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# ASSESSMENT OF EXCAVATION-INDUCED BUILDING DAMAGE

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## ABSTRACT

Ground movements during excavation have the potential for major impact on nearby buildings, utilities and streets. Increasingly ground movements are controlled at the source. They are assessed by linking the ground loss at the excavation wall to the volume change and displacements in the soil mass, and then to the lateral strains and angular distortion in structural bays or units, and are related to damage using a damage criterion based on the state of strain at a point. Numerical and physical models of excavation-induced building damage were used to vary parameters and develop procedures for assessing distortion and damage. Examples of building distortion and damage are presented for brick bearing wall structures of the 1800's and early 1900's, as well as later frame structures, that illustrate how geometry, era of construction, stiffness, and condition influence building response to ground movement.

## INTRODUCTION

Increasingly, ground movements are controlled at the wall of the excavation, with less reliance on underpinning or ground modification, although such procedures may be used to reduce the risk of impacts from ground movement. The impacts are assessed by linking the magnitude of ground loss at the source to the volume change and lateral and vertical displacements in the soil mass, and then to the lateral strains, angular distortion, and resulting damage in the structure, using a damage criterion based on the average state of strain in a structural bay or unit.

In this paper, emphasis is placed on relating the given ground displacements to the building response. The settlement slope, tilt, and change in ground slope across the structure serve as a basis for assessing angular distortions, which will be modified by the geometry, condition, stiffness, and strength of the building. Added to this are the lateral ground strains imposed on the building foundation. The distribution of lateral strains throughout the structure will be affected by bending or rotation at the foundation level and modified by the lateral stiffness and variation in stiffness of the structure. Grade beams and continuous, reinforced foundations will limit – or eliminate -- lateral strains in the base of the structure. At upper levels of a structure, structural frames and floors tied to the walls will limit lateral strains in-

duced by both lateral ground strain and by bending. Structural weaknesses such as construction joints, windows, stairwells, and poor connections at façade walls and between walls and floors will allow concentration of lateral strains.

The damage criterion is based on the average state of strain, determined from lateral strains and angular distortions near the base and in upper levels of a given structural bay or unit. The sensitivity of the structure to damage and the significance of the structure must also be considered in evaluating the impact of the damage and the cost of pre-emptive measures or required repairs.

Numerical and 1/10<sup>th</sup> scale physical models of brick bearing walls adjacent to excavation walls in sand were used to vary parameters and develop procedures for assessing building distortion and damage. Examples are presented of building distortion and damage for brick bearing wall structures of the 1800's and early 1900's, as well as later frame structures, that illustrate how the geometry, era of construction, stiffness, and condition of the building influence its response to ground movement.

## **SOURCES OF GROUND MOVEMENT**

### **Lateral displacement of excavation wall**

The lateral displacement of the excavation wall that develops during excavation is largely controlled by relative soil/wall stiffness, which is a function of the EI of the wall and the distance, L, excavated below a strut or tieback level before setting struts or tiebacks at the next level. Distortion of adjacent buildings can be controlled by placing a stiff wall with small enough vertical spacing between brace levels and by limiting the depth of excavation below brace levels before installing the tiebacks or struts. In sands and stiff clays, a relationship relating wall/soil stiffness ( $E_s L^3 / EI$ ) to normalized lateral wall displacements can be used where  $E_s$  is the secant Young's modulus of the soil in the stress range of interest. Numerical analyses provide a means of assessing the effect of soil/wall stiffness on lateral wall displacement. Papers describing excavation wall displacements in clays as a function of wall stiffness and factor of safety against basal heave due to excavation include Clough and O'Rourke, 1990 and Hashash and Whittle, 1994.

Lateral wall movement patterns include cantilever deflection due to excavation prior to placing the first brace, and bulging deflections that develop below brace levels as the excavation is deepened. Mueller, et al (1994), based on model tests of 1/4-scale tieback walls in sand, observed a third lateral deflection pattern when the toe depth and capacity of the soldier pile was inadequate and the vertical component of tieback force caused penetration of the pile. The resulting lateral wall displacement at the tieback level was  $s_{\text{wall}} \tan a$  where  $s_{\text{wall}}$  is wall settlement and  $a$  is tieback angle.

A standard approach during excavation is to monitor the lateral and vertical settlement at the top of the soldier piles and on the adjacent building wall. Inclinometers installed in the wall provide a profile of lateral displacements over the wall height. To obtain a complete record of the causes of lateral wall deflection, measurements should be made every time the excavation is deepened and lateral braces are installed. To measure deep-seated movements that may occur below the tip of the pile, inclinometer casing is extended below the bottom of the wall.

## **Displacement due to wall installation**

Ground losses can also occur due to installation of the vertical wall elements, such as excavation of a slurry trench for a concrete diaphragm wall, or installation of timber lagging in a soldier pile wall. Lateral displacements immediately adjacent to slurry wall installation have typically been reported in the range of 0 to 20 mm, and are dependent on soil type, slurry density and panel width. Local ground losses due to installation of the excavation wall are of greatest concern when building foundations are immediately adjacent to the wall, which is commonly the case where building walls are set on the property line. In this case, underpinning may be used or an excavation wall provided that will limit movements of the adjacent foundation during wall installation, as shown for the case in Figure 1b. Inclinometers and settlement points placed adjacent to the excavation wall, prior to its installation, can be used to record lateral displacements due to both wall installation and, later, excavation.

## **Control of excavation wall movement**

Early experience on the Washington Metro with excavation walls of soldier beams and lagging in medium-dense sands and stiff clays, resulted in maximum settlements ranging between 0.1% to 0.3% of the excavation height (O'Rourke and Cording, 1975). Most walls were supported with cross-excavation struts. The larger settlements developed when excavation was extended far enough below strut levels before installing the next strut to allow passage of excavation equipment. Additional displacements also developed as the station structure was built and backfilled and the struts were removed. For an 18-m-deep excavation at G St, lateral movement averaged 15 to 20 mm and produced vertical settlements of 35 mm at a distance of 3 m behind the wall. The volume of lateral wall displacement and the volume of surface settlement were approximately equal at 0.4 cu m/m.

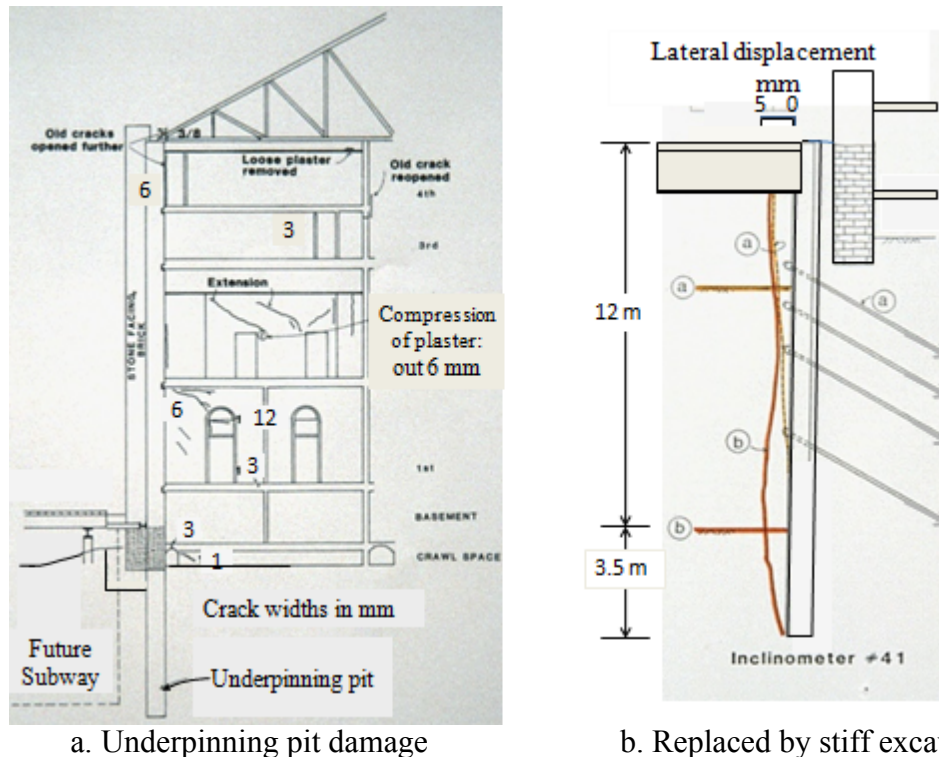
Tighter control of excavation wall displacements has been achieved on other projects, and is particularly important in order to limit damage to decorative finishes in historic buildings and other sensitive structures.

