Evaluating Damage Potential in Buildings Affected by Excavations

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Abstract

Predicting building damage due to ground movements caused by excavations is an important design consideration when building in a congested urban environment. Current predictive approaches range from empirical methods to detailed finite element calculations. Limitations inherent in the simpler of these models preclude them from accurately predicting damage in cases where important assumptions are invalid. A new simple model for representing buildings is presented to allow a designer to make realistic simplifications to a building system that is consistent with major features of the structure so that the response to ground movements can be adequately represented. This model assumes that the floors restrain bending deformations and the walls, whether load bearing or in-fill between columns, resist shear deformations. Closed-form equations are presented that relate bending and shear stiffness to normalized deflection ratios. The proposed model is shown to adequately represent the response of a three-story framed structure which was affected by an adjacent deep excavation. The proposed model represents a reasonable compromise between overly simplistic empirical methods and complex, burdensome detailed analyses.

INTRODUCTION

Damage to buildings adjacent to excavations can be a major design consideration when constructing facilities in congested urban areas. As new buildings are constructed, the excavations required for basements affect nearby existing buildings, especially those founded on shallow foundations. Often excavation support system design must prevent any damage to adjacent structures or balance the cost of a stiffer support system with the cost of repairing damage to the affected structures. In either case, it is necessary to predict the level of ground movements that will induce damage to a structure. Practically speaking, a designer is attempting to limit/prevent damage to the architectural details of a building, which occurs prior to structural damage.

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A number of methods found in literature relate building damage to associated ground movements. Several of these methods are based on movements caused by settlements of a structure due to its own weight, and do not consider the deformations that may occur as a result of a nearby excavation. Other methods do attempt to account for the additional modes of deformation, but are limited in their ability to adequately represent the affected structure. The purpose of this paper is to present a laminate beam method of evaluating potential building damage due to excavation-induced ground movements. This method avoids oversimplification common in current empirical methods, yet is not as computationally burdensome as a detailed finite element analysis. Published criteria applicable to excavation-induced building damage are reviewed and compared to the laminate beam method. In addition, assessments of damage potential as determined by both the existing and proposed methods are compared to detailed records of damage to a three-story framed structure.

BACKGROUND

Definition of terms

Figure 1 shows a sketch of a hypothetical settlement profile. The settlement of any point $i$ is denoted as $\rho_i$. Differential settlement between two points, $i$ and $j$, is given the symbol $\delta_{ij}$. The distance between two points $i$ and $j$ is denoted $\ell_{ij}$. Distortion between two points, $i$ and $j$, is defined as $\delta_{ij}/\ell_{ij}$, and is not explicitly shown on the figure. A concave-up deformation is called “sagging,” while a concave-down deformation is called “hogging.” An inflection point, $D$, separates two modes of deformation. The length of a particular mode of deformation, bounded by either the ends of a building or inflection points of the settlement profile, is denoted $L$. The average slope, $m$, of a specific mode of deformation is defined as $\delta_{kl}/L_{kl}$, where the subscripts $k$ and $l$ are boundaries of the mode of deformation. This slope differs from the distortion, $\delta_{ij}/\ell_{ij}$, which is the ratio for two adjacent points. The relative settlement of each mode, $\Delta$, is the maximum deviation from the average slope of a particular deformation mode. The deflection ratio, $\Delta/L$, not explicitly shown, is the ratio of the relative settlement to the length of the deflected part. This term will be used herein as a measure of the ground deformation profile when evaluating damage predictions.
Rigid body rotation of the building, $\omega$, is the tilt of the building and causes no stresses or strains in the building. Angular distortion, $\beta$, is the difference between distortion, $\delta_{ij}/\ell_{ij}$, and rigid body rotation, $\omega$. When multiple modes of deformation occur in a building, i.e., when the slope, $m$, of each portion is not equal to $\omega$, additional shearing strains, $\gamma_{\text{add}}$, arise from the difference between the rigid body rotation and the slope and must be considered when computing strains. This point will be illustrated in more detail later.

The critical tensile strain, $\varepsilon_{\text{crit}}$, the strain at which cracking becomes evident, may vary significantly from one material to another. Tensile strains, $\varepsilon_t$, can be caused by bending, $\varepsilon_b$, diagonal tension due to shear, $\varepsilon_d$, or horizontal extension, $\varepsilon_h$, caused by lateral extension of the building due to lateral movement in the soil mass below the footings. If multiple modes of deformation are superposed, these components can be combined in a Mohr circle of strain to determine the maximum principal tensile strain, $\varepsilon_p$. Critical strains that cause failure in common building materials vary widely as a function of material and mode of deformation, as summarized by Boone (1996).

**Criteria to evaluate excavation-induced damage**

Selected criteria that are applicable to evaluate excavation-induced damage are summarized in Table 1, wherein the relevant parameter and its limiting value are shown. Note that the parameter used to relate structural movements at the foundation level to damage depends on the method. Deep beam methods are more general than empirical methods (e.g., Skempton and McDonald 1956 and Polshin and Tolkar 1957) which were limited to damage of structures based on settlements arising from the weight of the structure.

