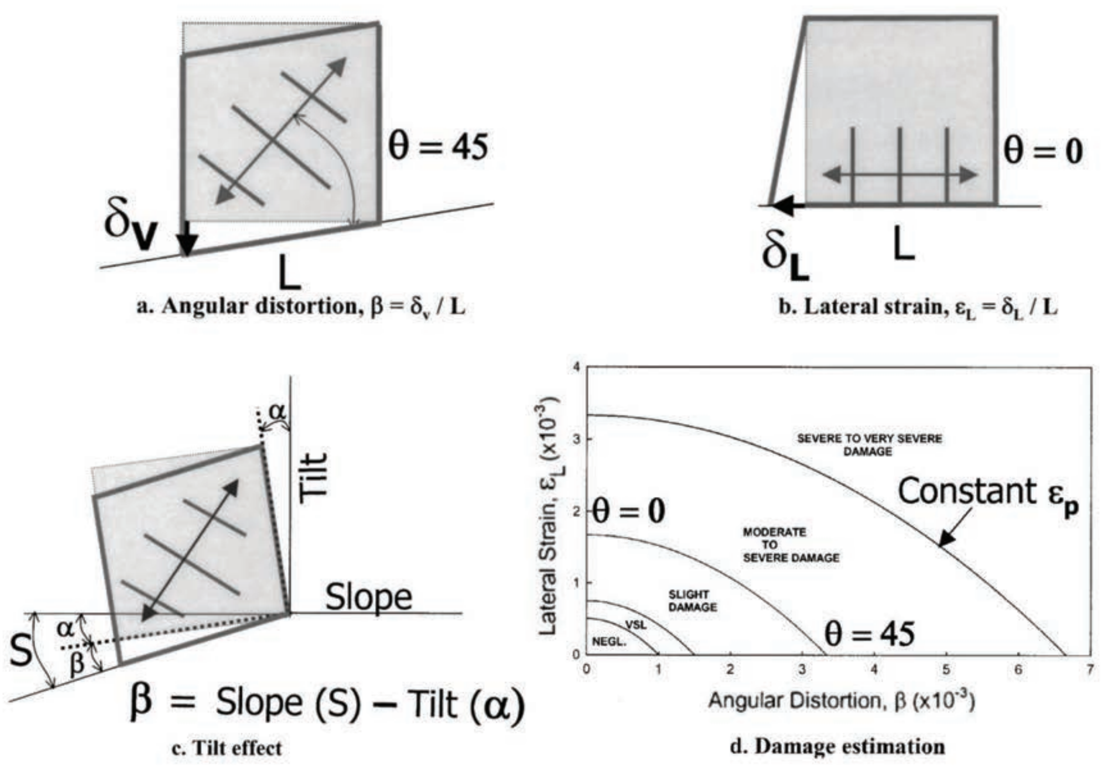

Fig. 3 State of strain at point or average state of strain in distorting portion of structure (Son et al. 2005)

Burland et al. (1977) presented a table that gives a description of each of the damage categories shown in Fig. 3d. Part of Burland's table is presented in Fig. 4. It was developed for masonry buildings and was not intended for reinforced concrete structural elements. It is conservatively assumed that the structure is flexible enough to move with the ground and is most appropriate for masonry structures that are not tied or reinforced.

BUILDING DAMAGE CLASSIFICATION

Negligible Damage

- Hairline Cracks. Crack Width < 0.004 inches
- Principal Strain <0.0005

Very Slight Damage

- Fine cracks easily treated during normal redecorating. Crack Width < 0.04 inches. Perhaps isolated slight fracture in building. Cracks in exterior brickwork visible upon close inspection
- Principal Strain - 0.0005 to 0.00075

Fig. 4 Building Damage Classification (after Burland et al. 1977)

The goal for the design of the SOE adjacent to the buildings on the Capitol Crossing project was to limit any damage to "very slight" or "negligible". One of the adjacent buildings, the HRC, had beautiful stained-glass windows on its south side which were sensitive to any differential type of movement. Per the descriptions given in Fig. 4 the goal would limit damage to fine cracks with crack widths less than 0.04 inches. By definition, fine cracks could be easily treated during normal redecorating. There could also be isolated slight fractures in the building interiors and some cracks might be visible in the exterior brickwork upon close inspection.

GROUND MOVEMENT CONCEPTS

The lateral displacement of the SOE wall that occurs during excavation is mainly controlled by the relative soil - wall stiffness. This is a function of the bending stiffness of the SOE, the distance between supports, and the modulus of the soil. This relationship is expressed as the Flexibility Ratio and is presented in Equation 1:

$$\text{Flexibility Ratio} = \frac{E_S L^3}{EI} \qquad [1]$$

Where E_S = Young's secant modulus of the soil, L = distance between wall supports, E = elastic modulus of the SOE wall, and I = moment of inertia of the SOE wall.

