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Examples of building response to excavation and tunneling
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1 INTRODUCTION
Monitoring of ground movements around tunnels and excavations on the Washington Metro led to development of procedures for assessing ground loss or movements at the boundaries of the excavation or tunnel and the distribution of movements through the soil mass to the ground surface and to adjacent structures (Cording and Hansmire, 1975, O’Rourke and Cording, 1974). As the field investigations progressed in Washington, instrumentation and observations were concentrated on the effect of ground movements on structures. (Boscardin and Cording, 1989). More recently, a research program consisting of numerical and model studies correlated with field observations was conducted to assess the relation of building distortion and damage to excavation-induced ground movements. (Son, 2003, Laefer, 2001, Ghahreman, 2004).

This paper provides examples of building damage and distortion resulting from excavation or tunneling and evaluates the behavior of the buildings using methods developed in previous studies. In many cases, the investigations were begun after the damage was observed in order to determine mitigation and repair procedures or in order to assess the causes of, and responsibility for, the damage. In most cases, some pre-existing settlement data was available, and, in some cases, pre-condition surveys were available. Most importantly, the structures themselves served as indicators of the type and causes of distortions and damage that were imposed on them. The ability to observe and read the building response is aided by an understanding of the chain of relationships that extends from the excavation or tunnel and adjacent ground loss or ground movement, to the distribution of ground movement and volume change through the soil mass, to the interaction of the ground with the building, and then to the building distortion and damage. To properly assess the building behavior it is necessary to understand not only the ground movement patterns but also the building characteristics.

Cases include buildings where (1) the damage is clearly a result of the excavation or tunneling and there is no other cause of significant building damage, (2) the distortion and damage due to tunneling or excavation is imposed on and affected by pre-existing weaknesses or deterioration in the structure, (3) the distortion and damage due to tunneling or excavation can be separated from pre-existing damage and deterioration in the structure.

The buildings are on shallow foundations in U.S. cities. Most are masonry bearing wall structures built in the 1800s or early 1900s.
2 REVIEW OF GROUND MOVEMENT AND BUILDING DISTORTION

2.1 Sources of ground loss

Sources of ground loss around a shield tunnel are related to (1) the volume loss into the tunnel face, which may be sudden and large if face stability is not controlled, and to soil filling annular voids around the shield and tail of the shield.

2.2 Volume loss, volume change and surface settlement volume

Once the ground loss at the excavation or tunnel is estimated, the volume of the surface settlement trough can be determined. For soft clays, the volume of the surface settlement is approximately equal to or slightly greater than the volume of ground loss as the shield passes. With time, as the clay around the tunnel consolidates due to disturbance, increase in mean stress, or drainage into the tunnel, additional surface settlements will occur, with a wider settlement trough that is influenced by the size of the consolidating zone around the tunnel. As described in Case 2, for the Chicago clay, consolidation developed in a zone extending approximately 4 radii from the center of the tunnel.

For medium to very dense sands, the volume of the surface settlement is less than the volume loss. For loose sands, such as fill around utilities, the volume of the surface settlement may be greater than the volume loss. For twin tunnels, the loosened zone above the first tunnel will be recompressed by the second tunnel so that the total surface settlement volume approaches the volume loss for the two tunnels.

2.3 Distribution of settlements

The magnitude of the settlements can be found once the volume, shape and width of the settlement trough is determined. A Gaussian distribution provides a reasonable fit to observed settlements above a tunnel. The half width, \( w \), of the settlement trough can be estimated by using the angle from the vertical, \( b \), of a line extended from the springline of the tunnel to the ground surface. The angle \( b \) is typically in the range of 20 to 30 degrees for tunnels in sand and 35 to 45 degrees for the short term ground movements around tunnels in clay. Maximum settlement, \( \delta_{\text{max}} \), and average settlement slope, \( \delta_{\text{max}} / w \), can then be obtained (Figure 1).