Such a case was the historic Masonic Temple in Philadelphia, built in 1870 (Figure 1). It is a 24-m- high masonry bearing-wall structure with interior plaster finishes and decorative murals. Initially, in 1975, the plan was to support the exterior bearing wall with pit underpinning prior to excavating a cut and cover structure for an adjacent subway. As the initial pits were being installed, cracking of plaster walls developed in the bay adjacent to the bearing wall, on all floor levels (Figure 1a). Crack patterns showed both diagonal shear cracks above doorways as well as vertical cracks between the bearing wall and adjacent cross walls. Opening of pre-existing cracks was observed in non-public areas. Estimated angular distortion was  $1.5 \times 10^{-3}$  (very slight to slight damage, point A in Figure 2).

To prevent further distortion, the underpinning operation was terminated and a 12-m high wall was installed adjacent to the footing with sufficient stiffness to limit lateral wall displacements and prevent further settlement of the bearing wall. It consisted of tangent H piles and tiebacks at close vertical spacing (Figure 1b). A row of tiebacks was installed immediately below the bearing wall foundation before excavating below foundation level. The next 2 tieback levels were at close 2-m vertical spacings.

The wall was designed to have an average lateral displacement of 3 mm, using beam-on-elastic-foundation and finite element analyses correlated with the data from the more flexible excavation walls in Washington, DC.

The measured lateral displacements were in the anticipated range, with a maximum of 5 mm and average of 3 mm (0.25% of excavation wall height, H), and displacement was held to zero when excavating the first 2 m below footing level. Lateral wall displacement volume was 0.05 cu m/m. There was no further extension of damage in the building.



**Figure 1. Masonic Temple: a) Damage due to excavation of underpinning pits and b) displacement of tied back tangent pile wall, with no further damage.**

## PATTERNS OF GROUND MOVEMENT

### Settlement at the ground surface

The volume of the surface settlement trough can be estimated from the volume of lateral wall displacement. In dense sands, the volume of the surface settlement will be less than or equal to the volume of the lateral wall displacement but they can be assumed to be equal. For soft clays, the volume of the settlement trough will initially be approximately equal to the lateral displacement volume and will increase with time due to drainage and consolidation of the clay.

The boundaries of the settlement profiles for excavations in clay were described by Peck (1969). Observed settlements adjacent to a series of excavations in soft clay, sands, and stiff to very stiff clays were summarized in Clough and O'Rourke (1990). The envelope of ground displacements is shown to extend laterally

a distance of 2 to 2.5 the excavation height. For sands, the envelope of the settlement zone is shown to be a triangular region extending laterally from the excavation wall a distance of twice the excavation depth,  $H$ . Deep seated movements in soft clays will cause displacements to extend to greater distances, which are a function of the height of the zone of lateral displacement extending below the excavation bottom.

Field measurements show that an individual settlement profile typically exhibits a decreasing slope with distance away from the excavation wall. The maximum settlement is near the excavation wall, although there may be a reduced settlement close to the wall due to the soil shear stresses developed on the excavation wall, which, if it has good bearing, will settle less than the soil mass. Large scale-model tests in sand also show a similar pattern (Mueller, 1994, Laefer, 2001).

A parabola can be used to approximate the settlement profile so that structural distortions can be estimated. Often, the measured settlement profile adjacent to an excavation is not precise enough to obtain accurate measure of changes in slope and curvature, but the parabola provides a sense of the parameters controlling distortion. Settlement with respect to the maximum settlement is simply  $\delta/\delta_{\max} = (1-x/L)^2$  where  $x$  is distance from excavation wall and  $L$  is the length,  $L$ , of the settlement profile. The settlement slope decreases with distance,  $x$ , and is equal to  $2(1-x/L)$ . A unit or bay of a structures distorting with a parabolic settlement pattern between 0 and  $x = L/2$  would have an average settlement slope of  $1.5 \delta_{\max}/L$ . Although the parabola can be assumed to extend a distance of  $L = 1.5$  to  $2H$  from the excavation in sands and stiff clays, the displacements beyond  $0.75 L$  are likely to be in the range of the precision of the survey measurements (less than 1.5 mm for a maximum settlement of 25 mm near the wall)

### **Lateral displacements of the ground surface**

For the cantilever deflection of a braced excavation, which occurs prior to installing the first brace level, lateral displacement of the ground surface will be high, on the order of 1 to 1.5 times the vertical displacement. For the bulging displacements that develop as excavation proceeds below strut and tieback levels, lateral displacements at the surface will be on the order of 0.5 to 1.0 times the vertical. (Milligan, 1974), O'Rourke, et al, 1977, Clough and O'Rourke, 1990.

Measurements of lateral and vertical displacement were made for model excavation walls in sand. Wall height was 2 m for a  $1/4$  scale wall constructed of soldier beams and steel lagging (Mueller et al, 1994) and 1.8 m height for  $1/10$  scale wall of sheet steel (Laefer, 2001).

For the bulging displacements, the field and lab data show that the vectors of near-surface soil displacement are steepest near the wall and flatten further from the wall. To estimate lateral displacements from the vertical settlement profile, it is recommended that  $0.5 \delta_l/\delta_v$  be used near the excavation wall and that it be increased to 1.0 at and beyond a distance of  $0.75 L$  where  $L$  is the length of the settlement profile.

With deep-seated displacements on weak, flat-lying clay seams or sheared surfaces extending behind the excavation, lateral displacements will predominate and can be concentrated at lateral distances well in excess of the excavation depth,  $H$ . In several projects, large ground movements did not develop until the braced excavation approached full depth. Deep-seated movements on weak layers have caused lateral displacement of the overlying ground mass and produced opening of cracks at

distances behind the excavation from 1 to 3 times the excavation depth, H. In these cases, significant cracking did not develop near the excavation wall.

