LESSONS LEARNED FROM FIELD PERFORMANCE OF RETENTION SYSTEMS

Richard J. Finno
Outline

• Stability – Self-sinking caisson

• Serviceability
  • Establishing acceptable ground movements
  • Movement predictions and damage onset
    Excavation for Chicago-State subway renovation
  • Sources of ground movements other than stress relief
    Excavation for One Museum Park West
“You can observe a lot just by watching”

Yogi Berra, Hall of fame catcher for the NY Yankees, philosopher
Caisson sinks when:

Caisson weight > Q_s + Q_t + buoyancy

Q_s - Side resistance

Q_t - Tip resistance

Annulus?
Subsurface conditions

Fill

Glacio-lacustrine clays

Glacial outwash

Glacial till

Limestone

Water pressure in outwash and limestone is artesian
Attempts to sink caisson when glacial till reached

- Add weight at top
- Inject bentonite through ports on outside of caisson
- Undercut tip
Bentonite injection

Results of injection
(looking down from top of caisson)

Injection ports
Jetting below tip of caisson
Undercutting tip

Soil movement

$Q_s$ - Side resistance

$Q_t$ - Tip resistance
Progress through hard strata

Average tip elevation vs time

- **Elevation (ft)**
- **Date**

- **tip elevation**
- **Position 2**
- **Position 1**
Design basis

- Weight of caisson selected based on sinking
- Fully dewatered state and at-rest pressures governed compressive stresses
- Designer’s experience with sinking caissons in the area
- Treated as a “flexible” tunnel

“Our practical experience can be very misleading unless it combines with it a fairly accurate conception of the mechanics of the phenomena under consideration” - Terzaghi 1939
Relative stiffness of caisson (Peck’s tunnel concept) “wished in place”

Stress distribution on lining

$K_0 \sigma$

In-situ stresses

$\sigma$

$1/2(1-K_0)\sigma$

Deflected shape

Original shape

Rigid caisson

Flexible caisson
Comments on design

• Uniform pressures consisting of at-rest pressures representative of fully dewatered case after construction

• No consideration of construction-induced lateral stresses

• Apparently considered caisson as a deep structure – but B/D ratio ~ 0.5

• Sinking plan
  
  Strength selection should not be “conservative” in typical sense, ie. Low value is “safe”
Variations in lateral load

• Important in large diameter caissons, $D/B < 1$ i.e., a shallow foundation

• Caused by
  – stratigraphy differences
  – Property variations in same strata
  – Tilt of caisson, $1^\circ$ allowed in specifications
  – Local deformations in response to excavation
  – Localized failure as a result of undercutting toe to help advance caisson
  – Non-uniform downdrag

• Stiffness of caisson changes when cracked
Measured lateral loads post-construction

- Within ft of caisson
- Top of hard layers
- Tip of caisson

**KEY**
- WOR = Weight of Rods to advance 2 ft
- * = No Interpretable Data

**SPT**
- N-Value
- 67: 16,000 Pa (psf)
- 1.58: Ko
Concluding remarks

• Shafts with large D/B ratios are subjected to smaller horizontal stresses due to arching in horizontal plane
• Large diameter shafts can be subjected to variations in lateral loads at same elevation due to natural variations in ground and construction-induced stress changes
• Depending on ground conditions, shafts may be subjected to significant bending stresses and design must account for the resulting non-uniform stresses
“Do not design on paper what must be wished into place”

-Terzaghi-
Serviceability for deep excavations

• Assess damage potential
  – A number of methods to assess damage potential exist
  – Most relate damage to cracking of architectural details or load-bearing masonry walls
  – Wide range of limits can be calculated depending on building to be protected
  – Need estimate of movement distribution from wall

• Set by regulatory agency
• Maximum movement or distortion?
Settlements, cracking and damage

Bending

Inflection point

Diagonal shear

Deflection ratio, $\Delta_{\text{max}} / L$
## Methods to evaluate when tensile cracking develops