For a braced excavation, a parabola provides a reasonable distribution for settlements adjacent to the excavation, although the displacements will decrease close to an excavation wall that has good bearing and settles less than the adjacent ground. For excavations in sand, the parabola extends laterally a distance of approximately 2 times the excavation depth and most of the displacement is concentrated in a zone extending laterally 1.5 times the excavation depth.
As the tunnel advances, a moving settlement wave advances ahead of the tunnel, so that structures in the path of the tunnel are impacted by a longitudinal settlement profile that begins at an angle $\beta$ ahead of the face and has a slope that depends on the distribution of ground losses along the length of the tunnel shield and tail, but typically has a maximum slope approximately equal to the average slope, $\delta_{\text{max}}/w$, of the settlement cross-section. Thus, although the portion of a building located above the tunnel, in the center of the settlement trough, will be in a zone of lateral compression and sagging (concave) settlement, the advancing longitudinal settlement profile will create a moving zone of lateral extension and hogging (convex) shaped settlement beneath the structure, that can cause cracking due to shear and extension in walls oriented parallel to the tunnel. In the case of the Evanston tunnel, over half of the ground settlement was caused by long term consolidation of the clay so that the longitudinal strains were not large and cracks did not develop in walls oriented parallel to the tunnel.

Similarly, a moving ground settlement profile may advance away from a braced excavation as it is deepened. Thus, an adjacent building is first impacted by smaller displacements, concentrated nearer the excavation, which may have a sufficient slope to initiate cracking in the portion of the building close to the excavation. At this time, the remainder of the building is likely to be outside the zone of movement so that tilt is small and most of the settlement slope results in angular distortion. With further extension of the settlement profile up to and beyond the full width of the structure, displacements will increase but an increasing proportion of the settlement slope will be comprised of tilt, so that the ratio of angular distortion to settlement slopes drops. Numerical modeling of distortion and cracking in brick bearing walls has shown the strain-path dependence of the cracking. When a settlement wave was advanced across the base of the structure, the damage in the later stages tended to concentrate on the cracks formed in the early stages in the portion of the bearing wall nearest the excavation so that the pattern of cracking
was different than the cases in which the final settlement profile was applied, without a progressive movement across the structure (Son, 2003).

In several projects, large ground movements did not develop until the excavation approached full depth. In these cases, weak layers near or below the base of the excavation caused deep seated movement that developed ground cracks and displacement at distances behind the excavation of 1 to 2.5 times the depth of the excavation. Movements and cracking were sudden, rather than progressive, and a pattern of displacement and cracking near the wall of the excavation did not develop.

2.3 Lateral displacements

For a tunnel, lateral displacements at the ground surface will be largest at the point of inflection of the settlement trough, \( i = 0.4 \, w \). Beyond the point of inflection, lateral strains are in extension, and can be estimated using the relationship shown in Figure 2.

![Figure 2: Estimating lateral displacements due to tunneling](image)

The lateral strain is proportional to the maximum settlement and increases with increasing \( b \). For an excavation, lateral displacement of the ground surface will be largest for cantilever deflection of the excavation wall (on the order of 1 to 1.5 times the vertical displacement) and the lateral displacement at the ground surface will be of the order of 0.5 to 1.0 times the vertical for the bulging displacements occurring below strut and tieback levels (Milligan, 1974).

2.4 Long term displacements

Long term volume change (consolidation) occurs in clay surrounding the tunnel due to (1) disturbance of the clay around the tunnel perimeter, (2) increase in the mean stress in the ground due to the pressure applied during shoving of the shield and expansion or grouting of the lining, and (3) drainage into the tunnel reducing porewater pressures below the original ambient
porewater pressures. The volume change results in surface settlements that typically are wider and flatter than the initial settlement trough created by ground loss into the tunnel. Case 2, the Evanston tunnels at a depth of 18 m, had consolidation resulting from both increases in mean stress and drainage into the tunnel. Over one half the surface settlement was caused by the long term settlement.

