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Impact of Tunneling on Two Brick-Bearing-Wall Structures

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SYNOPSIS: The responses of a pair of brick-bearing-wall structures to nearby construction of twin, shield driven 21-ft-diameter tunnels in soil are examined. Horizontal and vertical ground displacements are summarized and discussed, as well as, horizontal and vertical displacements, tilting, distortion, and damage sustained by the structures. Transient features of the developing settlement trough and effects on building response are also examined and discussed.

INTRODUCTION

This paper reviews the response of a pair of two-story, brick-bearing-wall structures to excavation of two nearby subway tunnels. The tunnels are part of the Washington, D.C. METRO System, and are 20.8 ft in diameter with a springline depth of 45 ft. The center to center tunnel spacing is 42 ft. Fig. 1 shows the relative positions of the structures and the tunnels in profile. As shown by the site plan, Fig. 2, the longitudinal axes of the buildings are not parallel to the tunnel axes.

SUBSURFACE CONDITIONS

The soil profile shown on Fig. 3, indicates that the test section is located near a transition from dense sands and gravels in river flood plain deposits to hard, clayey, estuarine soils. Observations made at the tunnel heading during excavation beneath the test section indicated that the heading material was a hard red clay with occasional weathered and sandy zones near the tunnel crown. The clay material is hard and fissured with some slickensides present. Deep dewatering wells were used to lower the ground water level during construction. Ground control in the test area was not a problem and ground losses appeared to decrease with passage through the transition zone.
EXCAVATION AND CONSTRUCTION PROCEDURE

The tunnels were excavated using a Robbins articulated shield. The shield was 21.17 ft long at the crown with an outside diameter of 20.83 ft. The shield was equipped with hydraulically operated breasting flaps. The front of the shield included a 4.5 ft long hood with a 1/2 in. overcutter bar all around the leading edge. The shield was composed of three sections, each of approximately equal in length with articulation joints connecting the sections. Hydraulic jacks also connected the front and middle sections and provided control of the attitude of the front section relative to the middle section. The connection between the middle section and the tail section was such that the tail could freely trail the middle section. The tailskin was 1/2 in. thick to minimize ground loss as the temporary support passed out of the tail section.

The excavation cycle consisted of: 1) shoving the shield forward into the soil with hydraulic jacks reacting against the temporary lining, and 2) raking the muck onto a conveyor belt with a hydraulically operated spade. The conveyor then carried the muck from the face into muck cars. A temporary lining consisting of steel ribs and timber lagging was assembled within the tailskin of the shield and then expanded as each rib cleared the tail. The ribs were four-piece W6X25 sections and were spaced about 4 ft center to center. The lagging consisted of 5 in. by 3.75 ft long timbers. The tunnel excavation and support system is described in detail by MacPherson et al., (1978).

STRUCTURES

The two brick masonry structures and their positions relative to the tunnels are illustrated in Figs. 1 and 2. The buildings are two-stories high with full basements. The longitudinal axes of the buildings are oriented approximately 22 degrees from the tunnel axes with the corner of Building I 5 ft from the center line of the inbound tunnel. Because of their proximity to the tunnel excavations these structures were vacated during tunnel construction.

The two buildings are similar in construction. The bearing walls are parallel to the longitudinal axes of the buildings and composed of brick with lime mortar. There is no structural connection between the two buildings. A steel beam supported by the facade walls and three equally spaced interior columns, extends along the length of each building, midway between the bearing walls. The timber floor joists, 2-in. by 10-in. at 16-in. intervals, span between the center beam and the bearing walls. The joist bearing at the masonry pockets was about 4 in. The bearing walls and columns are supported by shallow footings at depths ranging from 4 to 8 ft below the exterior ground level. Information about the exact nature and size of the footings was not available. However, rubble type footings probably support both buildings. Based on type of construction, materials and present condition, the structures are estimated to be 80 to 90 years old. There appears to have been some renovation and restoration of the joists and front facade walls.

The bearing walls are 14 in. thick at basement level and are reduced 1 in. in thickness for each story thereafter. The facade walls are 12-in.-thick brick masonry walls. The front facade walls are faced with one wythe, approximately 4 in., of cement mortar brick masonry backed by 8 in. of lime mortar brick masonry. The exposed lime mortar is generally soft and quite easily scraped from the joints of both the bearing and facade walls. In many instances there are gaps where the lime mortar has been eroded or has fallen from the joints. The exterior of the front facade wall has better mortar and presents a more competent appearance; the joints are tight and very hard with few cracks or gaps. The interior walls of Building I are either exposed brick or plaster over brick. Many cracks were present prior to tunneling similar to that observed in Building I.