## BUILDING DISTORTION AND DAMAGE

### Damage criterion for assessing building distortion and damage

The damage criterion presented in Figure 2 compares damage levels to the angular distortion and lateral extension strain that develops within a structure due to lateral and vertical ground movements acting on the structure foundation. It is applicable to a full range of building geometries and distortions and strains, and is not limited to a single value of building length/height ratio (L/H). The relationship gives the state of strain at a point, which is used to describe the average strain within a structural bay or unit. Each of the boundaries between damage categories represents a constant principal extension strain, determined from the combination of angular distortion and lateral strain. (Cording et al, 2001). The structure is strained by the ground movement acting along its base (Figure 3). The angular distortion, or shear strain, is equal to the average settlement slope minus the tilt of a structural bay or unit. The lateral strain at the base is equal to the extension of the base divided by the base length. Separate values of lateral strain may be estimated for the lower and upper portions of the building unit. In the upper portions of the building, lateral strains may reduce due to the stiffness and restraint provided by the upper floors, or, conversely, they may increase due to bending or rotation for a convex (hogging) soil settlement profile, and will concentrate in areas where the building is weak in tension.

The state of strain at a point criterion was developed from, and is almost identical to, the criterion developed by Boscardin and Cording (1989); There is a only a minor adjustment in boundaries between damage levels so that each boundary represents a constant value of maximum principal extension strain, rather than the relationship for a deep beam with L/H =1. Additionally, as recommended by Burland (1995) the zone of moderate damage is not described as moderate-severe.

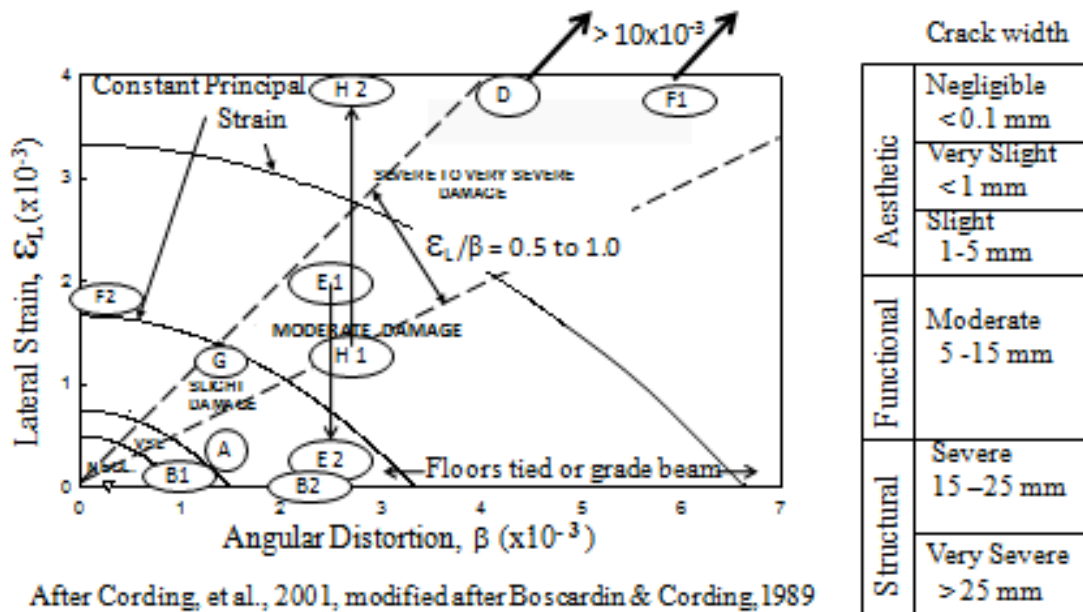
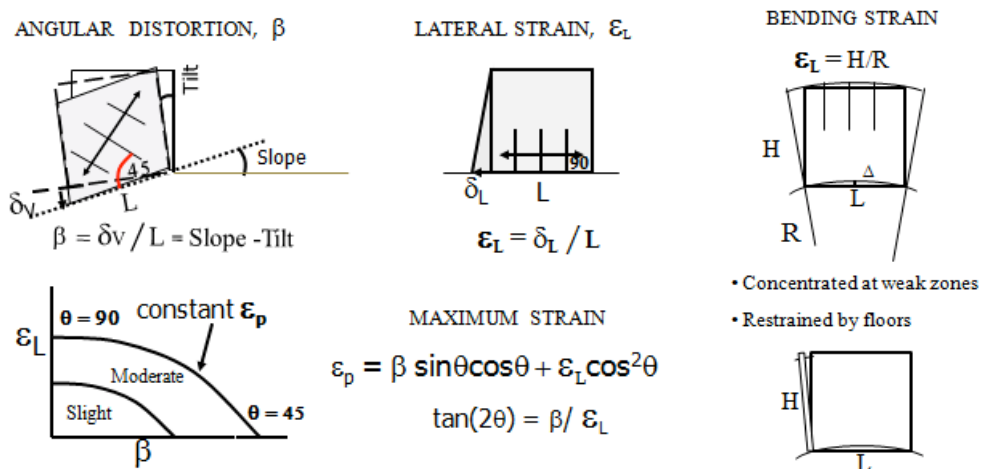


Figure 2: Damage criterion based on state of strain at a point

Burland and Wroth (1974) used beam theory to describe the effect of the length/height ratio ( $L/H$ ) on distortions for different ratios of Young's modulus to shear modulus ( $E/G$ ) and provided the relationships for brick and mortar structures with a higher ratio of  $E/G$  than an elastic continuum, and therefore shear strains would be critical for larger  $L/H$  ratios than would be predicted for an isotropic elastic continuum. From their observation of bending cracks in upper levels of the historic Westminster cathedral due to excavation of an adjacent car park, they concluded that criteria should be based on bending, not shear distortion alone, as had been described by Skempton and MacDonald (1956).

Boscardin and Cording (1989) added to this relationship the lateral strain imposed by the ground movement. They observed that, for many settlement profiles adjacent to excavations and tunnels, the portions of a structure impacted by ground movement have a relatively low length/height ratio and a low effective shear stiffness so that they act as deep beam, and shear distortions control damage. They set damage levels for the lateral strains to correspond with criteria used by the National Coal Board (U.K) for lateral displacements imposed on structures due to deep coal mine subsidence where the settlement profile is so large that buildings are subject to lateral strain and tilt, and not angular distortion. In setting damage levels for angular distortion they considered the relationships developed by Skempton and MacDonald (1956) for buildings settling under their own weight.



**Figure 3: Angular distortion and lateral strain in a structural element due to ground movement**

The relationships for bending of a continuous beam provides an estimate of damage due to bending in the central portion of structures distorted over a low  $L/H$  ratio that are weak in tension. However, for a deep beam that is continuous and elastic, significant bending and extension cannot develop near the façade, whereas cracks and lateral extension in the upper portion of the structure often are concentrated near the façade wall along pre-existing weaknesses, such as the boundary between a façade wall and a bearing wall, or along shear cracks that have extended up the wall because of large angular distortion. The equations for bending of an elastic continuous beam are not applicable to this case. One way to assess the potential lateral strain at the top of the wall is to determine the radius of curvature of the settlement trough for the unit of the building that is expected to have pre-existing cracks or joints or a

low enough tensile strength that a crack can form and separate, and then calculate the maximum lateral displacement at a weak point from the strain in the upper portion of the structure (Figure 3). The procedure is similar to the strain superposition method proposed by Boone (1996, 2008). However, once the façade wall of a building separates, the displacement magnitude becomes unpredictable. The primary effort should be to limit ground movements at the base so that lateral strains do not amplify in the upper portion of the structure, as described in Section 4.7.