Burland and Wroth (1975) modeled a building as a deep isotropic beam to relate strains in the building to the imposed deformations. Tensile strain served as the limiting criterion for visible crack development when used with an elastic analysis of the building. They suggested that for sagging type deformations, the neutral axis is located at the middle of the beam. For hogging type deformations, the foundation and soil provide significant restraint to deformations. In the
limit, the bottom of the beam does not deform, in effect moving the neutral axis to its bottom. Equations for limiting $\Delta/L$ were written in terms of maximum bending strain and maximum diagonal tensile strain for a linear elastic beam with a Poisson’s ratio, $\nu$, of 0.3 subjected to a point load with the neutral axis at either the center or bottom of the beam. The effects of a building that is not adequately represented by an isotropic elastic beam are accounted for by varying the ratio of Young’s modulus, $E$, to shear modulus, $G$, for the beam, depending on the type of structure. They postulated that for buildings with significant tensile restraint, or very flexible in shear (i.e. frame buildings), an $E/G$ ratio greater than the theoretical value of 2.6 would be appropriate, and recommended that the value be taken as 12.5. However, for buildings that have little or no tensile restraint (i.e. traditional masonry buildings), they recommended that the $E/G$ ratio should be reduced to 0.5.

Voss (2002) extended the Burland and Wroth (1975) equations to allow explicit input of $E/G$ and location of the neutral axis, resulting in the following equations that relate limiting $\Delta/L$ to bending strain at the top, $\varepsilon_{b(top)}$, and bottom of a beam, $\varepsilon_{b(bottom)}$, and the maximum diagonal tensile strain, $\varepsilon_{d(average)}$:

\[
\frac{\Delta}{L} = \left( \frac{1}{12(1-\lambda)} \right) \frac{L}{H} + \alpha \left( \frac{1}{12} + \left( \frac{1}{2} - \lambda \right)^2 \right) \frac{H}{L} \frac{E}{G} \varepsilon_{b(top)}
\]

(1)

\[
\frac{\Delta}{L} = \left( \frac{1}{12\lambda} \right) \frac{L}{H} + \alpha \left( \frac{1}{12} + \left( \frac{1}{2} - \lambda \right)^2 \right) \frac{H}{L} \frac{E}{G} \varepsilon_{b(bottom)}
\]

(2)

\[
\frac{\Delta}{L} = \left( \frac{1}{18} \right) \frac{L^2}{H^2} + \frac{\alpha}{12} \left( \frac{1}{12} + \left( \frac{1}{2} - \lambda \right)^2 \right) \frac{E}{G} \varepsilon_{d(average)}
\]

(3)

where $H$ and $L$ are the height and length of a beam, respectively, $\lambda$ is the ratio of the distance from the neutral axis to the bottom on the beam to its height, and $\alpha$ is the ratio of the of maximum shear stress to average shear stress – equal to 3/2 for a rectangular beam.

Based on eqs. (1)-(3), Figure 2 shows the effects of different $E/G$ ratios and neutral axis
locations on the conditions required for initial cracking. In this figure, the kink in a curve represents the limit between shear critical and bending critical geometries of a beam. With the exception of flexible structures (E/G ≥ 12.5) and structures with small L/H ratios, bending strains are more critical. These results also show that the limiting deflection ratio that causes cracks varies over wide limits, implying that structural details must be considered when establishing criteria. However, it is difficult to provide guidance on the selection of the beam characteristic parameter E/G and the neutral axis location, especially when developing a deep beam model for a multi-story structure.

Boscardin and Cording (1989) extended this deep beam model to include horizontal extension strains, $\varepsilon_h$, caused by lateral ground movements induced by adjacent excavation and tunneling activities. A chart relating $\beta$ and $\varepsilon_h$ to levels of damage was developed for buildings with brick, load-bearing walls and an L/H ratio of 1 undergoing a hogging deformation with the neutral axis at the bottom. Similar to Burland and Wroth (1975), the building is idealized as a linear elastic beam with $\nu$ equal to 0.3. Direct transfer of horizontal ground strain to the structure is assumed in this approach. However, when the ground displaces laterally, relative slip will occur at the foundation level, and the horizontal displacement in the building will be less than that in the ground (Geddes 1977, 1991). Thus, this approach represents an upper bound of the effects of horizontal ground strain. Many modern buildings are well-reinforced laterally by stiff floor systems, which essentially eliminate lateral movements at the foundation level in presence of lateral ground movements (e.g., Finno and Bryson 2002).

Boone (1996) presented a more detailed approach to evaluate building damage due to differential ground movement caused by adjacent construction. This method considers structure geometry and design, strain superposition and critical strains of building materials. Load bearing walls are modeled as uniformly-loaded, simple-supported beams. Damage to frame buildings is assumed to occur from differential vertical movements of columns, depending on the column’s tilt and degree of fixity. Damage to infill walls is presumed to occur as a result of the deformed shape of the surrounding beams and columns. If a structure is subjected to horizontal extension, then these strains are superposed on the ones caused by bending and shear.
Note that proper application of any of these methods requires an accurate prediction of the magnitude and distribution of ground movements adjacent to an excavation. Empirical envelopes that bound observed settlements at sites as a function of soil conditions have been presented by Peck (1969) and Clough and O’Rourke (1991). Hsieh and Ou (1998) developed a semi-empirical method to predict the distribution of ground movements in a direction perpendicular to an excavation wall. While finite element methods also can be used to compute the desired settlements, conventional plane strain analyses with commonly-used constitutive models generally underpredict the settlements close to an excavation and overpredict them at larger distances (e.g., Finno and Harahap 1991). This results in an unconservative prediction of the distortions that will impact an affected building. Computation of the proper distribution of ground surface settlements is only possible if a constitutive model that accounts for strain-dependent (from very small to operational strains) modulus is incorporated in the analysis. Furthermore, while these approaches consider distress perpendicular to a supported excavation, significant distortions may develop parallel to an excavation (e.g. Finno and Bryson 2002; Finno and Roboski 2005), and these should be considered as well. One way to estimate these for excavations in clay is given by Finno and Roboski (2005). Proper evaluation of the expected deformations is an important step in any procedure used to estimate damage, but is beyond the scope of this paper.