OBSERVATION PROGRAM

Observations may be divided into three categories: Measurements of movement of the ground mass; Distortion measurements of the building; Inspection for visible evidence of building distortion (e.g. cracking, jammed doors, etc.). The observations in each case were made before, after, and periodically during tunnel excavation. The following is a brief description of the observations made. More detailed descriptions of observations may be found in Boscardin (1980).

Observations of movements of the ground mass were predominantly settlement measurements. However, the magnitude of the horizontal strain in the extension zone was estimated through observation and measurement of cracks in the sidewalks and pavement that developed parallel to the tunnel axes. There were three lines of settlement points perpendicular to the tunnel axes at Stas. 307+90, 208+15, and 308+70. A fourth line ran along the centerline of the inbound tunnel from Sta. 307+60 to Sta. 308+70, Fig. 2. Three deep settlement points were also monitored. The anchorages for the deep settlement points are about 4 ft above the crown of the tunnel. Bench marks were located 110 ft and 140 ft from the center of the inhoindor to the tunnel. Detailed descriptions of the ground movements may be found in MacPherson et al., (1978).

Building distortion was monitored using five types of observations:

Interior bay distortion was determined by changes in horizontal and diagonal distances between elements of the bay. Measurements were made using a tape extensometer having a sensitivity of 0.001 in. and a repeatability of 0.004 in.
Building settlement was based upon optical level surveys of exteriors of both buildings and the interior of the basement of Building I. The level-rod system had a repeatability of 0.04 in. and closure errors were on the order of 0.04 in.

Tilt of the south wall of Building I was measured using plumb bobs suspended from the roof. Measurements were repeatable to 0.03 in.

Relative horizontal displacements between Building I and II were determined from changes in distance between pairs of studs attached on either side of the vertical joint forming the interface between buildings. Measurements were made using a caliper with a sensitivity of 0.001 in. Repeatability was on the order of 0.01 in.

Change in bearing of floor joists was determined by displacement of a reference stud on the joist relative to the face of the wall. A caliper with a sensitivity of 0.001 in. was used. The repeatability of the system was 0.01 in.

The third category of observations was inspection for visible evidence of building distortion. Detailed surveys noting the condition of buildings were made. Cracks were mapped and selected cracks were measured before and after tunnel excavation. Building elements which often prove quite sensitive to distortion were also inspected. These included doors, windows, column-beam intersections, and corner areas.

GROUND SURFACE AND BUILDING SETTLEMENTS

The settlements discussed in this section are related only to excavation of the inbound tunnel. Excavation of the outbound tunnel, which was farther from the buildings, occurred first and was monitored by the contractor. Construction records indicate less than 1/8 in. of settlement occurred in Building I in response to outbound tunnel construction and no evidence of building distress due to excavation of the outbound tunnel was observed.

The pattern of ground surface settlement along the centerline of the inbound tunnel is shown in Fig. 4. The five curves illustrate surface settlements associated with various positions of the tunnel heading during excavation. The data indicate that surface settlement decreased as the tunneling passed through the transition zone from sandy Pleistocene terrace deposits to hard Cretaceous clay. Final surface settlements along the centerline range from 1.5 in. (Cretaceous clay) to nearly 3 in. (sandy terrace deposits). Deep settlement monitors in the Cretaceous clay indicated approximately 2 in. of deep settlement above the tunnel crown. Deep settlement also appeared to decrease with passage through the transition zone. Fig. 4 also indicates that the surface settlement preceded the tunnel heading by about 15 ft and 25 ft during tunneling in the Cretaceous clay and sandy terrace deposits, respectively. Ten to fifteen percent of the total surface settlement occurred before the face of the excavation reached a reference point. Forty to sixty percent of the total surface settlement appeared by the time the tail of the shield passed a given point. In addition, the sandy terrace material appeared to settle more than the hard clay material once the tail passed a given point and the ribs and lagging support was in place.

Surface settlements along a line perpendicular to the tunnel axis at Sta. 308+70 are shown in Fig. 5. Settlement profiles corresponding to several locations of the tunnel face are shown.