2.5 Damage criterion for assessing building distortion and damage.

The damage criterion of Figure 3 relates damage levels to the angular distortion and the lateral strain within distorting portions of the building. The equation for the state of strain at a point has been used to obtain the principal extension strain in the structure from the combination of angular distortion and lateral strain (Cording, et al, 2001). The principal extension strain boundaries shown in the plot of angular distortion vs lateral strain in Figure 3 have been adjusted slightly from that developed by Boscardin and Cording, 1989, in order to fit them to the relationship for the state of strain at a point. (Boscardin and Cording, 1989, described the relationship in terms of the distortion of a deep beam for the case of a length/height ratio, L/H, equal to one.)

![Figure 3: Damage criterion based on lateral strain and angular distortion](image-url)

* The results of two field cases and one numerical test are out of range

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<tr>
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<th>Damage level based on maximum crack width</th>
<th>Damage level based on field observation</th>
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Numerical tests

Field cases

Angular Distortion, $\beta$ ($\times 10^{-3}$)

- Constant Principal Extension Strain after Boscardin & Cording, 1989
- Reduced angular distortion due to side sway?

Figure 25 Damage level estimation and observed damage level

- Damage level based on maximum crack width [Burland et al., 1977]
- Damage level based on field observation [Boscardin and Cording, 1989]
In fact, with slight modification, the relationship has been expressed in terms of the state of strain at a point, as shown in Figure 3, and has broad application to the distortion of building walls over a wide range of L/H ratios. Each of the boundaries between damage levels (negligible, very slight, slight, moderate, and severe) represents a constant value of the principal extension strain. The description of the damage levels was developed by Burland et al (1977). The impact of a given distortion level and cracking will differ for different buildings, and depending on the details of their sensitivity and significance, and they should be evaluated on a case by case basis.

The state of strain within a point can be used to describe the average strain within a structural element or bay. The structural elements are strained by the ground movements acting along the base. The angular distortion, or shear strain, is equal to the average settlement slope across the structural element minus the tilt (Figure 4). The lateral strain at the base is equal to the extension of the base divided by the base length. These two values represent a single point on the plot of lateral strain vs angular distortion. A separate measurement can be made of the lateral strain at the top of the element.

The lateral strain at the top may increase due to a convex (hogging) soil profile. Bending effects can occur in the central portion of a structure with low height/width ratios impacted by a wide settlement profile. In this case, the distortion level includes the lateral strain at the top due to bending, as well as any shear distortions. However, for most of the observed settlement profiles, the portions of a structure impacted by ground movement have a relatively high height/width ratio and a low effective shear stiffness that they act as a deep beam. In such cases, significant bending and extension cannot develop at the top, near the end of the building, unless there are weaknesses or cracks extending up the wall, such as are formed by the junction between façade and bearing wall or by cracks that originally formed and extended because of the angular distortion of the wall (Fig 5). Joists and roof structures or cross walls perpendicular to the displacing wall may reduce the opening of cracks in the upper floor levels.
2.6  Effect of building stiffness on reducing distortions and strains.

In cases where the building is relatively stiff, the green-field ground movements will be modified, and the distortions of the structure will be less than those estimated assuming the structure conforms to the shape of the green-field settlement profile. Boscardin and Cording (1989) show a relationship between the axial stiffness of grade beams within a structure and the reduction in lateral building strain ($\varepsilon_h$) from the green-field lateral ground strain ($\varepsilon_{hg}$), where $E_gA$ is the stiffness and area of the grade beam foundation, $E_s$ is the soil stiffness, $H$ is the height of excavation or the length of the section of the foundation being strained, and $S$ is the spacing between grade beams (Figure 6). Large reductions in lateral strain result if the foundation has grade beams, reinforced wall footings, or structural slabs.

Several cases are presented in this paper in which there are no grade beams or structural slabs in the basement floor, but the main floor and upper floors are concrete and tied into the bearing walls. In these cases, lateral extension and cracking may be present at the basement or first floor level, but wall cracks narrow and close as they extend to the level of the concrete floors. Vertical cracks extending higher may be limited to construction joints or weak zones within the structure, such as stair wells or boundaries between buildings.