A relationship relating the Flexibility Ratio to normalized lateral wall displacements is used to estimate ground movements. The volume of the surface settlement trough can be estimated from the volume of lateral wall displacement. In sands it is assumed that the settlement volume is approximately equal to the lateral displacement volume. In addition, the settlement profile is modeled as a parabola that extends a distance of 2H (where H equals the depth of cut) from the SOE wall face. Maximum ground settlement is assumed to be equal to 1.5 times the average lateral displacement.

Lateral SOE wall movements consist primarily of:

1. Cantilever movement due to excavation prior to placing the first tieback or brace
2. Lateral bulging movements that develop below brace or tieback levels as the excavation proceeds to subgrade
3. Lateral movement due to rotation of a tiedback wall as it settles

Note that the three (3) components of wall movements listed above do not include movements due to installation of the SOE and movements due to translation of the wall to develop lateral resistance of the toe. The three components of wall movement are determined as follows.

Cantilever movement of the SOE occurs before the first tier of tiebacks is installed. Typically, it is calculated for H1+2 feet where H1 = the depth to the first tieback elevation. The additional two (2) feet account for the tieback bench being located at an elevation two (2) feet below tieback elevation. "For the cantilever deflection of a braced excavation..., lateral displacement of the ground surface will be high, on the order of 1 to 1.5 times the vertical displacement." (Cording et al. 2010). The Flexibility Ratio is used to calculate the cantilever movement. The computed cantilever movement is converted to a volume of lateral soil displacement.

The lateral bulging movement occurs between the first tier of support and subgrade. The Flexibility Ratio is again used to compute the bulging movement except that depth of cut is based on the average span between tiers of support plus two (2) ft. Note that the lateral bulging movements can best be controlled by reducing the average span between tiers of support since the movement is proportional to the cube of the average span. The bulging movement is converted to a volume of lateral soil displacement.

The third component of lateral wall movement due to settlement is determined by first estimating the settlement of the SOE. Drilled piles were used for the SOE adjacent to the existing buildings. The settlement of the SASM piles was calculated based on known design methods for drilled shafts. This settlement was converted to a lateral displacement of the top of the SOE which was then converted to a volume of lateral soil displacement.

The total volume of lateral soil displacement is equal to the volumes computed for each of the three movement components. Based on the assumption that the parabolic settlement trough extends a distance of 2H from the SOE wall and that the average volume of lateral soil displacement equals the average volume of vertical soil displacement, a maximum ground settlement was calculated. Based on a parabolic distribution of soil settlement behind the SOE wall, the angular distortion was computed.

Finally, the horizontal strain was calculated as a function of the angular distortion and the type of building.

SOIL CONDITIONS AND PRESSURE DIAGRAMS

Fig. 5 on the following page shows a boring in the vicinity of the HRC.

The upper 5 feet to 10 feet of the site consisted of man-made fill and disturbed natural soils. Below the fill layer were alluvial deposits typically consisting of interbedded layers of silt, medium dense sand, clay, and gravel. These deposits extended to depths of 70 feet to 80 feet. The alluvial deposits were underlain by the Potomac Group deposits that typically consisted of medium to very dense interbedded, discontinuous sand and clay layers. The existing water table was located approximately 26 feet above subgrade. Prior to the start of excavation in front of the HRC and Bell Tower, the water table had been lowered to subgrade.