**PROPOSED METHOD**

Following the approach of Burland and Wroth (1975), criteria related to visible cracking will be developed herein. They modeled a building as a rectangular beam with unit thickness, implicitly assuming a constant value of $I/A_v$ for the building. However, in a beam, deformation due to bending is proportional to the bending stiffness, $EI$, where $I$ is the moment of inertia of the beam, whereas deformation due to shear is proportional to the shear modulus times the area contributing to shear resistance, $GA_v$. It is proposed to use $EI/GA_v$ as the parameter to account for variations in bending and shear stiffness of a structure.

Modern buildings are often designed with floor and roof diaphragms to efficiently distribute shear due to lateral loading. These diaphragms are concrete slabs or other types of floor systems
that are fairly inextensible in tension, and are often considered rigid for design purposes. The shear from lateral loads passes into the diaphragms, which transfer this shear to walls (usually parallel to the direction of loading) in different magnitudes depending on their in-plane stiffness. Even in buildings that are not specifically designed with floor and roof diaphragms, the large area of the floors provides a significant degree of restraint to in-plane deformations and thus to bending deformations. This is true for both framed structures and load-bearing wall structures provided there is some mode of shear transfer from the roof and floors to the walls.

To account for these observations, a laminate beam is proposed to model the response of a building to imposed deformations. It is assumed herein that the floors offer restraint to bending deformations, and the walls, whether load bearing or infill between columns, offer restraint to shear deformations.

**Parameters required**

As shown in Figure 3, this model can be developed by combining layers of “plates” separated by a layer of “core material” to model a building with \( n \) stories. Each of these plates can have different thickness and width, and the distance between plates may vary depending on floor-to-floor height. The core materials may also have different properties. Figure 3 also shows a cross section of this model. The thicknesses of the floors are assumed to be negligible compared to the height of the building, \( H \), implying that the stress in each floor slab is approximately uniform. To determine the properties of a laminate beam that represents a multi-story building, the distance from the neutral axis location to the bottom of the beam, \( \lambda H \), is found as:

\[
\lambda H = \frac{\sum_{i=0}^{n} A_i h_i}{\sum_{i=0}^{n} A_i}, \quad (4)
\]

where \( i \) is the floor number (basement slab is zero), \( A_i \) is the area of the floor slab contributing to bending resistance, and \( h_i \) is the location of the floor measured from the bottom of the laminate beam. The moment of inertia of the beam, \( I \), is calculated as:

\[
I = \sum_{i=0}^{n} A_i (h_i - \lambda H)^2. \quad (5)
\]
There is no allowance made for the moment of inertia of each floor slab about its centroidal axis because the thickness of the floor is very small in comparison with the overall height of a structure. The floor system may be the combination of separate materials, e.g., a concrete floor slab with steel reinforcement, wherein a transformed area must be determined so that a single value of Young’s modulus can be used.

The distribution of shear force defined by the ratio of shear in each story, \( V_i \), to the total shear in the laminate beam, \( V \), is:

\[
\frac{V_i}{V} = \frac{y_i}{I} \sum_{j=i}^{n} A_j (\lambda H - h_j).
\]

(6)

Two subscripts are used to show that this shear force ratio in story \( i \) is a function of the height \( y \) of the \( i^{th} \) story divided by the moment of inertia of the entire beam times the sum of the floor area of times the distance to the neutral axis of each floor above the \( i^{th} \) floor level.

If the infill walls are compliant, or are not intended to act as shear walls, the stiffness of the columns may significantly affect the shear stiffness of each floor level if they have moment resisting connections at each end. It is difficult to account for this stiffness exactly, but it is assumed herein that columns are fixed top and bottom, which will overestimate the stiffening effect of the columns slightly. The stiffness of each column, written in terms of the properties of the column and the height of the story in the form of equivalent shear stiffness is

\[
(GA_y)_{\text{column}} = \frac{12 E_c I_c}{y_i^2}
\]

(7)

where \( E_c \) is the Young’s modulus of the column, \( I_c \) is the moment of inertia of the column in the plane of wall, and \( y_i \) is the story height. In reality, since the ends of the columns will experience some small amount of rotation, the factor of 12 may be reduced as much as 25%. The stiffness of each column in the section that is being analyzed should be summed and added to the wall stiffness of each floor level. The total shear stiffness of each floor, \((GA_y)_i\), is then the sum of the column stiffnesses and the in-plane stiffness of the infill walls.
\[(G A_v)_i = \sum \left[ (G A_v)_{\text{column}} + (G A_v)_{\text{wall}} (1 - a) \right] \]  

where \( a \) is the percentage opening in a wall. The additional stiffness of the columns will not have much of an effect in the case of masonry infill. However, for drywall infill or wall lines with many openings the effect can be significant.