It is apparent that based on the final settlement data portions of the structures lie in the zone of lateral extension, while other portions (see Fig. 4) are in the zone of lateral compression due to continued development of the settlement profile. Therefore, Building I will be subjected to transient patterns of distortion potentially quite different than the final settlement profile would suggest.

A transient pattern of extension and compression zones is also present when considering lateral ground movements parallel to the tunnel axis. The settlement profile in the vicinity of the tunnel heading exhibits a reversal of curvature and a zone of maximum curvature similar to the transverse settlement profile of the trough. In effect, the buildings are subjected to two components of horizontal...
extension and compression, one transverse to the tunnel axis and one parallel to the tunnel axis. Evidence of the horizontal extension transverse to the tunnel axis appeared in the form of several new 1.32-in.-wide cracks, parallel to the tunnel, that formed in the sidewalks 20 to 40 ft from the tunnel centerline. However, the transient condition parallel to the axis of the tunnel is often totally masked when examining the final settlement profile along the centerline of the tunnel.

Settlement surveys of the ground surface and exteriors of the structures indicate that the buildings settled with the ground surface and little bridging occurred. The final settlement of Building I ranged from 1.4 in. to 0.14 in., and the final settlement of Building II ranged from 0.42 in. to less than 0.05 in.

MEASURED BUILDING DISTORTIONS

An exaggerated sketch illustrating the final distorted configuration of Buildings I and II along a transverse cross-section located near Sta. 308+50 is shown in Fig. 7. The sketch along with the settlement contours shown in Fig. 8 summarizes final settlement, tilt, tape extensometer, and crack width data at the cross-section. The dimensions along the diagonals and the horizontals of Fig. 7 are strains along those lines. Extension and compression strains are denoted positive and negative, respectively. Settlements and crack widths are in inches, whereas rotations and slopes are specified as tangents of angles.

The relative positions of Buildings I and II on the ground surface settlement profile should be noted. Building I is nearer the center of the settlement trough and predominantly in the zone of lateral compression. In this zone, vertical settlement dominates and horizontal ground strains are very small. On the other hand, Building II is near the edge of the settlement trough and in the zone of lateral extension. Here, settlements and differential settlements are smaller than those found nearer the center of the settlement trough, and horizontal tensile strains are more significant.

However, final distortion data only tell part of the story. Building I was in the zone of lateral extension during the early stages of development of the settlement trough. When the face of the tunnel was at Sta. 308+50, the total settlement of the bearing wall nearer the tunnel was 0.6 in. with no observable settlement noted at the bearing wall farther away from the tunnel. At this time, the horizontal extension strain at the basement level was 1/3300 and both diagonals were in extension as a result to the lateral extension of the ground (Fig. 9).
The shear strains, derived from the differential settlements, caused a greater extension along one diagonal than along the other. The rigid body rotation of the structure was about 1/2000 and the slope of the building settlement profile equaled 1/1300. Therefore, angular distortion of the structure (settlement slope across the building minus rigid body tilt) was about 1/750. Thus, during the early stages of the development of the settlement trough, the distortion of the structure appears to be dominated by the shear strain caused by differential settlement.

When the face of the tunnel moved to Sta. 308+60, the front door of Building I became tightly jammed. The distortion of the door frame was sufficient to bind the door which had previously opened easily. Later, when the settlement trough was nearly fully developed, the door again worked normally. This one instance illustrates a situation where a portion of the structure experienced more severe angular distortion during the development of the settlement trough than the final measurements indicate. In such cases, predictions of building response based upon estimates of final distortion alone may be misleading.

The final distorted shape of Building I is caused primarily by differential settlements across the structure. The differential movement between bearing walls is 1 in. and causes a slope of 1/230 across the structure. The final relative horizontal movement between the bearing walls at their base was negligible. The distortion caused by this combination of relative movements has primarily two components: a rigid body tilt and a shear or angular distortion of the building. The rigid body tilt of the structure was apparent from the plumb line measurements and from the opening of the door at Sta. 308+60. The final plumb line measurements lead to a calculated rigid body rotation of 1/710.

Shearing distortions are indicated by the strain measured along the diagonal tape extensometer lines. One diagonal of each pair exhibited extension whereas the other exhibited compression as shown in Fig. 7.