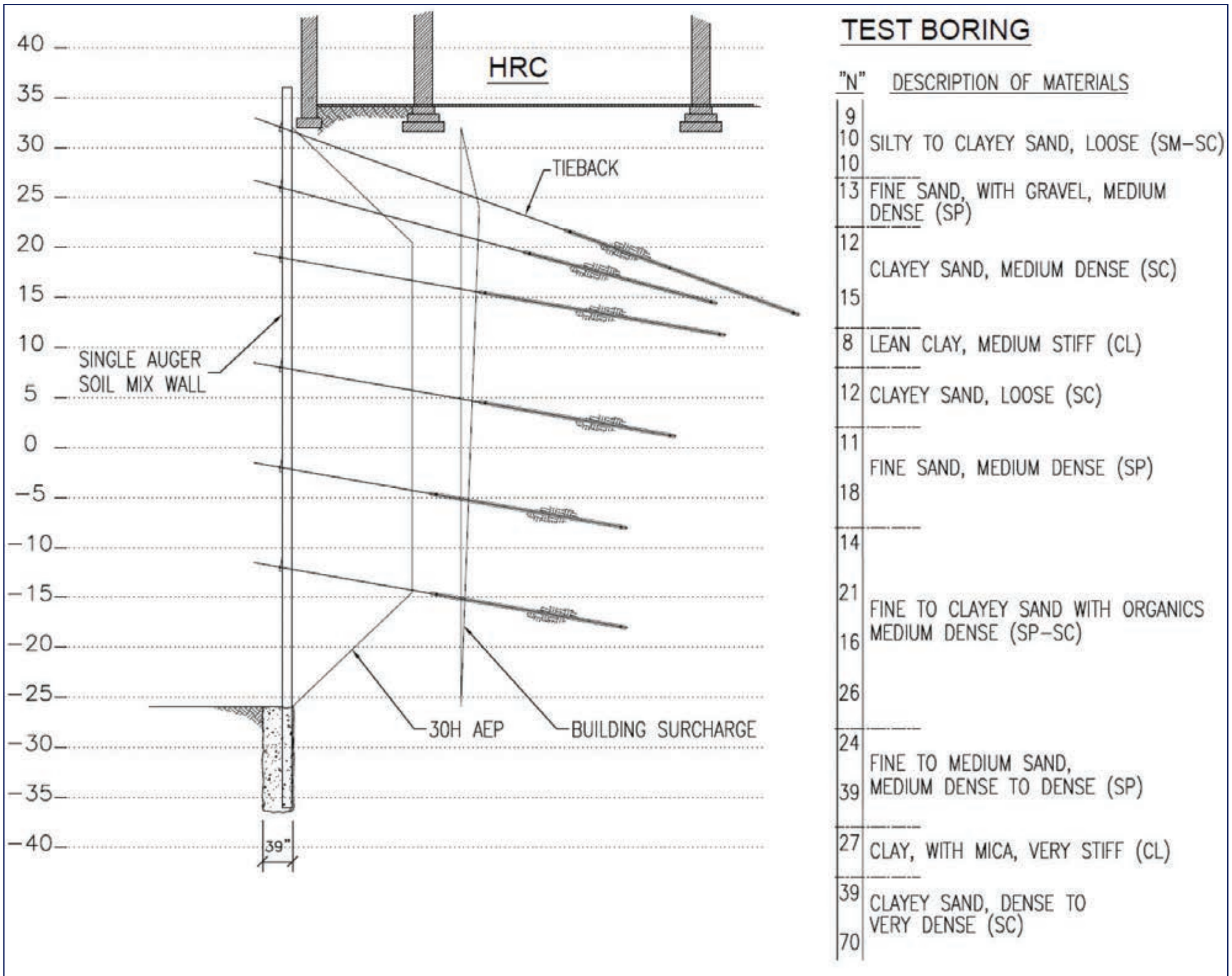

Fig. 5 Cross Section through SOE and HRC with Earth and Surcharge Pressures and Test Boring

Fig. 5 also shows that a 30H apparent earth pressure (AEP) diagram was used to design the SOE in front of the two buildings. Building surcharges were added to the AEP diagram. The design did not include any hydrostatic pressures. Once the tieback loads and pile sizes were calculated, deflection-based design was performed so that any building damage would be limited to “negligible” or “very slight” as defined in Fig. 3. Due to the deflection-based design considerations, the pile size was increased and an additional tier of tiebacks was added to the upper 17 feet of the SOE wall.

SUPPORT OF THE HRC AND THE BELL TOWER

The HRC was built in the 1920s and construction of the Bell Tower followed at a later date. The two buildings were not connected until construction of the Capitol Crossing project started. Bolts were used to connect the adjoining walls of the two buildings. The top of the HRC building is approximately 50 feet above street grade and has one basement level. It is supported by exterior reinforced concrete footings and eight (8) interior column footings divided in two rows running in an east–west direction parallel to the long dimension of the church (see Fig. 6 on following page).

The interior column footings are located 12 feet and 40 feet from the exterior south wall. Fortunately, existing drawings and design calculations for the HRC were made available which facilitated the design of the SOE adjacent to the building. The maximum depth of cut adjacent to the south side of the HRC was 59 feet. A SASM wall was used to support this side of the building. It consisted of drilled shafts spaced at 2.5-foot centers with steel beams placed in every other shaft. Five (5) to six (6) tiers of tiebacks were used to provide lateral support. Conventional hand dug underpinning pits were used on the shallower east side of the HRC.

Of particular note is that the design called for three (3) tiers of tiebacks to be installed in the first 17 feet of the excavation in front of the HRC. This was done to create a much stiffer wall in the upper 17 feet and therefore minimize the amount of potential settlement of the columns located 12 feet from the exterior wall. The first tier of tiebacks was installed 3 feet to 4 feet below the top of pile to limit cantilever movements. Since the design load per pile for the first tier was relatively low, wales were used so that two piles could be supported by one tieback with twice the design load. In addition, all of the tiebacks were locked off at 100% of their design load to further limit deflections.