An equivalent shear stiffness, \( G A_v \), of the beam representing the entire structure is determined by:

\[
\overline{G A_v} = \frac{1}{\sum_{i=1}^{n} \gamma_i \frac{V_i}{H V (G A_v)_i}} . \tag{9}
\]

Eq. (9) is derived from the shear strain of each floor, \( \gamma_i \), and its relation to the total shear strain of the cross section, \( \gamma \), such that:

\[
\sum_{i=1}^{n} \gamma_i \gamma_i = \gamma H = \frac{V H}{G A_v} \tag{10}
\]

where

\[
\gamma_i = \frac{V_i}{(G A_v)_i} . \tag{11}
\]

In some cases when large deformations occur, the connections of the infill to the frame are not sufficient to provide full composite shear action of the infill panel. In these cases, using the shear modulus of the material multiplied by the equivalent area will overestimate the stiffness of the wall, and thus that of the building. Although there are detailed methods in literature to determine the rigidity of shear walls with openings, e.g., Drysdale et al. (1999), smeared shear stiffness is considered adequate since the mechanisms being modeled through the laminate beam will give an average shear strain that remains constant along the length of the building. Guidance for selection of material properties for infill wall materials can be found in Boone (1996) and ACI (1999, 2002).
Equations to determine critical strain as a function of $\Delta/L$

Voss (2003) used a complimentary virtual work approach to determine the strain-deflection relationships of a laminate beam to relate tensile strains to deflection ratio, $\Delta/L$. This approach can be used to set limits on deformation to avoid cracking when an appropriate critical strain value is used.

**Simply supported beam**

Making the same assumption as Burland and Wroth (1975) concerning insignificant differences between the concentrated and uniform loads, Voss (2003) derived equations for deflection ratio in terms of the maximum bending strain $\varepsilon_{b}$, at both the top and bottom of the sandwich beam, and the diagonal tension strain in floor $i$, $\varepsilon_{d_{i}}$:

\[
\frac{\Delta}{L} = \left( \frac{L}{12(1-\lambda)H} + \frac{EI}{G_{A}L(1-\lambda)H} \right) \varepsilon_{b\text{(top)}} 
\]  \hspace{0.5cm} (12)

\[
\frac{\Delta}{L} = \left( \frac{L}{12\lambda H} + \frac{EI}{G_{A}L\lambda H} \right) \varepsilon_{b\text{(bottom)}} 
\]  \hspace{0.5cm} (13)

\[
\frac{\Delta}{L} = \left( \frac{L^{2}(G_{A_{i}})}{24EI \frac{V_i}{V}} + \frac{(G_{A_{i}})}{2 \frac{V_i}{V} G_{A_{y}}} \right) \gamma_{i} 
\]  \hspace{0.5cm} (14)

In eq (14), $\gamma_{i}$ is the engineering shear strain. Required parameters and the appropriate critical strain, $\varepsilon_{\text{crit}}$, are substituted into eqs. (12) through (14) to determine the deflection ratio required to cause cracking. Note that a different value of $\Delta/L$ will be obtained for diagonal tension in eq (14) in each story because the magnitude of shear strain varies through the cross section. The minimum value of $\Delta/L$, from eqs 12 through 14, provides a critical deflection ratio after which cracking is predicted. This method also predicts cracking potential when provided with a
deflection ratio due to ground settlements.

**Deviations from simply supported beam assumption**

In cases where the building settles under its own weight, the assumption of a simply supported beam is usually adequate. However, when excavation-induced settlements are considered, different shapes of the settlement profile may develop, as illustrated in Figure 4. In cases where the building is not very long, a simply supported beam is an adequate assumption (Figure 4a). There will be a single mode of deformation in the building, the rigid body rotation will be equal to the slope, and with no moment applied at the ends of the building, simple supports can be assumed.

Alternatively, if the excavation-induced movements impinge on a building such that a building undergoes both sagging and hogging modes of deformation (Figure 4b), the simply supported beam assumptions are invalid because such a model requires that the rigid body rotation be equal to the slope of each simply supported beam (i.e. \( \omega = m \) for each deformation mode). For this to be true, there must be a discontinuity in the beam at the inflection point so that each portion acts independently as a simply supported beam. This generally is not the case, and the discrepancy results in additional shearing strains, \( \gamma_{\text{add}} \), equal to the slope minus the rigid body rotation (\( \gamma_{\text{add}} = m - \omega \)). The sign of \( \gamma_{\text{add}} \) is important when considering where the additional shearing strain is added to the shear strain due to the simply supported beam assumption. The absolute value of \( \gamma_{\text{add}} \) is employed to find the maximum shear strain in the beam when assessing the possibility of cracks.

The maximum shear strain from the simply supported beam assumption is added to the additional shearing strain caused by differences between rigid body rotation and slope. To account for this superposition of shear strains, equation (14) is modified such that the sum of \( \gamma_i \) and \( \gamma_{\text{add}} \) is related to the critical shear strain \( \gamma_{\text{crit}} \). To define \( \Delta/L \) where first cracking occurs, the value of \( \gamma_{\text{crit}} \) is reduced by \( \gamma_{\text{add}} \), and equation (14) becomes:
The inclusion of this additional shear strain is a major difference between the proposed method and existing deep beam approaches.

In a case where the deformation affects only a small portion of a building, the building may act like a cantilever beam, especially if it has a diaphragm system. In this case, eqs. (12) through (15) are not sufficient to describe the actions of the building. A cantilever load condition must be used in the complimentary virtual work equations to determine the critical deflections to cause cracking.