Fig. 7 shows the relationship between the shear stiffness of a masonry bearing wall building and the angular distortion, $\beta$, with respect to the change in ground slope, $\Delta GS$, between adjacent structural units (Son, 2003). The relationship was developed from a series of parametric analyses using the distinct element code, UDEC, in which the stiffness of the masonry blocks and the shear and normal stiffness of the mortar joints is modeled, and the joints can separate and slide when the stresses reach the strength of the mortar.
Figure 6: Effect of axial stiffness of foundation on lateral strain in structure

Figure 7: Effect of building shear stiffness on angular distortion

The effective shear stiffness, $G$, to be entered in the calculation of the relative shear stiffness in the abscissa of the plot, is a reduced value that takes into account the presence of window...
penetrations in the wall. The ratio of angular distortion to the change in ground slope, $\frac{\beta}{\Delta GS}$, increases approximately linearly with the logarithm of relative soil/wall stiffness. However, as strains increase, with respect to the tensile cracking strain of the mortar (represented by $DGS/\varepsilon_t$), and cracks between the masonry blocks extend and enlarge, the ratio of angular distortion to the change in ground slope, $\frac{\beta}{\Delta GS}$, increases dramatically and approaches one. The downdrag of the façade (FD) on the brick wall causes the ratio to increase from the case of no façade downdrag (NFD).

3 TYPES OF STRUCTURES

Most of the cases described in this paper are masonry bearing wall structures on shallow wall foundations in urban areas of the United States. Most were built in the 1800s and early 1900s.

Typically, in the early to mid-1800s, the multiple wyths of the brick walls, including the façade walls, were tied together with a combination of headers and stretchers. In the late 1800’s and early 1900’s, almost all bearing wall structures had alley, foundation, and interior walls of common bricks consisting of two or more wyths of stretchers tied together every 6 rows with a row of headers, whereas the building facades were faced with a single wyth of a running bond (stretchers, no headers) with common brick behind. Over the long term, the façade walls tend to be more susceptible to cracking or displacement due to deterioration and environmental effects than the common brick walls.

Brick-bearing wall townhouses typically have bearing walls perpendicular to the street with timber joists extending between bearing walls, with spans of approximately 6 m between adjacent brick bearing walls. The joists are seated in pockets, usually one wyth wide, The end of the joist may have a tapered (fire) cut (shorter joist length at the top) that allows it to fall out of the wall more easily if it burns in a fire and not cause the wall to collapse. The joists set on the seat, they are not tied to the wall.

Houses with wider spans between exterior brick bearing walls had intermediate timber bearing walls. Floors may have sagged inward toward the timber bearing wall because of shrinkage of the timber bearing wall after it was installed. This is the case for several of the historic houses in Washington DC, built in the early 1800’s. Door frames can be observed to have undergone shear distortion which is down toward the center of the building.

Commercial brick bearing wall structures have timber beams and posts supporting timber floor joists between brick bearing walls.

Monumental structures, such as the Masonic Temple built in 1870 in Philadelphia, had masonry bearing walls and floors consisting of I beams with jacked arches of brick between the beams.

In the early 1900s, many of the larger brick bearing wall structures, had cast reinforced concrete floors formed by pouring a T beam -- a concrete slab and beam formed by placing clay tile in the form to fill the space between concrete beams. The concrete floors, unlike the timber joists, were tied to the masonry walls.
4. EXAMPLES OF BUILDING RESPONSE: BRICK BEARING WALLS PARALLEL TO EXCAVATION

For bearing walls parallel to an excavation or tunnel, the wall distortions largely occur on the cross walls or side walls, and the relative stiffness relationships apply to those walls. Between the brick side walls, the stiffness of the cross-section will be controlled by the floor stiffnesses and their connections to the bearing walls, and to the infills between floors.

The other issue for brick bearing walls constructed in the 1800’s is displacement of the floor joists out of their seats, reducing the bearing area to the point that the joist falls and the floor collapses.