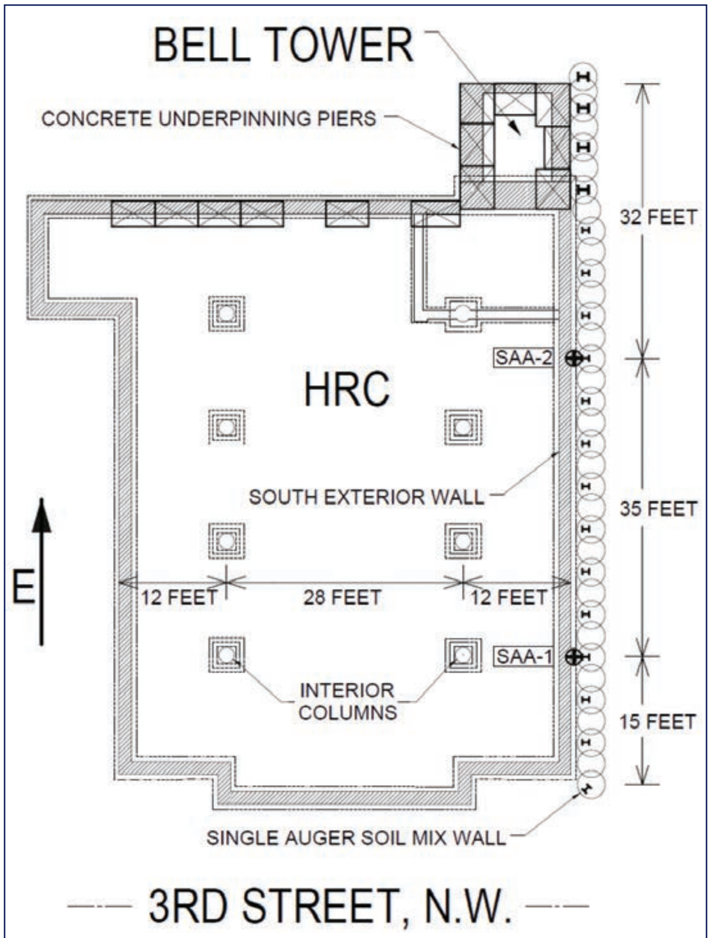

Fig. 6 Plan View of HRC and Bell Tower

The 85-foot tall Bell Tower has an approximate 14 foot by 14 foot footprint. No existing drawings could be found so a test pit was done to determine the type of footing that supported the Bell Tower. The test pit revealed that the tower was supported by a 28-inch thick reinforced concrete mat footing.

Excavation was done around three (3) sides of the tower. The south side of the tower was excavated to a depth of 56 feet, while the east and north sides were excavated to a depth of 27 feet immediately adjacent to the tower. Along the east side, at a distance of 18 feet from the tower, the excavation was advanced an additional 32 feet.

The original final design of the SOE comprised continuous conventional hand dug piers to support the Bell Tower. These piers varied in size from 5 foot by 3 foot to 5 foot by 4 foot. Six (6) tiers of tiebacks were designed to provide lateral support on the south side. The final design had to be modified during excavation of the first pier on the south side. Uncontrollable water was encountered at subgrade. Since the piers were designed to terminate five (5) feet below subgrade, a SASM wall was installed in front of the piers from a higher elevation to augment the capacity of the piers.

MONITORING PROGRAM

Due to the critical nature of the buildings an extensive, real-time monitoring program was implemented to measure movements of the buildings as well as of the SOE itself. Movements were measured in the x (normal to the excavation), y (parallel to the excavation), and z (vertical) directions.

Prisms were attached to the buildings and the SASM piles. Two (2) inclinometers were installed on the south side of the HRC (see Figs. 6 and 7). Inclinometer SAA-1 was located 15 feet from the southwest corner of the HRC and 3rd Street, while inclinometer SAA-2 was located 50 feet from the corner. Their location relative to the corner of the excavation is important in the explanation that follows of the movements that were measured at each location. Each inclinometer was attached to a pile in the SASM wall. Fig. 7 shows the monitoring points that were set up on the Bell Tower, the underpinning piers, and the HRC. The eight (8) interior columns of the HRC were monitored with tilt sensors.