The description of each of the damage categories was developed by Burland et al (1977). It was developed for brickwork and stone masonry and can be applied to plaster work, and was not intended for reinforced concrete structural elements. Crack sizes for each of the categories are indicated in Figure 2, but Burland (2008) states: “The strong temptation to classify the damage solely on crack width must be resisted. It is the ease of repair which is the key factor in determining the category of damage.” The categories are summarized as follows:

Aesthetic damage, including very slight to slight damage, affects interior finishes. Slight damage may require some re-pointing of visible masonry cracks. Re-decoration may be required.

Moderate damage affects building function and results in masonry cracks requiring patching and may require some re-pointing of brickwork.. “Doors and windows stick, service pipes may fracture, and weather-tightness is often impaired.”

Severe and Very Severe categories result in structural damage. Severe damage involves...”breaking-out and replacing of sections of walls, especially over doors and windows, distorted windows and frames, sloping floors, leaning or bulging walls, some loss of bearing in beams and disrupted service pipes.” “Very severe damage often requires...partial or complete rebuilding, beams lose bearing, walls lean and require shoring, and there is a danger of structural instability.”

It is understood that the impact of a given distortion will differ for different buildings, depending on their sensitivity and significance, and they should be evaluated on a case by case basis. As Burland (2008) notes, the descriptions relate to standard domestic and office buildings and may not be appropriate for a building with valuable or sensitive finishes. (Plaster finishes such as crown moldings and wall murals in historic structures are of particular concern in the aesthetic damage range.)

### **Physical and numerical models of ground movement and building damage**

Relationships between ground deformation and building distortion and damage were obtained in research programs conducted at the University of Illinois at Urbana Champaign. The work consisted of a combination of physical modeling and numerical analysis, as well as correlations with field measurements and case histories, for brick-bearing wall and frame structures adjacent to excavations.

Physical modeling of excavation walls was first carried out in a large model test pit constructed of segmental blocks, designed and utilized by M. Hendron for testing of tied back and soil nailed excavation walls. It was then used in a program of testing of ¼ scale (1.8 m high) excavation walls with single and double tiers of tie-backs (Mueller, et al, 1994). The test pit was moved to a new University of Illinois test facility, Schnabel Laboratory, where, for the first time, the testing program combined the modeling of excavation walls with adjacent building walls. Tied back excavation walls in sand were constructed adjacent to 600-mm-high, 2-story brick

bearing walls and frame walls in order to observe the effect of ground movement patterns on building distortion and damage (Laefer, 2001). The strength and stiffness of the buildings were scaled with the 1/10 scale structure dimensions. The results of the model tests produced realistic settlement and lateral displacement patterns in the ground. Down-drag of the façade walls against the bearing wall caused concentration of shear distortions and cracking between the windows nearest the excavation wall. The data was correlated with the numerical models, which were then used to conduct parametric studies.

Two types of numerical analyses were conducted. Distinct element analyses (UDEC) were conducted of brick bearing walls in which each brick was modeled as a block element and the mortar was modeled as the contact shear and normal stiffness and strength between blocks (Son, 2003). A parabolic pattern of ground settlement and lateral displacement was imposed on a system of elastic springs and contact elements modeling the soil at the foundation level of the structure. A series of runs were conducted to correlate the numerical results with the physical model tests. Additional runs were conducted using the dimensions of full-scale structures. Finite element analyses were conducted in which the full excavation, excavation wall, sand mass and adjacent building were modeled (Ghahreman, 2004). The two and three-dimensional numerical models used an Abaqus platform and a hypo-plastic constitutive model that produced displacement patterns consistent with field observations.

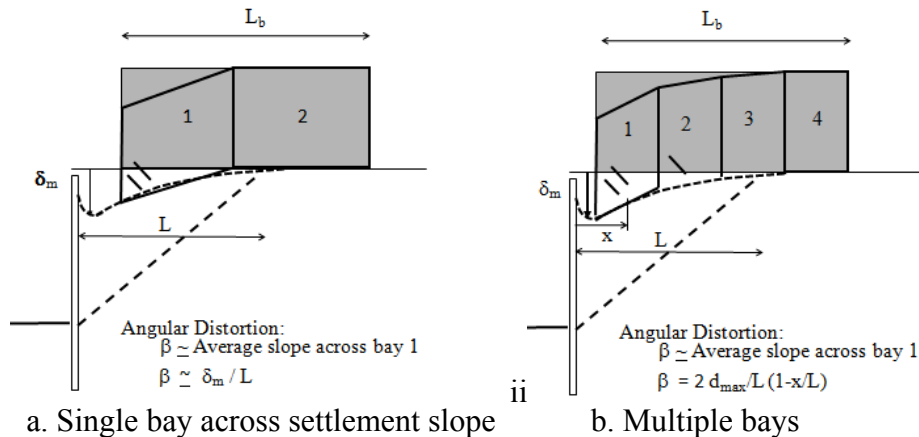
In both the physical and numerical models, lateral and vertical displacements at the top and base of the wall were recorded at four sections along the length of the building wall. From this data were obtained the tilt and angular distortion and their variation along the length of the wall, as well as lateral strains at both the base and top of the wall. The tilt and bending could not be determined from a settlement profile alone, but required the measurement of lateral displacement over the height of vertical sections along the wall. It was apparent that the tilt of the bearing wall structure was not simply the average slope of a chord extending over the settlement profile, as is the assumption when using beam theory, but was usually a lesser value, which depended on the distribution of the ground displacements beneath the building, the effect of façade downdrag, building stiffness, and mortar cracking.

The more flexible bearing wall had a deflection pattern close to the imposed soil displacements, whereas stiffer bearing walls would have a flatter settlement profile and smaller imposed angular distortions with respect to the change in ground slope. Once cracking developed in the walls, the angular distortions would increase, approaching the ground settlement slope or change in slope.

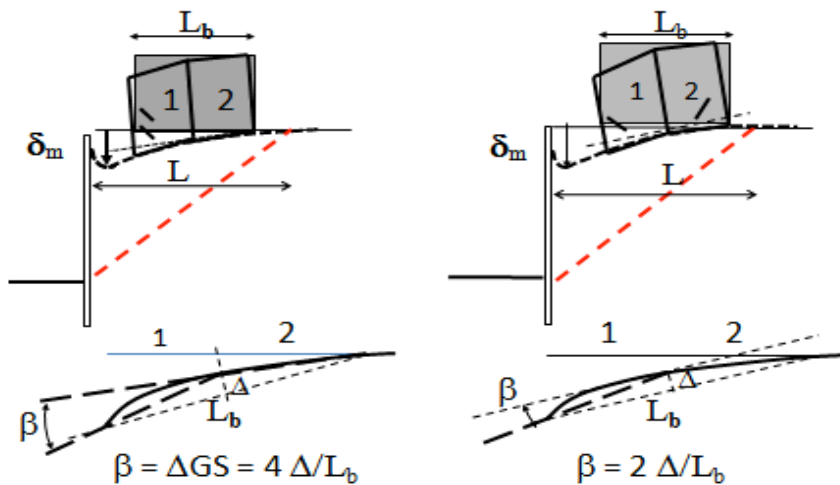
### **Estimating angular distortion from slope, change in ground slope, and tilt**

Figures 4 and 5 illustrate the method for estimating angular distortion for a building flexible enough to settle with the soil profile. For a building extending beyond the outer portion of the settlement profile, with the first unit spanning a significant portion of the settlement profile, there is no tilt, and the angular distortion, or shear strain, is approximately equal to the average slope of the settlement trough (Figure 4a). In Figure 4b, the structure also extends beyond the settlement profile but consists of several units that have different slopes on the parabolic settlement profile. Unit 1 has an angular distortion equal to approximately 1.5 times the average ground slope,  $\delta_{\max}/L$ , whereas Unit 2 has an angular distortion equal to the ground slope that is approximately 0.5 times  $\delta_{\max}/L$ . Thus, damage levels for unit 1 would be greater in the case of Figure 4b than the case of Figure 4a.