Note that when the critical bending strain is exceeded, cracks may not occur in the floor or roof slabs since it was assumed that all bending is resisted by the roof and floor systems. Rather, this cracking may be manifested as vertical cracks at junctions with columns or out-of-plane walls because as the floors are extended, the walls will extend with them. Additional resistance provided by the walls at this stage will be negligible since it is assumed that the floors are much stiffer in the lateral direction than the walls. Also, the critical tensile strain to cause cracking in the floors usually is greater than that for infill walls. Furthermore, the connection of infill walls to the framing has little resistance to tension. When the bending strain reaches the critical value of this connection, gaps will develop between the walls and framing members causing cracks at these joints. Thus, critical tensile strains in eqs. (12) and (13) can be used to evaluate cracking in the floors and separation at the junction of walls and columns if the appropriate critical strain is used. Note that vertical cracks at columns can also be caused by shear deformations, and thus should not be assumed to be caused solely by bending deformations.

Summary

The following procedure to evaluate damage potential in buildings affected by ground
movements resulting from deep excavations, applicable to horizontally stiff or other structures wherein lateral ground movements induced by excavations do not induce lateral movements in the building, can be summarized as:

1. Predict a distribution of ground movements.

2. Locate the affected structure in relation to the expected ground movements. If needed, divide the settlement profile at each wall line to be analyzed into sagging and hogging zones at inflection, tangent and end of structure points.

3. Compute the slope \( m = \frac{\delta_{ij}}{L_{ij}} \) for each mode of deformation.

4. Estimate the rigid body rotation \( \omega \) of the section being analyzed. For many practical cases, one can use the slope of the entire building from one extreme end to the other.

5. Compute the additional shearing strain in each mode of deformation by using \( \gamma_{\text{add}} = m - \omega \).

6. Define the geometry and structural properties of the pertinent section of the building. See the Appendix for a summary of the required parameters.

7. Choose appropriate critical strain values for a given material and assumed failure mode from Boone (1999), ACI (1999, 2002) or other methods.

8. Use the bending stiffness and shear stiffness obtained by the laminate beam method and compute the value of the limiting \( \Delta/L \) to cause cracking for various failure modes from eqs. (12) through (15). If the deflection ratios from the predicted settlements are greater than the limiting deflection ratios, then cracking is likely.

This procedure can be used in the design stage of a supported excavation when the stiffness of the support system is selected to limit the expected ground movements to levels that will either prevent or minimize damage to an adjacent structure.
To illustrate the utility of this procedure, the laminate beam method is applied to the damage observed at the Frances Xavier Warde School in downtown Chicago, IL, as a result of excavation for the renovations to the Chicago-State Subway station. The performances of the excavation support system and the school have been described by Finno et al. (2002) and Finno and Bryson (2003). A plan view of the structure and excavation is shown on Figure 5. The project included excavation along State St. of 12.2 m of soft to medium clay within 2 m of the school, which is supported on shallow foundations. A shallower excavation was made along Chicago Ave. to provide access to the station. The main support system consisted of a secant pile wall supported by one level of cross-lot braces and two levels of tie-back ground anchors. Inclinometers and optical survey points in and on the building were used to monitor ground and structural movements that developed during construction. Because these movements can be correlated with the time when the cracks were first observed, it is possible to evaluate methods of assessing damage due to settlements.

Potential crack assessments of the wall (A-A’) and frame (B-B’) sections shown on Figure 5 were made using the deep beam and proposed methods. The wall section is an exterior section that was subjected to distortions arising parallel to the excavation, which are not always appreciated as being potentially damaging. The frame section is an interior section oriented perpendicular to the excavation in the area where the largest movements occurred.

Figure 6 shows details of the wall section, the laminate beam representation of the north and south segments of the wall, the location of the observed damage and the settlement profiles for construction days 74, 137 and 312. Day 74 was the day after cracking first was observed in the school. Day 137 represents a day soon after the excavation reached final grade. Day 312 was the final survey for the project when all movement and cracking had occurred (Finno and Bryson 2002).

The north and south segments of the wall are constructed of a limestone masonry veneer with a concrete masonry backing. The center segment of the wall contains many windows and the
main entrance doorway to the school. As can be seen, the north and south segments of this wall hog, the center segment sags, and as construction progressed, the building tilted downward to the north. The rigid body rotation at each construction day for this wall is computed as the angle from the horizontal of the line drawn from the end points of the entire wall, as sketched on the figure for day 137.

Figure 7 shows the details of the frame section, the laminate beam representation of the wall, the location of the observed damage, locations of the optical survey points and the settlement profiles for construction days 74, 137 and 312. The section consists of a reinforced concrete frame with primarily masonry walls with plaster finishes. The frame portion of the wall generally experiences sagging between points A and B and hogging east of point B. The inflection points used in the analysis are those calculated from settlement contours (Finno and Bryson 2002). The rigid body rotation for the frame section is the angle defined by the end points of the entire section.

Table 2 summarizes the observed damage at the wall and frame sections in relation to the construction activity at the site. The damage to the school consisted mainly of hairline cracks 300 to 500 mm long that occurred in non-load bearing walls. Cracks were noted within the frame section well before they were observed in the wall section parallel to the excavation. Based on the post-construction survey of the entire building, 29, 37 and 47 cracks were observed in locations on the first, second and third floor, respectively. On the first floor, damage was concentrated in the sagging zone, particularly on east-west walls. In the parts of the building that underwent a hogging mode of deformation, more cracks were found on the third floor. Only a few cracks had widths greater than 6 mm. The damage was classified as slight, according to the Burland et al. (1977) damage classification. A more complete description of the damage is found in Bryson (2002).