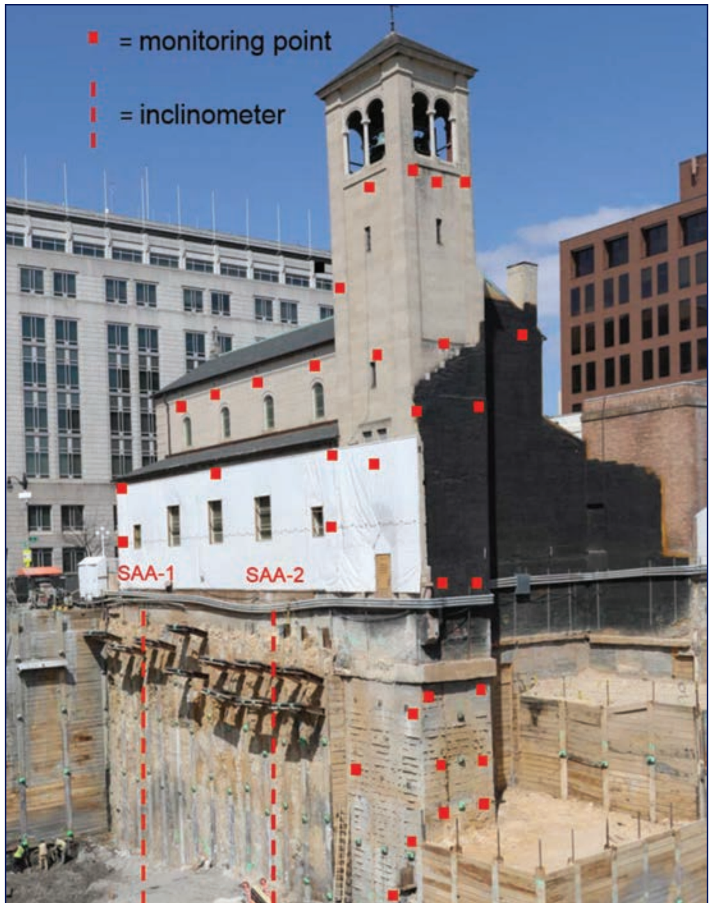

Fig. 7 Monitoring points on the HRC, Bell Tower and Underpinning Piers

The monitoring of SASM pile no. 8 and inclinometer SAA-2 was not started until the excavation had been advanced ten (10) feet. By this time, the first tieback had been installed and tested. As a result, pile no. 8 had slightly greater movements than those measured by the prism on the pile and the inclinometer.

These movements were corrected by adding the cantilever movements of SAA-1 and pile no. 15. The excavation in front of pile no. 15 had only been advanced about five (5) feet when the first readings were taken. The movements measured by the tilt sensors on the interior columns were relative movements rather than absolute movements. The measured movements were all relative to the western most sensors.

Fortunately, the monitoring of the exterior HRC walls provided a reasonable means of converting the relative movements of the interior columns to absolute ones. The prisms on the upper exterior wall, which are supported by the interior columns, provided an independent means of measuring movement of the columns.

ACTUAL MOVEMENTS VS. ESTIMATED MOVEMENTS

Fig. 8 presents a summary of the estimated and actual movements measured at each inclinometer location. The relationships developed by Cording et al. (2010) were used to estimate a maximum settlement of 0.56 inches of the south wall of the HRC at the location of SAA-2 and 0.45 inches of the first row of interior columns. An average lateral soil displacement of 0.37 inches over the final depth of the excavation was estimated. Angular distortion and horizontal strain were then calculated for the structure. In this case, the angular distortion was calculated over a span of 12 feet corresponding to the distance between the exterior wall and the first row of interior columns. The damage criterion graph (see Fig. 3d) predicted that the HRC could potentially exhibit "very slight" damage due to the 58-foot deep excavation in front of pile no. 8. Bulging movement was predicted to be the primary contributor to overall movement. It was estimated that it would be responsible for approximately 90% of the overall volume of lateral soil displacement.

INCLINOMETER	SAA-1	SAA-2
Depth of cut (feet)	52	58
Number of tieback tiers per design	5	6**
Lateral movement at pile top (inches)	0.0004H	0.0013H
Average lateral movement of SOE		
Estimated (inches)	0.33*	0.37
Actual (inches)	0.34*	0.52
Settlement of HRC at South wall		
Estimated (inches)	0.56	0.56
Actual (inches)	0.15	0.50
Settlement of HRC at South interior column		
Estimated (inches)	0.44	0.45
Actual (inches)	0.12	0.42

* Movements based on H=44 feet because SAA-1 readings become unreliable after the last tieback bench was reached
** Seven (7) tiers were actually installed due to field conditions.
Note: Estimated accuracy of measurements = +/- 0.06 inches

Fig. 8 Estimated and actual movements at inclinometer locations

Similarly, for inclinometer SAA-1, a maximum settlement of 0.56 inches was estimated for a 52-foot-deep cut. Note that five (5) tiers of tiebacks were designed to provide the lateral support in contrast to the six (6) tiers of support for the 58-foot-deep cut at SAA-2. An average lateral displacement of 0.33 inches was estimated.