Due to the orientation of the Building I relative to the tunnel axis, the structure cuts across the settlement contours at an angle and torsion is induced in the structure. This angle of twist was approximately 0.15 degrees over the 60-ft length of the structure. In this case the effect of the torsion of the building was slight. The amount of torsion induced was small and the lack of fixity of the structural connections between the wall and floor systems allowed this structure to tolerate this torsion with negligible deleterious effects. However, a transient torsion can also occur in a structure regardless of orientation relative to the tunnel axis due to the pattern of development of the settlement trough and should be considered in any evaluation of building response.

The final distortions of Building II, shown in Fig. 6, illustrate the behavior of a structure in the zone of lateral extension. The differential settlement between the bearing walls is 0.2 in., causing the building settlement curve to have a slope of 1/1250. The rigid body rotation of Building II is on the order of 1/1300 or less. The horizontal settlements and the rigid body rotations of Building II are less than those of Building I. Therefore, the differential settlements, caused a greater extension of the settlement trough and should be considered in any evaluation of building response.

The horizontal tape extensometer measurements show lateral strains between the bearing walls ranging from 1/3100 in the basement to 1/1300 at the roof. Both diagonals of each set showed extension. The diagonal extension strains range from 1/3000 to 1/1300 for the basement and second story tape extensometer lines, respectively. The greater extension measured along the horizontal and diagonal tape extensometer lines higher up in the structure are caused by a relative rotation of the bearing walls. The bearing wall nearer the center of the settlement trough is on a steeper portion of the ground surface settlement curve and thus rotates more than the farther bearing wall (Fig. 6).

**VISIBLE EVIDENCE OF BUILDING DAMAGE**

Visual inspections were made before, after, and at intervals during the tunnel excavation under the test site. The initial conditions of both Buildings I and II were quite poor. Extensive cracking was noted on the interiors and exteriors of both structures and the interior plaster walls were cracked and loosened at many locations. The initial state of each building was recorded through photographs, mapping of cracks, measurement of selected cracks, and written descriptions. Additional cracking and the increase in size of pre-existing cracks were noted during and after the tunnel excavation. When viewed in light of the very poor initial condition of both structures any damage caused by the tunnel excavation was termed as negligible to very slight. However, if the same structures were in good repair and had been occupied, the same response would probably have been considered to be slight to moderate damage, see Boscardin (1987).

New cracks and an increase in the width of existing cracks were found in Building I during and after tunnel excavation. Areas where the
cracking was noticed include a front and rear facade walls, the south bearing wall, and the basement slab. Examples of the cracking at these locations are shown in Fig. 10. The rear facade wall experienced a 1/64 in. increase in the width of several of the existing cracks. An increase in crack size was also noted in the south bearing wall near the front facade wall. Here a diagonal crack from the second-story window down to the facade wall became clearly visible (Fig. 10b). In the front facade wall of Building I, the cracks were concentrated around the doors at the first floor and the windows at the second floor (Fig. 10c).

Cracks around the door nearest the excavation ranged from 1/32 in. to 1/8 in. wide at the bottom and top of the door, respectively. The door became jammed and difficult to open as a result of the tunnel-induced distortion. The door at the north end of the facade wall was surrounded by cracks about 1/32 in. wide. An increase in the widths of cracks on the front facade wall were also evident at the second floor where vertical cracks below the windows increased about 1/64 in. in width. A new crack also appeared in the basement slab of Building I near the south bearing wall. The crack was nearly 20 ft long and 5/64 in. wide and appeared when the tunnel face was at Sta. 308+30 (Fig. 11). The crack approximated the shape of the contours of settlement for this position of the tunnel face relative to the building. Tape extensometer data matched crack measurement data relatively well.

Cracking in Building II was concentrated at the corner of the south bearing wall and the front facade wall. A pre-existing 1/16-in.-wide vertical crack between the bearing wall and the facade wall opened to 1/8 in. in the basement to 1/4 in. at the second floor. Daylight was visible through the crack at several locations. Another crack at the corner between the ceiling and the front facade wall of the second story was initially 1/8 in. wide and increased to 3/8 in. wide. A pre-existing hairline crack at the corner of the south bearing wall and ceiling of the second story near the front of the building also grew to 1/4 in. wide. The tape extensometer data for Building II show that nearly all of the lateral extension experienced by the structure was concentrated in the few cracks described above.