In the early 1900’s, concrete floors were used in many brick bearing wall structures, providing a connection to the masonry bearing wall. These frames are subject to sidesway.

4.1 Distortion of two brick bearing wall buildings on G-1 tunnel alignment

After the initial program of measurement of greenfield movements around shield tunnels, the Washington, D.C. Metro investigations continued with observations of the impact of ground movements on the distortion and damage to buildings located near tunnels and excavations. Most of the buildings were old, and sometimes historic, and were constructed with masonry bearing walls on shallow foundations. The structures were not reinforced and were sensitive to both lateral and vertical ground displacements. During this period, a pair of unoccupied buildings along the Washington Metro G-1 tunnel alignment, south of the Anacostia River, was instrumented with tape extensometers extending horizontally and diagonally at each floor in the structure, as well as tiltmeters on the bearing wall of the building (Fig. 8). Building 1 was located in a zone with relatively small lateral strain. Building 1 is separated from Building 2 so that most of its movement was tilt. Extension of the diagonals showed that a very small shear displacement took place in Building 1. Building 2 was located in the outer portion of the settlement profile. Small lateral strains developed at foundation level, and increased in upper floors because of formation of an open vertical crack between the bearing wall nearest Building 1 and the facade. Small displacement of the joists in their seats would have occurred as a result of the opening of this crack.

The building was modeled numerically using a distinct element analysis. The same settlement and lateral displacement profile was applied as was observed in the field, showed close correlation with the building distortions (Son, 2003). Opening of a vertical crack at the bearing wall of building 2 was achieved in the discrete element analysis when the floor loads in building 2 were reduced to the weight of the flooring and joists alone, with no live loads. This was the condition existing in the building, because the rooms were empty. When the analysis was first run, live loads of 10 psf were applied to the floors of building No. 2, and there was no opening of a crack between the bearing wall and the facade in the upper part of the building.
4.2 Distortion and damage to brick bearing wall structure with concrete floors, Evanston, Illinois.

Tunneling was carried out beneath an EW street in Evanston, Illinois, adjacent to a brick bearing wall structure with concrete floors, built in the early 1900's. The bearing walls of the building are parallel to the tunnel (Figure 9). The tunnel was 3.6 m in diameter, was advanced with a shield with a wheel excavator, and was supported with 100 mm steel ribs spaced 1.2 m on center and timber lagging, with a final cast concrete lining. The tunnel was located at a depth of 17 meters in a soft to medium strength clay, which was deposited during late glacial stages in Lake Chicago (an extension of Lake Michigan that covered most of Chicago and Evanston) and is often referred to as the Chicago Clay, the medium in which the Chicago Subway was built in 1938-1941 (Terzaghi, 1942a, 1942b).
Ground movement patterns and settlements typical of this reach of the tunnel alignment were obtained at an instrumentation test section (TS-3) located approximately 300 m to the west of the building, which was installed to monitor movements and pore-pressures in the soft to medium strength clays prior to passing under a railroad line (Srisirirjanakorn, 2005). The consistent tunneling method, similarity of the soil conditions, and similarity of movement patterns in this reach of the tunnel justifies using TS-3 settlement data for estimating the green-field ground movements at the building. The immediate and long term settlements at test section 3 are shown in Figures 10 and 11, respectively.
Figure 10: Immediate ground movements during shield tunneling at Test Section 3, Evanston, Illinois

Figure 11: Long term ground movements after shield tunneling at Test Section 3, Evanston, Illinois

At TS-3, immediate ground settlement at tunnel centerline was 30 mm, and the settlement zone extended from tunnel springline to the surface at an angle, b, of 38°, giving a half width, w, of the
settlement trough of 19 m from tunnel centerline. (The half width, \( w \), is determined as the location where a triangular settlement distribution of the same volume as the surface settlement volume, would intersect the surface, and \( w \) is 2.5 \( i \), where \( i \) is the inflection point.)