ACTUAL LATERAL MOVEMENTS

Fig. 9 shows the lateral movements measured by inclinometer SAA-2 at various stages during the excavation to subgrade. The movement of the SOE wall is plotted as each tieback bench was reached. The upper three (3) tiers of tieback benches were reached between late December 2017 and late February 2018. The movement data shows that the SOE wall was actually pulled back slightly from the excavation during the first 17 ft. of excavation.

During this time, the interior footings exhibited negligible movement. Bulging movements started to appear by the time the 4th tier tieback bench was reached at a depth of 26 feet in mid-March. Note that the maximum movement occurred at and just below the tieback bench elevation. Also note that a fair amount of movement occurred well below the bench elevation. This was also the case as the excavation advanced to final subgrade. This pattern of movement stresses the importance of limiting the depth of excavation below each tieback elevation.

By inspection, it is clear from Fig. 9 that bulging movements comprised the majority of the total measured movement volume as predicted. At subgrade, maximum lateral movement was just short of 0.8 inches. This movement occurred at depths between 30 feet and 40 feet. The corrected maximum movement measured by inclinometer SAA-2 at the top of the pile was about 0.50 inches. The total movement volume between the top of pile and subgrade can be estimated from Fig. 9. It was about 360 in^3, which translates to an average lateral movement of about 0.52 inches over the depth of the excavation. This was approximately 40% greater than the estimated average lateral movement of 0.37 inches.

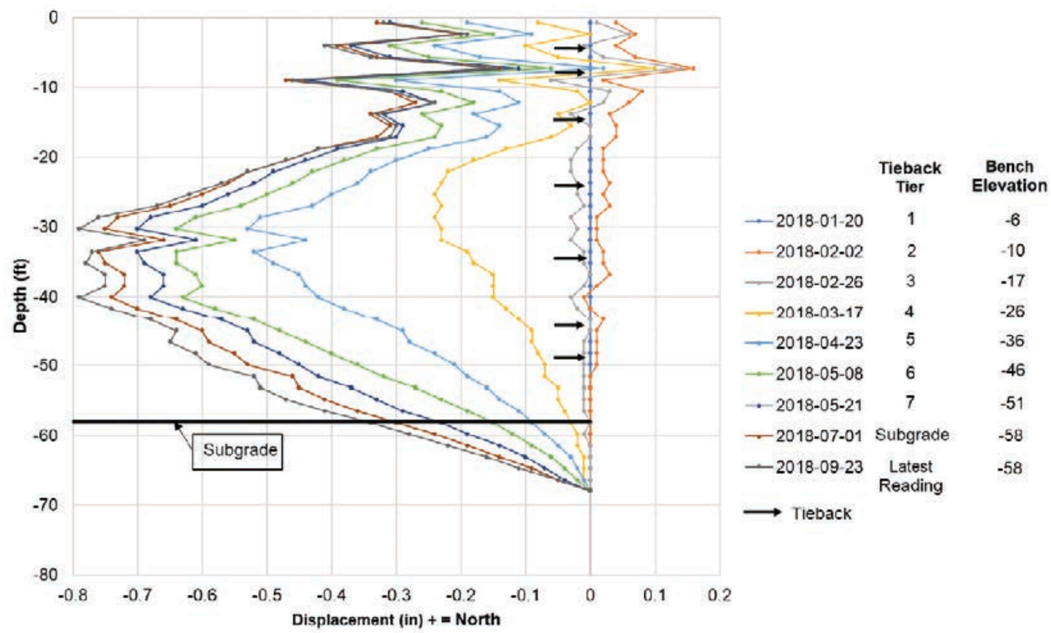

Fig. 9 SAA-2 Inclinometer Movements into the Excavation at the South Wall of the HRC