**Idealization of sections**

Criteria from the Burland and Wroth (1975) approach and the laminate beam approach are applied to the data collected from the Frances Xavier Warde School. The horizontal movements
of the school itself were measured to be zero, so $\varepsilon_h = 0$, and the Cording and Boscordin method reduces to the Burland and Wroth approach. Thus only the latter is considered and called the deep beam method hereafter.

Both deep and laminate beam methods are elastic. Properties of the concrete were based on an unconfined compressive strength, $f'_c$, of 27.6 MPa such that $E = 24.8$ GPa, $G = 10.3$ Gpa and $\nu = 0.2$. Properties of masonry are selected from the ACI 530 recommendations for the average value of unconfined strength, $f'_m$, of 13.8 MPa such that $E = 12.4$ GPa, $G = 5.0$ GPa and $\nu = 0.24$.

The parameters used as input to the deep beam analysis are given in Table 3, including assumptions concerning the neutral axis location and the $E/G$ ratio. For the north and south sections of the wall, the neutral axis is taken at grade and $E/G$ is assumed equal to 0.5 for a solid masonry wall which is assumed to be very stiff in shear. The center section is more flexible than the end sections due to the presence of the windows. In this case, $E/G$ was somewhat arbitrarily selected as 2.6. The east west section is a frame with many openings, so this section is assumed flexible in shear and $E/G$ is taken as 12.5. In the sagging zone, the neutral axis is assumed to remain at the center of the building. In the hogging zone, the neutral axis is taken at the foundation level.

The laminate beam representation of the north and south segments of the wall section is shown in Figure 6. These walls do not contain any openings. The joists in this section run parallel to the wall, so the area of the closest of these joists are added to the area of the floor slab when deriving the bending stiffness of these floors. The center segment is similar to the north and south segments, except for the openings located within the masonry wall at this location; the percentage of openings in this center section was 70% for the first floor and 50% for the second and third floors.

The laminate beam representation of the east-west section is shown in Figure 7. This section is a reinforced concrete frame with non load-bearing masonry infill walls. The presence of an exterior concrete basement wall gives the section different properties between the western
(between points A and D) and the eastern (between points D and E) portions of the section. However, the settlement east of point D shows very little curvature such that only shear distortions need to be considered in the exterior masonry portion of the section. Note that no damage was observed in this section of the frame, as expected for such small movements. When computing the deflection ratios for the frame, sagging occurs between point A and the inflection point, hogging occurs between the inflection point and the end of the curvature – essentially at point D. Thus the portion of the wall east of point D was not considered when computing the structural properties of the frame section.

For the flange width of the floor slabs to be included in the analysis, the ACI 318 recommendation of eight times the slab thickness was used. This length contained two joists, or ribs, on either side so their area was included in the area of the floor slab. The shear stiffness used for the walls were calculated using the smeared stiffness as a function of the percent openings in the frame between points A and D, as indicated on Figure 7.

The critical shear strain that is assumed for the masonry in this analysis is 0.11%. This is at the lower end of the range given by Boone (1996) for clay brick with cement-lime mortar and about the value for cement-line mortared concrete blocks. The critical shear strain chosen for the concrete basement walls is 0.15%.

The parameters used in eqs. (12) through (15) are shown in Table 3. The E/G ratios, recommended by Burland and Wroth (1975) can be compared to the ratio of bending-to-shear stiffness used herein. The E/G ratios for the segments vary by a factor of 25 whereas the corresponding bending-to-shear stiffness ratios only vary by 4.5. The differences in the bending-to-shear ratios for the segments arise primarily from differences in the shear stiffness of the walls. The E/G value for the center segment of wall A-A' should be higher, i.e., more flexible, than the 0.5 value used for the north and south segments due to its large percentage of openings. Burland and Wroth (1975) provide no means to quantify these effects, other to provide recommendations for stiff or flexible buildings. However, the bending-to-shear stiffness ratio given in Table 3 for this section is 2.7, greater than that of the solid north and south segments. Perhaps an appropriate value of E/G for the center segment is 0.5(2.7) = 1.4. In any
case, a value of 2.6 was used in this calculation. This example highlights the difficulties in characterizing a structure as a deep beam solely by its E/G ratio.

Results of crack assessments

To illustrate the utility of the laminate beam approach, the results of crack assessments along the wall and frame sections are presented herein. Rather than focus on the maximum values, the $\Delta/L$ values are presented as they developed during the excavation, so as to more fully evaluate the proposed method.

Wall Section A-A'

Figures 8, 9 and 10 summarize the results of laminate beam analyses of wall section A-A'. Two plots are shown for each segment, ratios of additional shear strain to critical shear strain vs. time, and $\Delta/L$ versus time. The former plot is presented to show the effect of $\gamma_{add}$ on the deflection ratio required to cause cracking. As the ratio increases, the required deflection ratio to cause cracking decreases (eq. 15). The latter figure shows the $\Delta/L$ based on field observations, the $\Delta/L$ required to cause cracking per the deep beam method, and the $\Delta/L$ to cause cracking at each level of the building based on the laminate beam method. Eqs. (12) and (13) for tension due to bending yielded high critical deflection ratios compared to those computed by eq. (15) for tension due to shear. Thus only the latter are shown in the figures. Note that in the equation for shear strain critical, the subscript i appears, implying that there will be a different deflection ratio to cause cracking for each floor because the shear strain differs in each of these floors (eq. 15). This also allows for different critical shear strain values to be employed for floors with dissimilar materials.