Additional long term ground settlement of 34 mm was measured at TS-3 over a period of 445 days. The additional settlement resulted from consolidation of the Chicago clay around the tunnel. The two primary causes of the consolidation were (1) drainage into the tunnel, through the relatively permeable initial lining of steel ribs and timber lagging, and (2) a permanent increase in the mean stress in the clay surrounding the tunnel caused by the pressures applied to the ground as the shield was shoved forward and as the lining was expanded against the ground behind the tail of the shield. The increase in mean stress was initially evidenced by the development of excess pore pressures in the clay surrounding the tunnel, in a zone concentrated within approximately 4 radii of tunnel centerline. Dissipation of the excess pore pressures and further reduction of pressure due to drainage over the following year resulted in the consolidation. The half width of the settlement trough was 23 m for the long term settlements, which was wider than the settlement trough that developed during tunneling because the zone of consolidation around the tunnel is wider than the zone of ground loss into the tunnel. (The immediate settlements as the shield passes and long term settlements due to consolidation should be evaluated separately in order to properly assess their zones of influence.)

The South Wing of the building is located in the tension zone of the settlement trough. The green-field ground surface settlement at tunnel centerline is estimated from TS-3 data to be 75 mm at tunnel centerline and 27 mm at the location of the South bearing wall, which is close to the measured settlement of 30 mm for the south bearing wall at the east end of the building. Lateral ground displacement at the edge of the building was estimated from test section measurements to be 29 mm (Figure 12).
Figures 12 and 13 show the pattern of displacements and the crack damage at Section A on the east facade wall resulting from tunneling. Diagonal shear cracks developed at ground level and above the windows closest to the South wall. Damage to plaster ceilings also took place in the 3rd floor room near the South wall. Further from the South wall, cracking due to lateral extension could be observed in the basement floor and walls. A vertical tension crack was located outside the zone of lateral greenfield displacements because the stiffness of the floors in the South Wing caused the tension crack to extend to the boundary between the South and Central wings. Cracks opened a total of approximately 17 mm. The lateral extension did not extend above the...
basement level because of the stiffness of the concrete floors

Cracks and out-of plane brick displacement above lintels unrelated to tunneling

Figure 13: Location of cracks on East Façade, Section A, Evanston, Illinois

Approximately 5 to 20 mm of outward displacement of the brick was present above many of the lintels on the façades (Figure 13). Further from the tunnel, in the courtyard, the facades had not been video-taped in the pre-construction survey and it was being alleged that the displacements were caused by tunneling. Reports were discovered indicating that recommendations for repair of the cracks had been made several years prior to tunneling. Furthermore, the outward displacement above the lintels was not compatible with the pattern and location of ground movements due to tunneling.

Figure 14 shows the profile of the building at Section B. In this section, the building is narrower and therefore most of the ground movement resulted in tilt and translation of the building with little distortion. Damage was limited to one or two hairline cracks.
4.3 Case 3: Concentrated settlement due to trench excavation adjacent to gymnasium with brick bearing walls and concrete floor

Settlement of bearing walls parallel to an excavation can induce shear cracks in attached cross walls and end walls, as illustrated in the case of a brick bearing wall building with gymnasiums on two levels (Figures 14 and 15). Settlement of the bearing wall was only 22 to 31 mm due to a small adjacent trench that was mistakenly excavated along the length of the wall in stiff clays below the footing level. The settlements caused large angular distortion and damage that was concentrated over a short distance of approximately 3 m along the cross walls and end wall.

Because the floor between the two gymnasiums was concrete and tied to the bearing wall it did not separate from the wall. Outward displacement of the bearing wall at footing level caused the wall to rotate about the concrete floor level between the two gymnasiums and produced a minor horizontal tension crack on the inside of the bearing wall in the upper gymnasium, approximately 2 m above the level of the concrete floor.
Figure 14: Case 3, Settlement of bearing wall of Gymnasium

Figure 15: Case 3: Shear cracks in cross wall due to settlement of bearing wall,
4.4 Case 4: Distortion and damage to a 3 story apartment building with timber floor joists. Bearing wall parallel to excavation, loss of bearing of floor joists.