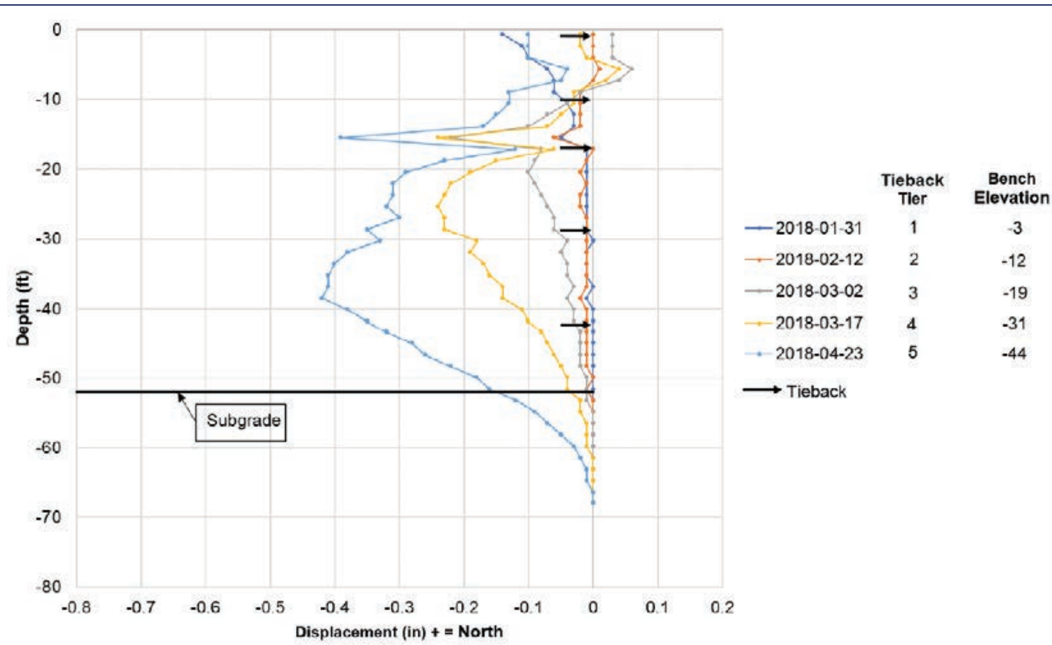

Fig. 10 SAA-1 Inclinometer Movements into the Excavation at the South Wall of the HRC

The monitoring points located on the building and the top of SASM pile closest to inclinometer SAA-2 showed maximum corrected lateral movements of 0.80 inches and 0.90 inches, respectively. This corresponds to a movement of 0.0013H where H is equal to the depth of the cut. Although these movements cannot be compared directly to the average lateral movement of 0.52 inches, they can be compared to the corrected maximum lateral movement of 0.50 inches measured at the top of inclinometer SAA-2. One possible explanation for the smaller movement measured by inclinometer SAA-2 is that the pile toe was not fixed at the lower tieback bench elevations. The inclinometer was attached to the pile and did not extend below the tip of the pile. This would also increase the average lateral movement of 0.52 inches determined from Fig. 9.

The lateral movements measured by inclinometer SAA-1 are shown in Fig. 10 for excavation depths up to 44 feet. The inclinometer produced unreliable readings after that depth was reached. However, the movements measured up to that point can be compared with those measured by inclinometer SAA-2 at a bench elevation of 46 feet. For example, at a depth of 40 feet, inclinometer SAA-1 shows a maximum movement of 0.40 inches, while inclinometer SAA-2 shows a maximum movement of just over 0.6 inches. There is significantly less bulging movement as well. The pile toe showed much less movement.

The monitoring points located on the building and the top of pile closest to inclinometer SAA-1 showed maximum lateral movements of 0.20 inches and 0.25 inches, respectively. This corresponds to a movement of 0.0004H. These movements were significantly less than those measured at inclinometer SAA-2. The most plausible explanation for the significant difference in movements measured at the two inclinometer locations is the influence of corner effects on the movements at inclinometer SAA-1. This inclinometer was located only 15 feet from the southwest corner of the excavation.

ACTUAL VERTICAL MOVEMENTS

The maximum settlement of the HRC was estimated to be 0.56 inches at both inclinometer locations. The actual corrected settlement of the exterior wall at the location of inclinometer SAA-2 was 0.50 inches which was very close to the estimated settlement. At inclinometer SAA-1, the actual settlement of the wall was 0.15 inches, which was less than half the estimated settlement. These movements showed that there was negligible differential settlement of the exterior wall. The settlement of the row of interior columns located 12 feet from the south wall was 0.12 inches. Again, this meant there was negligible differential settlement between the column footings and the south wall. The insignificant differential settlement correlated well with the lack of any significant cracking in the HRC. The stained glass windows remained intact!

OTHER MOVEMENTS

Fig. 11 on the following page presents SAA-2 inclinometer movements measured parallel to the south side of the excavation. According to inclinometer SAA-2, the SOE wall at the location of the inclinometer moved east 0.7 inches to 0.8 inches. Inclinometer SAA-1 showed similar behavior but on a smaller scale. It showed movements to the east of 0.2 inches to 0.3 inches. The movement pattern measured by the inclinometers was supported by the movement patterns measured by the monitoring points on the HRC and the Bell Tower. Note that there were monitoring points on both the south and east sides of the Bell Tower which all showed a general pattern of movement in an easterly direction. In fact, the monitoring points on the HRC indicated that the entire building moved as a block from the west to the east. The lateral movement of the west side of the HRC towards the east measured by three (3) different monitoring points was approximately 0.6 inches.