<table>
<thead>
<tr>
<th>Reference</th>
<th>Method type</th>
<th>Limiting parameter</th>
<th>Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burland and Wroth (1975)</td>
<td>Deep beam model of building</td>
<td>$\Delta/(L \varepsilon_{\text{crit}})$</td>
<td>Load bearing wall ($E/G = 2.6$), framed structures ($E/G = 12.5$), and masonry building ($E/G = 0.5$) with no lateral strain</td>
</tr>
<tr>
<td>Boscardin and Cording (1989)</td>
<td>Extended deep beam model</td>
<td>$\beta, \varepsilon_h$</td>
<td>$L/H = 1$ and assumption horizontal ground and building strains are equal</td>
</tr>
<tr>
<td>Son and Cording (2005)</td>
<td>Semi-empirical</td>
<td>Average strain</td>
<td>Masonry structures; need relative soil/structure stiffness; use average strain in distorting part of structure</td>
</tr>
<tr>
<td>Finno et al (2005)</td>
<td>Laminate beam model</td>
<td>$\Delta/(L \varepsilon_{\text{crit}})$</td>
<td>Load bearing walls, framed structures, masonry buildings, need bending and shear stiffness of components of walls and floors</td>
</tr>
<tr>
<td>Boone (1996)</td>
<td>Detailed analysis of structure</td>
<td>crack width</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>
</tr>
</tbody>
</table>
Burland and Wroth (1975)

\[
\Delta = \frac{P L^3}{48 EI} \left(1 + \frac{18 I E}{L^2 H G}\right)
\]

Actual building

Bending deformation with cracking due to direct tensile strain

Beam – Simple idealization of building

Deflected shape of soffit of beam

Shear deformation with cracking due to diagonal tensile strain
Burland and Wroth approach

- Relate tensile strains in beam to onset of cracking
- Use $E/G$ to define characteristic of building
  - $E/G = 2.6$ (theoretical value for $\nu = 0.3$)
  - $E/G = 0.5$ for buildings with little tensile restraint
  - $E/G = 12.5$ for buildings very flexible in shear
- Beam of unit thickness – implication is that flexural deformation depends on $E$ (rather than $EI$) and shear deformations depend on $G$ (rather than $GA_y$)
Example of range of distortions to cause damage

Neutral Axis at Center of Beam ($\lambda = 0.5$)

$\Delta/(L \epsilon_{crit})$ vs $L/H$

- $E/G = 25$
- $E/G = 12.5$
- $E/G = 2.6$
- $E/G = 0.5$

Shear Critical
Bending Critical

from Voss 2005
Alternate approach based on field performance data
<table>
<thead>
<tr>
<th>Category</th>
<th>Description of Damage</th>
<th>Crack Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>Hairline Crack.</td>
<td>&lt; 0.1 mm</td>
</tr>
<tr>
<td>Very Slight</td>
<td>Fine cracks which can easily be treated during normal decoration.</td>
<td>1 mm</td>
</tr>
<tr>
<td>Slight</td>
<td>Cracks can be easily filled. Redecoration probably required. Several slight fractures showing inside building. Cracks are visible externally.</td>
<td>5 mm</td>
</tr>
<tr>
<td>Moderate</td>
<td>Cracks may require cutting out and patching. Repointing of external brickwork. Doors and windows sticking. Service pipes may be fracture. Weather tightness often impaired.</td>
<td>5 mm to 15 mm or several cracks &gt; 3 mm</td>
</tr>
<tr>
<td>Severe</td>
<td>Extensive repair involving removal and replacement of sections of wall, especially over doors and windows. Windows and door frames distorted, floor slopes noticeably. Walls lean or bulge noticeably, some loss of bearing in beams. Utility service disrupted.</td>
<td>15 mm to 25 mm, depends on number of cracks</td>
</tr>
<tr>
<td>Very Severe</td>
<td>Major repair required involving partial or complete reconstruction. Beams lose bearing; walls lean badly and require shoring. Danger of instability.</td>
<td>Usually &gt; 25 mm, depends on number of cracks</td>
</tr>
</tbody>
</table>
Limitations for quantitative evaluation of framed structures

- Preventing cracks in architectural details
- Cracking related to tensile stresses in walls
- What are strains in walls when adjacent excavation is made? - or -
- When are walls attached to frame in terms of “self-weight” settlements that develop as building constructed?