A major concern for bearing walls parallel to an excavation is illustrated in Fig 16 and 17 where settlement and lateral displacement in the range of 40 to 75 mm occurred along the bearing wall. The lateral displacement at the building foundation wall caused loss of 58 mm of joist bearing so that the floor joists on all floors and at the roof had to be temporarily supported with posts and beams. The building was subsequently demolished. The average lateral strain between adjacent bearing walls exceeded 1/100, in the severe to very severe damage range.

Figure 16: Case 4: Distortion and damage to apartment building

Figure 17: Case 4: Plan and profile of distortion and damage
5. LATERAL SEPARATION OF WALLS AT UPPER FLOOR LEVELS

Tunneling beneath 7th St with an open face shield on the first phase of the Washington Metro project in the early 1970’s resulted in ground loss into the face and surface settlements on the order of 50 mm of the façade wall of a brick bearing wall building. The settlement resulted in diagonal shear cracks between windows located on the bearing wall adjacent to the façade, as well as distortion of the window frame (Figures 19 and 20). The shear cracks extended almost to the roof of the four story building and separated the façade wall from the bearing wall, causing an outward lateral displacement of 28 mm near the top of the façade wall. Other buildings along the street were similarly affected and facades were temporarily braced and tied back to prevent their collapse.

Independently, in the last few years, a series of distinct element analyses using UDEC were performed in which four-story buildings on elastic foundations were subjected to settlement and lateral displacement patterns. Individual bricks were modeled, assigning stiffness and strength values to the mortar between bricks. With shear distortion and lateral strain, the mortar was capable of cracking and the individual bricks could displace. As the settlements increased, the shear cracks widened and propagated upward between window levels until a continuous series of shear cracks between windows caused separation of the façade from the bearing wall and allowed outward lateral displacement of the façade wall. Settlements of 25 mm produced shear cracks and separation over the full height of the wall, with lateral displacements of 25 mm at the top of the wall. Once a critical settlement level was reached, the shear cracks would extend and coalesce and the lateral displacements at the top of the wall would amplify. Subsequently, the writer has observed cases in the field in which settlements of less than 20 mm have caused
shear cracks and outward lateral displacements at the top of the wall on the order of 3 to 4 times the settlement magnitude.

Figure 19: Washington Metro, 7th Street building, bearing walls perpendicular to tunnel line: Settlement due to ground loss into tunnel causes outward lateral displacement of 28 mm near connection of façade wall with bearing wall.

Figure 20: Progression of shear cracks, 7th Street building
Figure 21: Distinct element analysis on left shows progression of shear cracks upward and opening of 27 mm-wide crack adjacent to façade wall.

Separation of either façade walls or bearing walls can occur due to settlements caused by excavation or tunneling. Lateral displacements can occur even without the propagation of shear fractures as was described in the previous paragraphs. Pre-existing weaknesses on or between the walls or at the connection between the wall and floors can cause separation and lateral displacement at the top of the wall.

When undergoing ground settlement, brick walls may displace outward when subjected to

1. Low floor loads on joists seated on the bearing wall

2. Intermediate walls, such as cross walls or non bearing walls that pick up the load of the floor joists as the bearing wall settles causing the joists to become unloaded on the bearing wall.

3. Water damage and rotting of joists in their seats.
4. Uncontrolled drainage from roof, water damage, deterioration, lack of maintenance or repointing of mortar joints in the side wall.

5. Poor or deteriorated connection of façade wall to bearing wall

6. Angular distortion and shear cracks extending between window openings to upper portion of wall.

6. FRAME STRUCTURES

Figure 22 illustrates the results obtained with a 2-D finite element analysis which modeled both the excavation wall, the soil mass and the adjacent building frame using a hypo-plastic constitutive model for sand (Ghahreman, 2004). Side sway of the frame reduced the distortion of the bay nearest the excavation, and resulted in an almost equal and opposite distortion of the third bay. Angular distortions at each floor level above the lower floor, in a given bay, were similar. From the angular distortion, the change in bending moment in the beam due to the distortion can be estimated. The first floor columns were not laterally tied with grade beams and were sensitive to the lateral displacements at the foundation level, a condition similar to that encountered in the Heathrow Car Park (Powderham and Burland (2004) where the first floor columns were damaged. Direct analysis of the bending in the columns can be made by applying the displacement to the base and subtracting the sidesway at the top of the column.