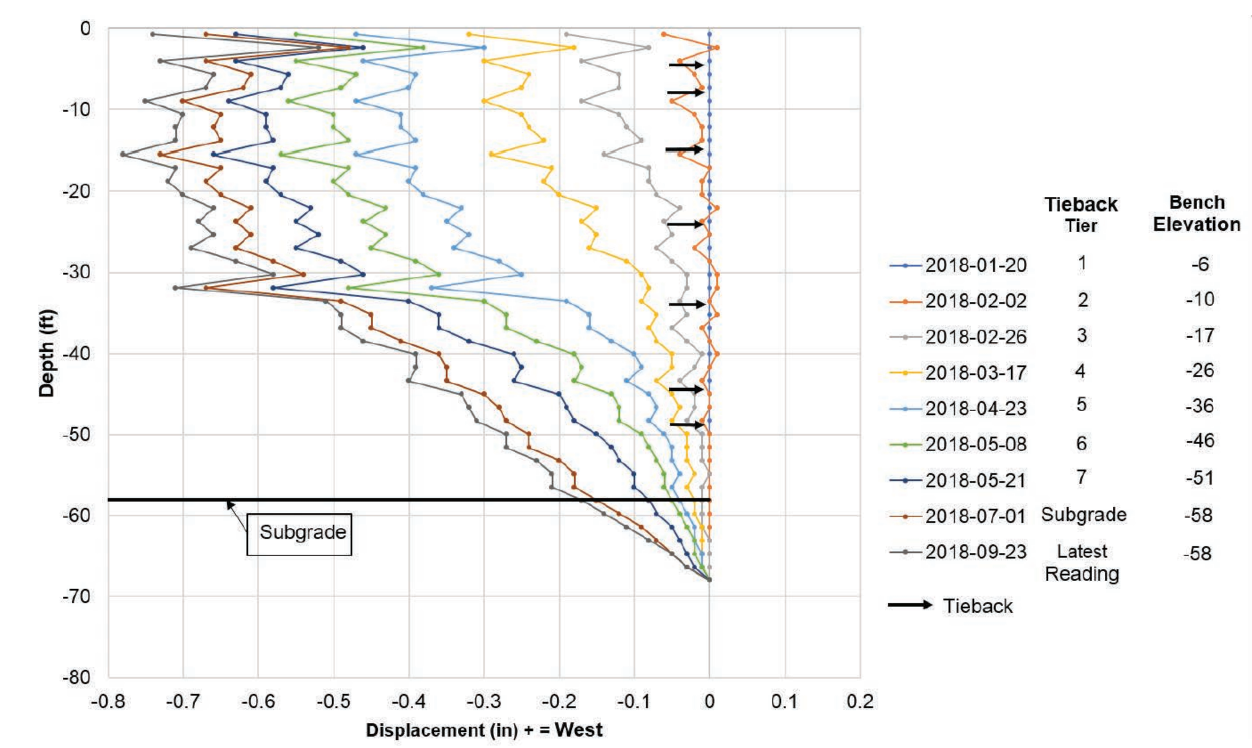

Fig. 11 SAA-2 Inclinometer Movements into the Excavation at the South Wall of the HRC

Recall that there was also an open excavation on the east side of the Bell Tower. The base of the Bell Tower moved east approximately 0.6 inches. Some movement of the HRC and the Bell Tower towards the east was expected, but what was unexpected was the movement of the HRC as a block. This behavior was also observed for the Casa Italiana and the modern office building.

SUMMARY AND CONCLUSIONS

This paper presented the design and performance of the SOE adjacent to some existing buildings at the Capitol Crossing site in Washington, DC. Deflection-based design was used to estimate the movements of the SOE and ultimately the strains in the adjacent buildings. Existing relationships were used to estimate potential building damage. An extensive and successful monitoring program was set up to provide real time movement data. Actual movements were then compared to estimated movements from which the following conclusions can be drawn.

1. The performance of the SOE met the design goal of limiting building damage to “very slight” or “negligible”.
2. The relationships developed by Cording et al. (2010) provided a reasonable method of estimating potential building damage. However, the measured bulging movements were greater than estimated at the inclinometer location near the east end of the HRC. This led to an average lateral soil displacement greater than estimated, but the settlement of the building was close to the estimated amount. The reasons for this behavior need to be studied further.
3. The inclinometer located near the west end of the HRC measured movements that were less than estimated. This was most likely due to the influence of corner effects on SOE movements.

4. For any excavation adjacent to an existing building, the SOE design intent is to limit potential damage to the building. Selecting the earth pressure and surcharge pressure diagrams is just one step in designing the SOE. It is essential to consider the stiffness of the SOE in the design as well.
5. Unexpected eastward movement of the buildings as blocks was measured. The reason for this behavior needs to be investigated further.
6. More case studies need to be done on well-monitored buildings adjacent to excavations to expand the database on estimated performance versus actual performance.

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