It is evident in Figures 8 and 9 that the additional shearing strain has a large effect on the value of the deflection ratio that will cause cracking, and is what causes the limiting values of $\Delta/L$ to decrease with time for the laminate beam method. When the “diagonal shear critical” lines cross the measured value of deflection ratio in the figures, shear cracks are predicted by the laminate beam method. Figures 8 and 9 show that this method indicates cracking of the first and
second floor walls at day 150 for both the north and the south segments. Cracking was first noticed in the exterior walls of both segments near the top of the first floor and the bottom of the second floor on day 127, in reasonable agreement with the model assessment. Both the cracks were step cracks in the mortar oriented at about 45°, consistent with expected patterns for shear cracking. Cracking also is indicated in the third floor wall of the north segment at about day 190. While no cracks were observed in the upper elevation of the exterior brick walls, the exterior masonry at that level was difficult to observe. Also, results of the deep beam method indicates that cracking would occur at Δ/L values 2 to 5 times greater than that which occurred at the south and north segments, respectively, suggesting that no cracking would be expected based on this approach.

Figure 10 shows that both the laminate and deep beam and methods suggest the center portion of the west wall would not likely crack, except perhaps within the first floor. In fact, no cracks were observed in this exterior wall. The main difference in the responses of the three segments of the west wall was the added shear strain for the center portion was about 1/10 that of the two end segments.

Frame section B-B’

Figures 11 and 12 show the results of the laminate and deep beam analyses for the sagging and hogging portions of frame section B-B’, respectively. These plots show the deflection ratio vs. time and also the γ_{add}/γ_{crit} vs. time to illustrate the effects of both. As shown in Figure 11, the ratio of additional shear strain to critical shear strain becomes greater than 1.0 between days 86 and 103, causing the Δ/L to cause cracking to go to zero at this time. This behavior suggests cracking regardless of the deflection ratio. Cracking is predicted for all floors between days 86 and 103 for the sagging portion of this wall. Cracks first were observed on all floors at this location on day 73. The Δ/L required for cracking per the deep beam analysis was about 5 times greater than the maximum observed value - again suggesting that this method did not adequately represent the responses of the building.

Figure 12 shows the results for the hogging portion of the frame section. No wall existed on
the first floor in this hogging zone, hence no calculation is made for the first floor. For this portion of the wall, the ratio of additional shear strain to critical shear strain is much smaller than in the sagging section. Comparison of the critical and measured deflection ratios vs. time show that cracking is expected on the second floor at about day 175. Several hairline cracks did occur at the second floor, but not along the section analyzed herein. No cracking was observed at this location on the third floor, in agreement with the results of the proposed method. The results of the deep beam method suggested that no cracking would be expected at any level in this section.

**Summary**

The laminate model was reasonably successful in assessing cracking potential in both the wall and the frame sections. In contrast, the deep beam analysis did not “predict” cracking as a result of the settlements that occurred at the building. This is due to both oversimplification of the structural details and neglect of the additional shear strains arising from the differences between the slope of the deformation profile and the rigid body rotation. While the laminate beam model includes a number of simplifications, it represents a building with stiff floor systems more faithfully than the deep beam approach.

The model has not been developed for the consideration of residual strains that develop when the building settles under its own weight; however, the extension would not be extremely cumbersome. One could estimate the residual strains and superpose them to those computed with this model in much the same way as the additional shear strains caused by multiple modes of deformation are included. However, defining how much settlement would have occurred prior to attaching in-fill walls to a structural frame during the original building construction would be a difficult task. Certainly, the movements to which the architectural portions of the structure would have been subjected were less than the total settlements. Furthermore, the authors are unaware of any performance data that includes both self-weight and excavation-induced movements, so the former effects were not considered. In any case, this aspect of response warrants further study.
CONCLUSIONS

Based on the analyses presented herein, the following conclusions can be drawn:

1. Deep beam methods of evaluating damage potential are not adequate to describe the full range of responses of a variety of buildings. Specifically, the methods evaluated do not adequately account for the structural properties of the Frances Xavier Warde School, or account for the deformation pattern caused by the excavation-induced ground movements at this building.

2. The laminate beam model, considering its simplicity, provided an adequate assessment of the cracks observed in the Frances Xavier Warde School as a result of the excavation-induced ground movements. The model represents a more detailed approach than the deep beam methods, without sacrificing much of the simplicity.

ACKNOWLEDGEMENTS

The authors thank Dr. Andy Longinow and Mr. Nicholas Hyatt of Wiss, Janney Elstner Associates Inc. (WJE) for providing the structural details and damage records of the Warde School and Dr. Howard Hill of WJE for reviewing the manuscript. Financial support for this work was provided by the Infrastructure Technology Institute (ITI) of Northwestern University and National Science Foundation grant CMS-0219123. The support of Mr. David Schulz, ITI’s director, and Dr. Richard Fragaszy, program director at NSF, is greatly appreciated.
REFERENCES


APPENDIX. List of parameters for laminate beam analysis

Input to laminate beam analysis

\( A_{vi} \) = shear area of wall \( i \)
\( A_i \) = area of floor slab contributing to bending resistance
\( a \) = percentage of open area in wall
\( H \) = building height
\( h_i \) = distance from the bottom of the building to floor \( i \)
\( y_i \) = height of story \( i \)
\( E \) = Young’s modulus of building component
\( G \) = shear modulus of building component