Data from the plumb bob survey are summarized in Fig. 12. The resultant displacements of the top of the wall relative to its bottom at each plumb bob location are shown as vectors for various stages of tunnel progress through the site.

Both the distance of the wall from the centerline of the tunnel and the orientation of the wall with respect to the tunnel axis influence the tilt and its pattern of development. In this case the wall is oriented such that it cuts across the settlement trough so that the final tilt occurs both perpendicular to the building wall, toward the tunnel axis, and parallel to the building wall, toward...
the point where the wall is closest to the tunnel centerline (in this case in the direction of tunnel advance).

Changes in the width of the vertical joint forming the interface between Buildings I and II were monitored. Initially, the joint was approximately 1/8 to 3/16 in. wide. The joint opened an additional 1/8 to 3/8 in. in response to tunnel excavation as shown in Fig. 7. Comparison of joint separation with tape extensometer and plumb bob data indicate the data to be compatible.

DISCUSSION

The building distortion data was also used to study the development of the settlement trough. For example, the plot of the tape extensometer-measured displacements at Sta. 308+50 in Fig. 13 illustrates the behavior of the structure for various locations of the tunnel face.

Initially, as the tunnel heading approaches the station of the cross-section being monitored, only the wall nearer the tunnel displaces toward the tunnel to cause an increase in the distance between the bearing walls. During this early phase of the trough development the wall is in the zone of horizontal extension. The wall tilts, moves horizontally towards the tunnel, and settles slightly. As the tunnel heading passes by the station of the cross-section, the settlement trough widens and the wall is no longer in the zone of extension, but in the zone of compression. The horizontal movements are slight, yet the vertical movements are significant resulting in extension of one diagonal and compression of the other. Later, as the tunneling progresses the settlement trough continues to widen and the zone of extension begins to influence the next wall farther out causing it to displace horizontally toward the tunnel. The horizontal distance between the two walls now decreases while the diagonal distances remain constant. The increase in differential settlement between the bearing walls compensates somewhat for the decrease in horizontal distance, and the diagonal distances do not change.

The tape extensometer data for reference points located along the longitudinal axis of Building I, perpendicular to the tape extensometer lines described above, demonstrate the response of the structure to the transient settlement wave in the plane of the tunnel axis. In the vicinity of the tunnel heading, the longitudinal ground surface settlement profile exhibits zones of lateral tension and compression, an inflection point, and a point of maximum curvature similar to the typical surface settlement profile perpendicular to the tunnel axis. As the shield approaches a reference point, the ground moves horizontally toward the shield and the point is in the zone of lateral extension. Once the shield passes the point in question, the absolute horizontal motion is reversed as the ground continues to move toward the shield, but now the reference point is in the zone of lateral compression. Chronologically, the longitudinal span should first tend to extend horizontally, then compress horizontally, and finally extend again if the axis of the building is parallel to the axis of the tunnel. However, the change in a span during passage of the shield will vary somewhat depending upon orientation of the span relative to the tunnel axis and the ground conditions. The case shown in Fig. 14 exhibits this behavior.

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The final modes
of the structure are directly related to the position of the structures relative to the settlement trough. Building I, located within the concave or bowl-shaped portion of the settlement trough, sustained primarily shear related deformation. The building width was approximately equal to 1/3 the half width of the settlement trough and so significant rigid body rotation of the building occurred. This resulted in an angular distortion of approximately one-half the average slope of the settlement trough beneath the building. The lateral extension sustained by the building is small and most distortion was in the form of angular distortion. Building II is located on the convex portion of the settlement trough where lateral extension is a significant factor in causing building deformation. The convex profile causes a bending mode of distortion which, in turn, produces larger lateral extensions in the upper story.

Cracking and damage to Building I was minor. The cracking and increase in crack widths that did occur was not significant due to the poor initial condition of the structure. The cracking at the front of Building I can be attributed primarily to the angular distortion of the structure. Cracking and damage to Building II was caused primarily by lateral extension and its amplification in the upper story by the independent rigid body rotation of the bearing walls. Nearly all the lateral extension strain across the building was concentrated in one crack.

It is evident that both Building I and Building II experienced some damage in response to the nearby tunnel excavation. However, considering the initial states of these structures, the damage was very slight to slight. If the buildings were initially in good repair, the cracking damage would have been considered slight to moderate.

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