Side sway causes the angular distortion to be spread across more bays and results in lower angular distortions than occur in a brick bearing wall in which the shear cracks and displacements are concentrated in the first section of the wall and the portions of the wall further from the first section may tilt but do not undergo significant distortion. For this case damage often concentrates near the wall, in the first building unit and sidesway is minimal, and angular distortion is approximately four times the deflection ratio. For frames, the sidesway can reduce the angular distortion to twice the deflection ratio.

Wood frame structures with brick veneer have been observed where settlement has caused the frame to side sway and has resulted in small lateral displacement of the wood frame on the far wall as evidenced by a small separation of the window frame from the brick veneer.

Excavation and installation of underpinning beneath a 12-story hotel with brick bearing walls resulted in a small amount of side sway which caused the far right wall in Figure 23 to displace and along with it, the adjacent bearing wall of the historic structure built in the early 1800’s. Lateral displacements were concentrated in the stairwell and hall on the left side of the historic structure, which forms a natural structural weakness. The side sway of the left wall of the house resulted in small cracks in the cross walls and crown molding in the hall (Figure 24), and sticking of a door at a cross-wall into the hall (Figure 25). The lateral strain was superposed on previous distortions that the building had undergone in which floors had sagged over time toward the center of the building, most likely a result of shrinkage of the central timber bearing wall. The door frame into the hallway had been racked by this previous displacement. Recognition of the pattern of distortion of the structures confirmed that the excavation had caused the side sway, but also showed that the needed repair to the historic structure was primarily patching of the plaster and repair of finishes in the hallway of the historic building. It was important to
demonstrate that the damage was not caused by any instability or settlement of the foundation of the structure and there was no need for replacement or repairs to the building’s bearing wall.

Abaqus FEM with Hypo-plastic Soil, Concrete Frame Excavation
Depth: 25 m
Displacement magnification factor: 100

Figure 22: Side sway of elastic frame on a soil mass adjacent to a braced excavation
Figure 23: Excavation to the left of the hotel causes small settlement of hotel foundation, resulting in sidesway of hotel and the left wall of the historic structure.

Figure 24: Cracking of plaster in hallway on left side of historic structure
7. CONCLUSIONS

In the cases illustrated, the structures themselves served as indicators of the type and causes of distortions and damage that were imposed on them. The ability to observe and read the building response is aided by an understanding of the chain of relationships that extends from the excavation or tunnel and adjacent ground loss or ground movement, to the distribution of ground movement and volume change through the soil mass, to the interaction of the ground with the building, and then to the building distortion and damage. To properly assess the building behavior — both distortion and damage — it is necessary to understand not only the ground movement patterns but also the building’s structural characteristics and finishes: how the building was built, maintained, and repaired. Most of the structures that are affected by excavation and tunneling are on shallow foundations and are older structures, which means that they have a long history of repair and, in some cases, neglect. Some are historic structures or within historic districts. Often the effects of excavation and tunneling are superposed on pre-existing distortions and deterioration. In many cases, pre-existing conditions are separate from and unrelated to the excavation- or tunnel-induced damage. It is most common and natural for residents and owners of buildings to observe and attribute pre-existing conditions in their building to the effects of the adjacent excavation and tunneling. Pre-construction photographs and video are important in resolving such issues. In addition, a logical evaluation and description
of the limits and magnitude of ground movements and building distortions due to excavation or tunneling and early recognition and acceptance of responsibility for excavation or tunneling induced-damage can be most helpful in avoiding disputes.

.8. REFERENCES


Terzaghi, K. (1942a) “Liner plate tunnels on the Chicago (Ill.) Subway, ASCE Proceedings, V. 68, No. 6