But all methods rely on knowing the distribution of excavation-induced ground movements
Example of cracking
Excavation for Chicago-State Subway renovation
Plan view of excavation support

Excavation ramps up to ground surface

- Chicago Avenue
- Upper Level Strut (typ)
- Secant Pile Wall
- Secant Pile Wall
- State Street
- Frances Xavier Warde School
Section view of excavation support
Settlements measured in basement of school at bottom of columns
crack

Stepped crack in masonry grout
Summary of damage at Chicago-State

• The first cracks were observed at distortions greater than $1/920$
• Most damage occurred when distortion increased from $1/1000$ at end of wall installation to $1/400$ at the end of excavation
• No structural damage was observed during the project
• Observed damage characterized as “negligible” to “slight” (Burland et al., 1977)
General approach to design excavation support system

• Establish damage threshold – or meet regulatory requirements
• Estimate deformation profile at foundation level
• Design support system to meet limit movements to acceptable limits (stiffness-based design)
• Monitor

Updating design predictions during construction can be automated – “adaptive management approach”
Adaptive management – automated observational approach

Initial Design

Input

Calculation

Curves or Output

New Parameters

Minimize difference between Observed and Calculated Responses

Data Collection

Data Analysis

Curves or Output

Improved prediction/adjusted design

Prediction

Optimization

Monitoring
Movement predictions

• Depend on soil conditions, retention system stiffness and construction procedures
• Two step process
  – Precedent
  – Site specific (numerical method)
“Accurate predictions in geotechnical engineering are a result of compensating errors”

Dr. Elio D’Appolonia
Movement predictions based on precedent

• Empirical
  – Peck (1969) and Goldberg et al. (1975)

• Semi-empirical
  – Excavation and bracing cycles
    • Maximum movement
      – Clough and O’Rourke (1990) ~ lateral wall movement and settlement
      – Clough et al (1989) ~ lateral wall movement in clays
    • 3-D adjustments (Finno et al 2007)
  – Distribution of movements
    – Hsieh and Ou (1999) ~ perpendicular to wall
    – Roboski and Finno (2005) ~ parallel to wall
Normalized movements: summary

Distance from Excavation \( \frac{d}{H} \)

Normalized movements:

Clough and O'Rourke (1990)

Sands and hard clays

Limits of settlements

Clough and O'Rourke (1990)
Corner effects

PSR = $\frac{\delta_{3D}}{\delta_{ps}}$

Early stages of excavation are likely to be plane strain

Plane strain conditions

Current Analysis-Flexible Wall-All L/B, All F.S.
Current Analysis-Medium Wall-All L/B, All F.S.
Current Analysis-Stiff Wall-All L/B, All F.S.
Roboski (2004)
Data from Chew (1997)
Data from Lin (2003)
Data from Ou (1996)
Extents of settlement in Clough and O’Rourke charts are not distributions of settlements

Settlement distribution – (Hsieh and Ou 1998)

“small” cantilever movements

“large” cantilever movements
Movements parallel to wall

\[ \delta(x) = \delta_{\text{max}} \left( 1 - \frac{1}{2} \ast \text{erfc} \left( \frac{2.8 (x + L [0.015 + 0.035 \ln \frac{H_e}{L}])}{0.5L + L [0.015 + 0.035 \ln \frac{H_e}{L}]} \right) \right) \]
Estimate lateral movements in clays – semi-empirical (Clough et al. 1989)