For cases in which the building is narrower than the settlement profile, as in Figure 5, the slope minus the tilt is a measure of the angular distortion. In Figure 5a, all the distortion is concentrated in unit 1, either because of the down drag of the façade wall on unit 1, increased curvature under unit 1, or because unit 1 has a lower shear stiffness than unit 2. As a result, the slope of unit 2 represents the tilt, and the change in ground slope between unit 1 and 2 is the angular distortion.



**Figure 4: Building moves with ground and extends beyond settlement zone**



**Figure 5: Buildings moves with the ground, Tilt reduces angular distortion.**

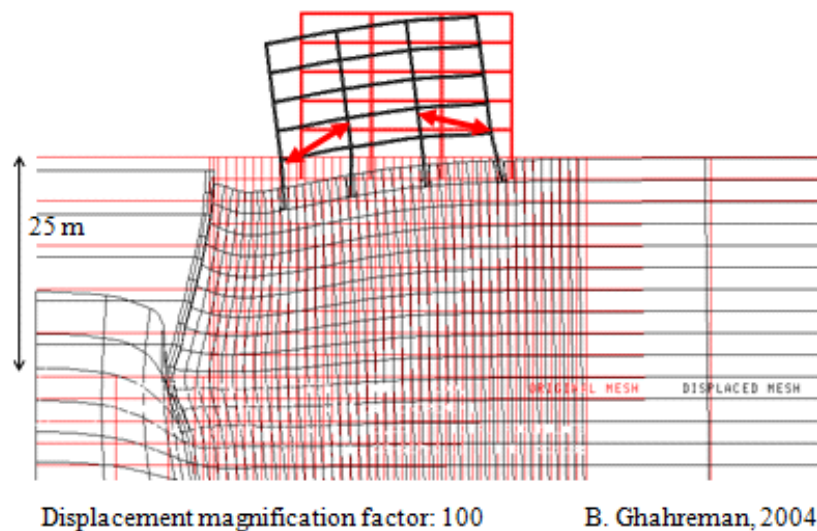
In Figure 5b, sidesway of units 1 and 2 results in increased tilt and reduces the angular distortion in unit 1. At the extreme, it could reduce the angular distortion in unit 1 to one half of the case in Figure 5a, and cause an equal and opposite angular distortion in unit 2.

A beam analysis is consistent with the assumption of Figure 5b. In this case, the tilt of the structure is assumed equal to the slope of the chord extending across the settlement profile. The slope of the chord is considered in much of the literature

to represent the structure's tilt, and damage criteria based on the deflection ratio,  $\Delta/L$  make this assumption. However, the actual tilt should be measured from the tilt of vertical sections within the structure.

Although the angular distortion,  $\beta$ , changes by a factor of 2 from Figure 5a to Figure 5b, the deflection ratio,  $\Delta/L$  remains the same because it does not consider the difference in tilt between the two cases and will predict the same level of damage for both cases. The deflection ratio damage criterion proposed by Burland (1995) based on beam theory is consistent with the damage criterion based on angular distortion (Figure 3) for the conditions described in Figure 5b but it underestimates the damage level for the conditions described in Figure 5a.

The series of UDEC numerical analyses by Son (2003) showed values of  $\beta$  that ranged from the conditions in Figure 5a ( $\beta= 4\Delta/L$ ) to Figure 5b ( $\beta= 2\Delta/L$ ). The distortions of Figure 5a occurred with the brick bearing walls in which there was downdrag of the façade wall and where cracking developed in the structure. The symmetrical distortions of Figure 5b were more characteristic of elastic beams and uniform, open, continuous frames that are narrow enough that they will sidesway, as is shown in the finite element analysis by Ghahreman (2004). Sidesway of the frame caused shear distortions within the third bay that were approximately equal and opposite to those in the first bay (Figure 6). In this case, in the absence of a continuous foundation or grade beam, lateral ground displacements caused significant bending strains in the columns between the foundation and the first concrete floor level. Such a condition caused cracking and structural damage to the columns at foundation level of a reinforced concrete parking structure at Heathrow Airport due to lateral displacements generated by adjacent tunneling (Powderham et al, 2004).



**Figure 6 Distortion of concrete frame adjacent to excavation in sand, finite element analysis with hypo-plastic soil model.**

Finno, et al (2005) described and analyzed a Chicago school building in which distortions due to adjacent excavation in clay were concentrated in the first bay of the structure, consistent with the pattern of Figure 5a. The brick bearing walls were clad with limestone and floors were concrete, which limited lateral extension strains in the building. Shear strains and cracking were concentrated in the first bay, which was on the steeper portion of the settlement slope. Damage consisted of shear

and horizontal cracks, typically less than 3 mm, as well as some distortion of doorways, requiring replacement. The settlement slope beneath the first bay, after consolidation of the clay took place, was  $2.7 \times 10^{-3}$ .

As noted by Boone (2008), analysis of the Chicago structure as a beam, using the beam analysis with the deflection ratio,  $\Delta/L$ , underestimates the actual damage level. The beam analysis assumes that the chord of the settlement across the structure represents tilt, and distortions are equal and opposite at the ends of the beam as illustrated in Figure 5b. Based on this assumption, the tilt is relatively high ( $1.2 \times 10^{-3}$ ), which, when subtracted from the bay 1 settlement slope of  $2.7 \times 10^{-3}$ , results in an angular distortion of  $1.5 \times 10^{-3}$ , in the very slight range (Point B1 in Figure 2).

The distortion pattern was more typical of Figure 5a, where unit 2 tilts but does not distort. Although there is no information on the actual tilt of the structure, it appears from the building sections and the steeper settlement profile near the excavation wall that distortions would be concentrated in the first bay of the structure and that the tilt would be represented by the slope of the 2nd and 3<sup>rd</sup> bays. Their tilt was relatively low ( $0.3 \times 10^{-3}$ ), which when subtracted from the settlement slope of  $2.7 \times 10^{-3}$  in bay 1 results in an angular distortion of  $2.4 \times 10^{-3}$  (Point B2 in Figure 2), in the slight damage range, consistent with the observed damage.

Structural analysis of the building confirmed that the distortions were concentrated in the first bay. Finno, et al, used a laminate beam model to analyze the structure in which the floors of the building acted as diaphragms and shear displacements predominated, and obtained results close to those that were observed for the onset of cracking. Boone analyzed the structure using his strain superposition approach and also obtained results consistent with the observed damage.

### **Reduction in lateral strain due to building stiffness**

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) give a relationship between the axial soil/beam stiffness of grade beams and the reduction in lateral building strain from the green-field lateral ground strain. Large reductions in lateral strain imposed by the ground will result if the foundation has grade beams, reinforced wall footings, or structural slabs on grade. In such cases, the lateral strain across the structural unit can be assumed zero. Additionally, the lateral strain imposed by the ground or by bending in the upper portions of a building will be reduced if the upper floors are reinforced and tied to the walls. Such a condition existed in the case of the school described in Section 4.3 (Finno, et al, 2004), and in the case of a 3-story apartment building in Evanston, Illinois constructed in the early 1900s, described in Section 4.6. In both cases shear distortions developed in the portion of the building nearest the excavation or tunnel subjected to the greatest angular distortion.