Computed quantities (equation number in text)

\( \lambda \) = distance from the bottom of the building to its neutral axis divided by \( H \) (4)
\( I \) = moment of inertia of laminate beam model (5)
\( \frac{V_i}{V} \) = percentage shear in story of laminate beam model (6)
\( \overline{GA_y} \) = equivalent shear stiffness of laminate beam model (9)
\( \gamma_i \) = shear strain in story \( i \) of laminate beam model (11)
\( \frac{\Delta}{L} \) = deflection ratio to cause cracking (12) through (15) if appropriate critical strain

is substituted in the equations
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Figure 12. Results of crack assessments: frame section B-B', hogging portion
Table 1. Selected Damage Criteria

<table>
<thead>
<tr>
<th>Type of method</th>
<th>Limiting parameter</th>
<th>Limiting Value</th>
<th>Applicability</th>
<th>Reference</th>
</tr>
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<tbody>
<tr>
<td>Empirical</td>
<td>β</td>
<td>1/150</td>
<td>Structural damage</td>
<td>Skempton and MacDonald (1956)</td>
</tr>
<tr>
<td></td>
<td>β</td>
<td>1/300</td>
<td>Cracking in walls and partitions</td>
<td></td>
</tr>
<tr>
<td>Empirical</td>
<td>δ/l</td>
<td>1/500</td>
<td>Steel and reinforced concrete frames</td>
<td>Polshin and Tokar (1957)</td>
</tr>
<tr>
<td>Deep beam model of building</td>
<td>Δ/(Lεcrit)</td>
<td>f(L/H, E/G, neutral axis location)</td>
<td>Load bearing wall (E/G = 2.6), frame structures (E/G = 12.5), and masonry building (E/G = 0.5) assuming no lateral strain</td>
<td>Burland and Wroth (1975)</td>
</tr>
<tr>
<td>Extended deep beam model</td>
<td>β, εh</td>
<td>Chart</td>
<td>L/H = 1 and assumption horizontal ground and building strains are equal</td>
<td>Boscardin and Cording (1989)</td>
</tr>
<tr>
<td>Detailed analysis of structure</td>
<td>crack width</td>
<td>εp, εt</td>
<td>general procedure that considers bending and shear stiffness of building sections, distribution of ground movements, slip between foundation and grade and building configuration</td>
<td>Boone (1996)</td>
</tr>
</tbody>
</table>
Table 2. Summary of construction activity and damage at the wall and frame sections

<table>
<thead>
<tr>
<th>Day</th>
<th>Construction activity</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-11</td>
<td>Secant pile wall installation</td>
<td></td>
</tr>
<tr>
<td>60-74</td>
<td>Install cross-lot braces</td>
<td></td>
</tr>
<tr>
<td>73</td>
<td>Excavate below first tieback level</td>
<td>Interior cracks; hairline cracks in infill walls on all three floors; second floor door replaned (first damage in frame section)</td>
</tr>
<tr>
<td>79</td>
<td>Install first level tiebacks</td>
<td>Cracks in first floor wall panels (frame section)</td>
</tr>
<tr>
<td>98</td>
<td>Install second level tiebacks</td>
<td></td>
</tr>
<tr>
<td>99</td>
<td></td>
<td>New cracks in first floor wall panels; existing cracks extend and widen (frame section)</td>
</tr>
<tr>
<td>108</td>
<td></td>
<td>Cracks in marble façade of entranceway foyer; existing cracks widen and extend (frame section)</td>
</tr>
<tr>
<td>116</td>
<td>Excavate to final grade</td>
<td>Existing cracks extend and widen on all levels (frame section)</td>
</tr>
<tr>
<td>127</td>
<td></td>
<td>Step cracks observed through mortar along the south and north segments of the west exterior wall of school (first damage in wall section)</td>
</tr>
<tr>
<td>205</td>
<td></td>
<td>New crack observed in marble façade in entranceway foyer (frame section)</td>
</tr>
<tr>
<td>220-310</td>
<td>Place backfill and remove cross-lot braces</td>
<td></td>
</tr>
<tr>
<td>Structure segment</td>
<td>Deep beam</td>
<td>Laminate beam</td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>-----------</td>
<td>---------------</td>
</tr>
<tr>
<td></td>
<td>L/H</td>
<td>λ</td>
</tr>
<tr>
<td>North and south segments of wall A-A'</td>
<td>0.67</td>
<td>0</td>
</tr>
<tr>
<td>Center segment of wall A-A'</td>
<td>2.2</td>
<td>0.5</td>
</tr>
<tr>
<td>Frame B-B' sagging zone</td>
<td>0.4</td>
<td>0</td>
</tr>
<tr>
<td>Frame B-B' hogging zone</td>
<td>1.1</td>
<td>0</td>
</tr>
</tbody>
</table>
Figure 1. Illustration of deformation quantities
Figure 2. Effects of E/G and neutral axis assumptions
Figure 3. Laminate beam idealization
a) Rigid body rotation equal to average ground slope

b) Two modes of deformation

Figure 4. Typical modes of deformation
Figure 5. Plan view of Warde School and excavation
Note: Approximate locations of diagonal cracks in external masonry.

Figure 6. Wall section A-A', settlement data and north and south segments laminate beam representation
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Figure 8. Results of crack assessments: wall section A-A', north segment.
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Figure 12. Results of crack assessments: frame section B-B', hogging portion.