Free field movements
Presence of building adjacent to excavation affects movements

25% reduction of maximum free field settlement

two factors: lower stress from basement stiffness of building
Empirical methods mostly developed by 1990

• Developments since then
  – Top down construction
  – Deep mix slurry walls
  – Hybrid support systems
  – Ground improvement for movement control
  – Use of cross-walls

• How applicable are empirical methods without correction?
Movements from causes other than excavation and bracing cycles

- Removal of existing foundations
- Wall installation
  - Densification of sands from vibrations
  - Displacements arising during installation
    - Slurry or secant pile wall
    - Sheet-pile wall
- Deep foundation installation
- Concrete shrinkage during top-down construction
Secant pile wall installation

Lateral Deformation (in)

0.4 0.2 0.0

Elevation, ft CCD

Sand
Soft Clay
Medium Clay
Stiff Clay
Hard Clay
Fill
Drilled shaft installation

Baker and Lukas (1978)

Baker and Gill (1985)

Significant movements
One Museum Park West project

Illustrate the impact of construction activities on the ground movements caused by excavations

“Nominally” top down construction
1. Excavation removed from critical path
2. Overexcavation is prevented
3. Relatively high stiffness
4. Temporary support system is also permanent
<table>
<thead>
<tr>
<th>Stage</th>
<th>Activity</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Perimeter pile wall and foundation installation</td>
<td>Level site&lt;br&gt;Install perimeter pile walls&lt;br&gt;Install caissons</td>
</tr>
<tr>
<td>2</td>
<td>Central core construction (Bottom-up)</td>
<td>Install sheet pile wall&lt;br&gt;Cycles of excavation and bracing&lt;br&gt;Place reinforced concrete mat&lt;br&gt;Construct core</td>
</tr>
<tr>
<td>3</td>
<td>Basement construction</td>
<td>Top-down construction</td>
</tr>
</tbody>
</table>
**INSTRUMENTATION**

- 5 inclinometers
- 101 settlement points
- 5 Strain gage stations: total of 72 strain gages
Perimeter wall and drilled shaft construction

Fully cased until tangent section reached
Central core construction
Top down construction
Summary of observed settlements

- Secant pile walls and foundations = 35%
- Cofferdam/Concrete core = 30%
- Top-down excavation = 35%
Concrete material time-dependence at One Museum Park West project

To quantify: FE analysis of below grade structural components
Concrete effects = 30% of the max. lateral displacement

Concluding remarks

• Methods to evaluate impacts of damage are semi-empirical – trying to protect architectural details
• Distribution of excavation-induced ground movements is a two-step process: empirical and FE analyses
• The *process* of predicting, monitoring and updating (adaptive management) is a useful design tool
• At times, most economical design is one where limited damage to adjacent structure occurs and contractor repairs it
• If one does not think hard about construction in the design stage of a project, unexpected performance is likely
Acknowledgements

- Schnabel Foundation Company
- Hayward Baker
- Case Foundation
- Thatcher Engineering Corp.
- GeoEngineers, Inc.
- WJE & Associates
- STS Consultants
- Ground Engineering, Inc.

- Sebastian Bryson, Michele Calvello, Paul Sabatini, Dan Priest, Jill Roboski, Kristi Kawamura, Tanner Blackburn, Terry Holman, Wanjei Cho, Young-Hoon Jung, Greg Andrianis, Miltos Langousis, Cecilia Rechea, Taesik Kim, Fernando Sarabia, Luis Arboleda, Charlotte Insull, Zhenhao Shi – former Northwestern University graduate students

- Turner Construction
- Walsh Construction
- O’Neill Construction
- Skanska
- Aldridge Drilling
- DBM
- Board of Underground – City of Chicago

- Funding provided by National Science Foundation and Infrastructure Technology Institute at Northwestern University
References