## **Reduction in angular distortion due to building stiffness**

Parametric studies using a series of UDEC analyses were conducted to evaluate the reduction in the ratio of angular distortion,  $\beta$ , with respect to the change in greenfield ground slope,  $\Delta GS$ , between adjacent structural units due to the shear stiffness of a masonry bearing wall for both downdrag cases and no downdrag cases.

The ratio is a function of relative building/soil shear stiffness and also the level of distortion and cracking in the structure. Window penetrations of 30% will reduce the effective building shear stiffness.

For a medium to stiff soil, and structure with lime rich mortar (type N), the soil/wall shear stiffness is  $E_s L^2 / GHb = 10$ , where  $E_s$  is soil stiffness,  $L$  is the distorted span of the structure,  $G$  is the effective shear stiffness of the building, including reduced stiffness due to window penetrations,  $H$  is building height, and  $b$  is the width of the wall. For this stiffness, cracking results in  $\beta / \Delta GS = 1$ . The value of  $\beta / \Delta GS$  reduces to less than 0.5 only for elastic deformations or for very small cracks at very low values of  $\Delta GS$ . For a stiffer wall or softer soil ( $E_s L^2 / GHb = 1$ ),  $\beta / \Delta GS = 1$  for larger changes in ground slope ( $\Delta GS \geq 3 \times 10^{-3}$ ) and  $\beta / \Delta GS$  reduces to less than 0.5 for  $\Delta GS \leq 1.5 \times 10^{-3}$  (Son and Cording, 2005, Cording et al, 2008).

## **Era and type of building construction**

### **Brick bearing wall structures with timber floor joists.**

A large number of brick bearing wall structures were constructed in cities and towns throughout the U.S. in the mid to late 1800s and early 1900s. They usually have only one basement level, and the brick walls are on brick or rubble wall footings, although in some cases, structures are on timber piles.

In the early 1800's, many of the brick bearing wall buildings in older towns along the eastern seaboard had both bearing and facade walls tied together with a pattern of alternating headers and stretchers (Flemish bond).

In the late 1800's and early 1900's, the hidden walls (party walls, alley walls and basement walls) of almost all the masonry structures in the U.S. consisted of a common bond consisting of multiple wyths of stretchers tied together, usually every 6 rows, with a row of headers. The building facades were clad with a veneer of a running bond (stretchers, no headers) mortared against the common brick wall behind the facade.

When evaluating buildings adjacent to a proposed excavation, existing damage and deterioration need to be recorded so that they can be separated from any damage that results, or is alleged to result, from the adjacent excavation and, in addition, to determine if the structure's condition will affect its response to excavation-induced ground movements. Over the long term, facade walls with brick cladding tend to be more susceptible to cracking or displacement due to deterioration and environmental effects than the common brick walls. Separation of the brick or stone cladding on the facade from the common wall behind, as well as separation of the facade wall from a perpendicular bearing wall can occur due to deterioration or distortion. Often, the brick cladding, although mortared against the common wall behind, is not well tied to the common wall. Details of window support are important, and their condition is often unrelated to settlement. Wood lintels are subject to shrinkage and can cause cracking and lateral displacement of a wedge of bricks above the windows. Iron or steel angles placed as a lintel to support the brick may be subject to

rust jacking if leakage occurs, which causes the brick above the lintel to be pushed outward.

Brick-bearing wall townhouses built in the 1800's and early 1900's typically have bearing walls with timber joists spanning approximately 6 m between adjacent brick bearing walls. The joists are seated in pockets on the bearing wall, usually one wyth (100 mm) wide. The joists are set in the pocket, they are not usually tied to the wall, unless the wall has been repaired or retro-fitted. One of the critical concerns is rotting of the joist ends due to water and moisture coming through the wall or leaking from drains. 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 readily in a fire, without causing the wall to collapse, an important detail that would prevent progressive collapse of a row of townhouses with common bearing walls during a fire.

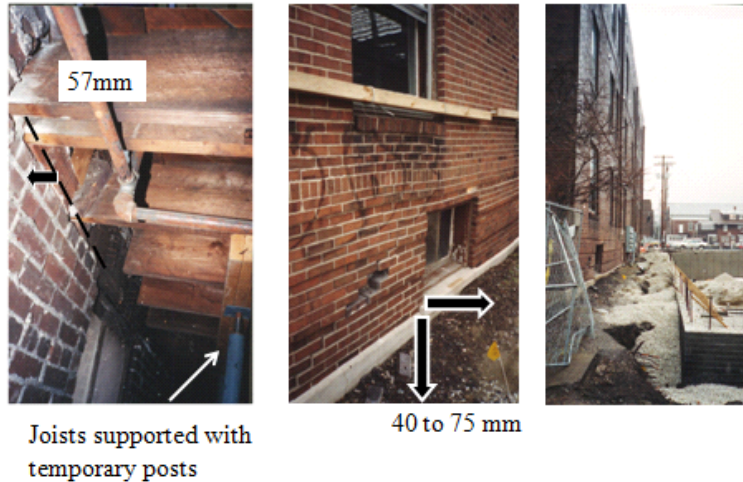
### **Larger masonry bearing wall structures.**

Structures with wider spans may have an intermediate timber bearing wall between the exterior brick bearing walls. In some structures, the upper level floors sag toward the center of the building because of shrinkage of the timbers. This is the case for several of the historic houses in Washington DC, built in the early 1800's. Door frames were observed to have undergone shear distortion, with the side of the door frame toward the center of the building displacing downward due to the sag in the center of the building.

Larger commercial brick bearing wall structures in the 1800's had intermediate supports of timber posts and beams supporting the 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 spaced approximately 1.2 to 1.5 m on center with jacked arches of brick between the beams, set on the lower flange. For the Masonic Temple, the beams were seated in pockets in the brick bearing walls but were not tied so that lateral displacement and settlement of the bearing wall during pit underpinning produced the damage shown in Figure 1a.

### **Bearing wall perpendicular to excavation, lateral displacement reduces floor-joist bearing.**

Figures 7a and 7b illustrate a case where bearing walls perpendicular and adjacent to an unsupported basement excavation were subject to settlement and lateral displacement in the range of 40 to 75 mm. The lateral displacement at the building foundation wall caused loss of 58 mm of joist bearing at the first floor level, reducing the remaining joist bearing to less than 50 mm. The joists on all floors had to be supported with temporary posts and beams and the building was subsequently demolished. The average lateral strain between adjacent bearing walls exceeded  $10 \times 10^{-3}$ , in the very severe damage range (point D1, off the plot in Figure 2).



**Figure 7. Concentrated lateral and vertical displacement of bearing wall**

### **Brick bearing wall structures with concrete floors.**

In the early 1900's, many of the larger brick bearing wall structures were constructed with reinforced concrete floors. In some structures a T beam was formed by rows of clay tile set in the bottom of the form to fill the space between the concrete beams. Some of the floors structures consisted of T beams with a top slab of concrete and the T consisting of an I beam encased in concrete. Concrete floors, unlike timber joists, were tied to the masonry walls preventing lateral extension and loss of bearing of the floors.

One such structure, a 3-story apartment structure built in the early 1900's in Evanston, Illinois, had no grade beams or structural slabs in the basement floor, but upper floors were concrete and tied into the brick bearing walls. Tunneling in the street adjacent to the building resulted in a settlement of 30 mm and a lateral extension in the 11.7 m wide bay of 17 mm that produced cracking of the basement slab and opening of cracks on the wall perpendicular to the tunnel axis. A vertical crack, open 5 mm at the basement level between bays 1 and 2, narrowed and ultimately terminated at the level of the reinforced concrete floor. Angular distortions at foundation level in the first bay were  $2.7 \times 10^{-3}$  and lateral strains were approximately  $2 \times 10^{-3}$  (moderate damage, Point E1 in Figure 2). A diagonal shear crack, open 5 mm, developed in the brick of the exterior wall, immediately adjacent to the bearing wall nearest the tunnel at the ground level, and there was some shear cracking and fall of plaster in a third floor apartment. In the upper floors, the absence of lateral strains reduced the damage level from that observed in the first floor level (slight damage, Point E2 in Figure 2).

The connection of the floor to the wall should be checked. In one case, post-tensioned pre-cast concrete T beams were attached to a concrete block wall with a short rebar that was embedded in the bottom of the beam but not tied to the reinforcement. Lateral extension of 25 mm across the bay of the structure could not be caused a continuous crack to form at the end of the beam, separating the beam from the wall. Temporary posts were placed on all the beams to prevent collapse.

## Extension of cracks and lateral displacement in upper floor levels

Tunneling beneath 7<sup>th</sup> St with an open face shield in sands on the first phase construction of the Washington Metro in the early 1970's resulted in running of the sands into the tunnel face and surface settlements on the order of 70 mm of an adjacent brick bearing wall building (Figure 8). Cracks of 50 mm developed in the basement wall due to lateral ground displacement. Both angular distortion and lateral strain at the foundation of the building are on the order of  $10 \times 10^{-3}$  (point F1, off the chart in Figure 2). The settlement resulted in diagonal shear cracks in the first and second vertical rows of windows located on the bearing wall adjacent to the façade. Shear displacements were concentrated near the façade wall as a result not only of the low shear stiffness of the windows, but also the downdrag of the façade wall on the bearing wall. The building behaves as a box structure, not as a single wall or beam. 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 25 mm near the top of the façade wall (Point F1 in Figure 2). Fall of portions of the cornice and bulging of the masonry finishes developed.

Minor bending cracks developed at the top of the bearing wall, toward the center of the building. Based on the estimated curvatures across the structure, the lateral strain at the top of the bearing wall is  $1.8 \times 10^{-3}$ , (moderate damage, Point F2 in Figure 2). Other buildings along the street were similarly affected by the large shear distortions and lateral strains at the building wall. Facades and cornices required temporary bracing and ties to prevent their collapse.

The effect of downdrag is illustrated by two UDEC numerical analyses of a 4-story brick bearing wall structure (Son, 2004). Figure 9a shows the results for no façade wall downdrag. Maximum settlement was 34 mm and angular distortion in the first bay was  $1.4 \times 10^{-3}$ . Shear cracks were concentrated in the second vertical row of windows from the façade wall. Lateral strain was  $1 \times 10^{-3}$  at foundation level of the first bay and in the upper level of the first bay (Slight damage, point G in Figure 2).

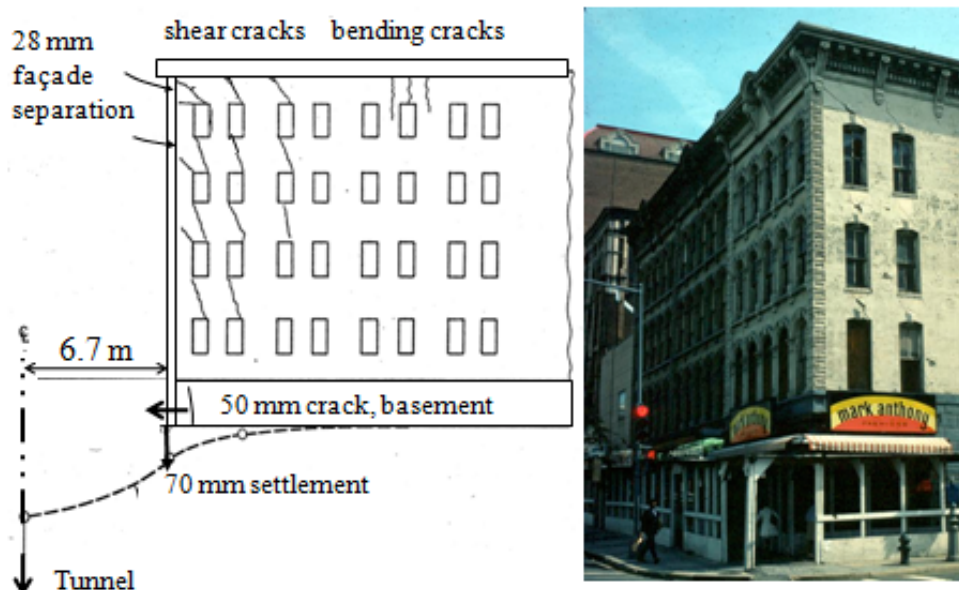


Figure 8: Washington Metro, 7<sup>th</sup> Street, bearing walls perpendicular to tunnel: shear cracks over height of wall caused façade wall to displace laterally.

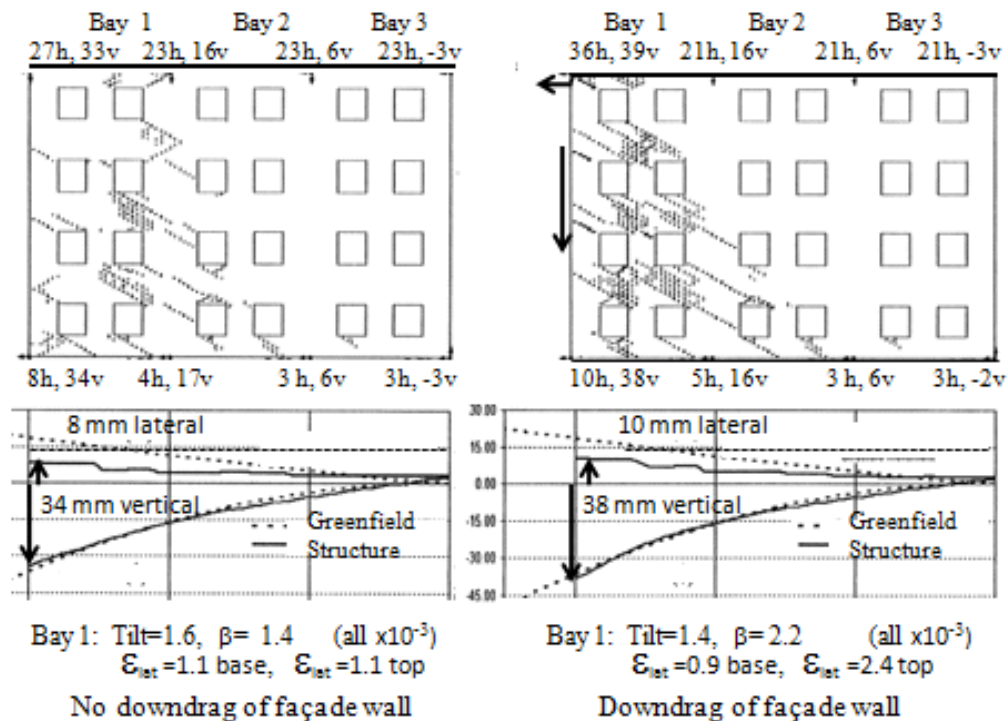


Figure 9: Numerical analysis of brick bearing walls

Figure 9b illustrates the effect of downdrag of the façade wall on the same structure. Downdrag of the façade caused slightly greater settlement at the wall (38 instead of 34 mm) and higher angular distortion in the first bay ( $2.2 \times 10^{-3}$ ). Lateral strain at the foundation level of the first bay was  $0.9 \times 10^{-3}$  (Point H1 in Figure 2) and lateral strains increased to  $2.4 \times 10^{-3}$  in the upper row as shear cracks caused separation of the portion of the wall nearest the façade wall (Point H2 in Figure 2).

Separation and rotation of either a non bearing façade wall or a bearing walls can occur due to settlements caused by excavation or tunneling. Lateral displacements can occur on the upper wall with the propagation of shear fractures or as a result was of pre-existing weaknesses on or between the walls or at the connection between the wall and floors. Structural floor diaphragms tied to walls prevent such cracks. Even for the older brick bearing wall structures, loaded floor joists, roof structures, or cross walls perpendicular to the displacing wall may reduce the opening of cracks due to bending. Investigation of the wall/floor connections and the condition of the building allows the potential strain distributions in the structure to be estimated.

When undergoing settlement, brick walls may rotate outward at the top when the following conditions are present:

1. For façade walls:
  - a. Poor or deteriorated connection of façade wall to bearing wall
2. For bearing walls: floor joists seated in the bearing wall may prevent outward displacement, except when the follow occurs:
  - a. Low floor loads on joists seated on the bearing wall,
  - b. Intermediate walls, such as cross walls or non bearing walls pick up the load of the floor joists as the bearing wall settles causing the joists to become unloaded on the bearing wall.
  - c. Water causes rotting and loss of joist bearing in their seats.
3.
  - a. Floor joists are absent due to open areas, such as stair wells, adjacent to the wall.
4. For both bearing and façade walls:
  - a. Reduced strength of bond between bricks in side walls due to uncontrolled drainage off roof, water damage, deterioration, lack of maintenance, lack of repointing of mortar joints in the side wall.
  - b. Downdrag, angular distortion, and lateral strain result in shear and tension cracks extending over the height of the side wall, usually between window openings close to the end wall.

## CONCLUSIONS

Understanding the characteristics of structures from a given period and in a given region or country --- both how they were built and how they deteriorate and are maintained --- will aid in determining the potential strains and distortions within the structure for assumed greenfield ground displacements. Structural and ground/structure analyses for structural types adjacent to the project site, or for specific structures can follow as needed. In both cases, the damage criterion based on strain and distortion within bays or units of the structure can be utilized. The following outlines steps in the process of evaluating potential distortions and damage.

1. Determine if structure is within the estimated width of the settlement profile.
2. For structures within the anticipated settlement zone, a first level of evaluation is to impose the estimated ground settlement and lateral displacement on the structure.
  - a. The volume of lateral soil displacement is estimated from the wall type, wall stiffness and support installation sequence. For sandy and stiff soils, the volume of surface settlement,  $V_s$  will approach the volume of wall displacement,  $V_L$ , as the wall displacements increase. For soft clays,  $V_s/V_L = 1$ , and will increase with time due to consolidation.

- b. For a parabolic settlement profile, the maximum surface settlement is  $3V_s/L$ , where  $L$  is the length of the settlement profile, and the slope at a distance,  $x$  from the wall is  $2(1-x/L)$ . Lateral displacements can be estimated in the range of 0.5 to 1.0 times the settlement for bulging wall displacements. For a bay close to the excavation that is less than half the length,  $L$ , of the settlement profile, a maximum slope in the range of 1.5 to 2 times the average slope,  $\delta_{\max}/L$ .
  - c. Assuming the building distorts with the ground and spans beyond the settlement zone, estimate building distortions and strains equal to the ground slope and lateral ground strain. The average slope across the settlement profile may be used, but, if bays or building units are significantly narrower than the settlement profile, the slope across the bay should be estimated.
- 3. Distortions will be reduced from those estimated in item 2 if the settlement profile is wider than the structure. Determine the expected changes in ground slope and potential tilt across units or bays of the structure. In selecting the building units that will tend to concentrate distortion or lateral strain, consider geometry and weaknesses in the structure (spacing of columns or bearing walls, presence of construction joints, stair wells, connection of brick cladding to wall).
- 4. Assess condition of the structure: pre-existing distortion and displacements, cracking of masonry and finishes, deteriorated masonry, leakage through walls, rotted floor joists, damaged lintels above windows and doors.
- 5. Consider lateral strains in upper levels of structure
  - a. Due to bending of a beam with displacement pattern imposed over a large  $L/H$  value, and weak in tension
  - b. Due to rotation and opening along shear cracks or pre-existing weaknesses within or between walls. Consider influence of downdrag and amplification of strains with increasing ground displacement.
- 6. Reduce lateral strains due to lateral and bending stiffness of the structure:
  - a. If there is a structural grade beam or reinforced continuous foundation, then lateral ground strains can be ignored. Lateral displacements will concentrate at construction joints, if present.
  - b. In frame structures, or structures with concrete floors tied to the wall, significant lateral strains due to bending or lateral ground strain will not develop in the upper levels of the frame. The detail of the wall/floor connections should be checked.
  - c. For a frame with columns on isolated foundations, lateral ground displacements will be imposed on the lower level columns, and can cause bending and damage to the columns.
  - d. In brick bearing wall structures with joists in seats, lateral strains in upper floors may be low if there is sufficient load on the joists to keep

walls from displacing or if structure has been retrofitted by tying joists to walls.

7. Reduce Angular distortion,  $\beta$ , due to shear stiffness of the structure.
  - a.  $\beta/\Delta GS$  reduces to less than one with reduction in  $E_s L^2/GHb$ .
  - b.  $\beta/\Delta GS$  increases to one with increasing  $\Delta GS$  due to cracking of masonry, which reduces shear stiffness.
  - c. Consider shear stiffness due to strength of mortar, percentage of window penetrations, presence of shear walls and infilled walls, and retrofitting with diagonal braces. (Braces set in second bay to reduce drift during seismic events will not increase stiffness of first bay, rather, shear strains due to settlement may be concentrated in first bay.)
8. Select specific structures or classes of structures for a detailed structural analysis in which strain distributions throughout the structure are determined based on building geometry and stiffness.

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 wall to the building distortion and damage.

Excavation wall stiffnesses and construction practices should be required that limit wall movements and prevent unacceptable building response. Observations and analysis show that ground movements as well as distortions and building damage are non-linear and increase at an increasing rate as the magnitude of wall displacement increases.

To properly assess 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: when and how the building was built, maintained, and repaired. Often the effects of excavation are superposed on pre-existing distortions and deterioration. In many cases, pre-existing conditions are separate from, and unrelated to, the excavation-induced damage. In other cases, the deterioration of the building reduces its strength and stiffness, causing more severe distortion and damage for relatively small displacements.

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