Geo Virginia/ October 2016 / Williamsburg, VA

LESSONS LEARNED FROM FIELD PERFORMANCE OF RETENTION SYSTEMS

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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

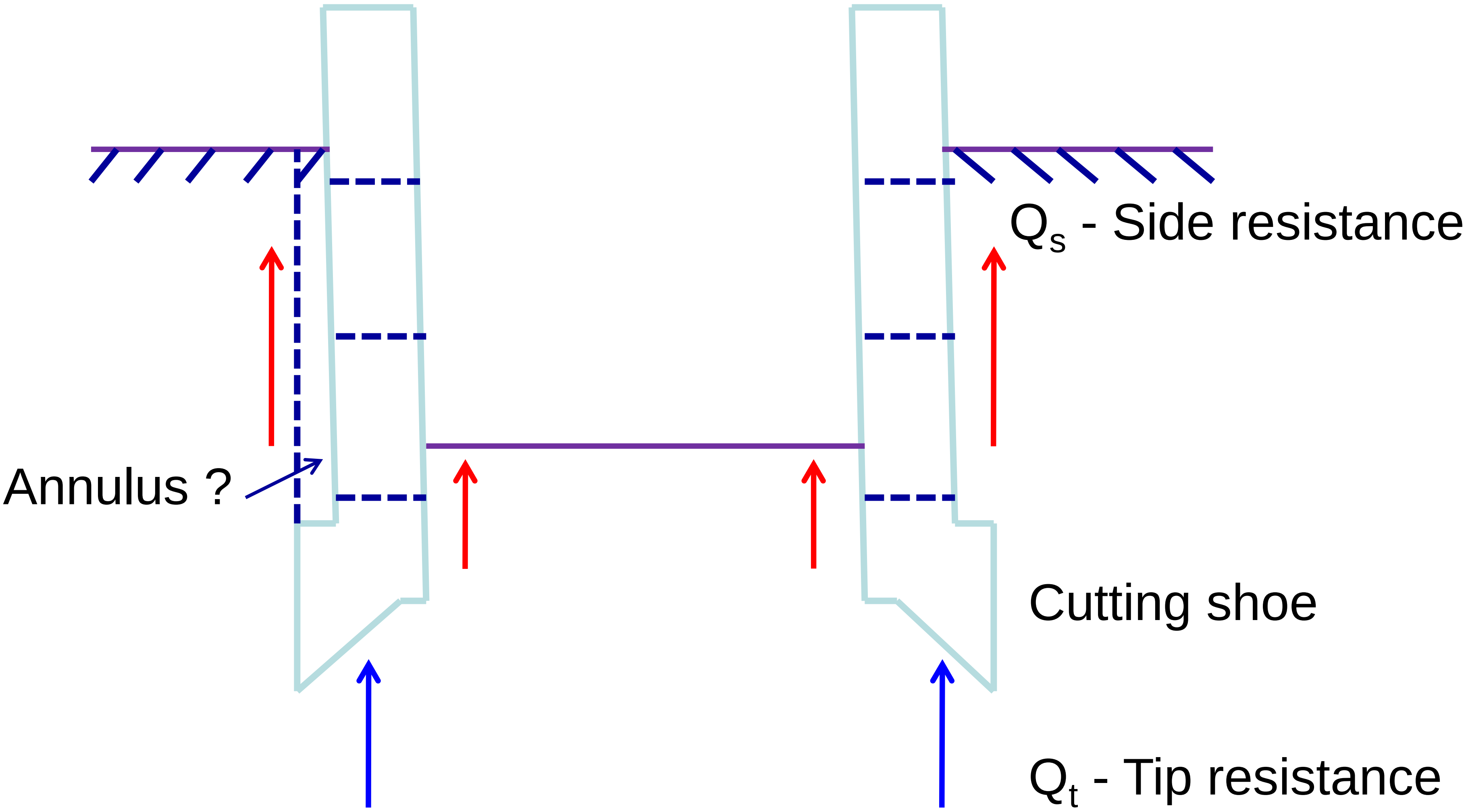
“You can observe a lot just by watching”

Yogi Berra, Hall of fame
catcher for the NY Yankees,
philosopher

2006 10 10

Caisson sinks when:

Caisson weight > Qs + Qt + bouyancy



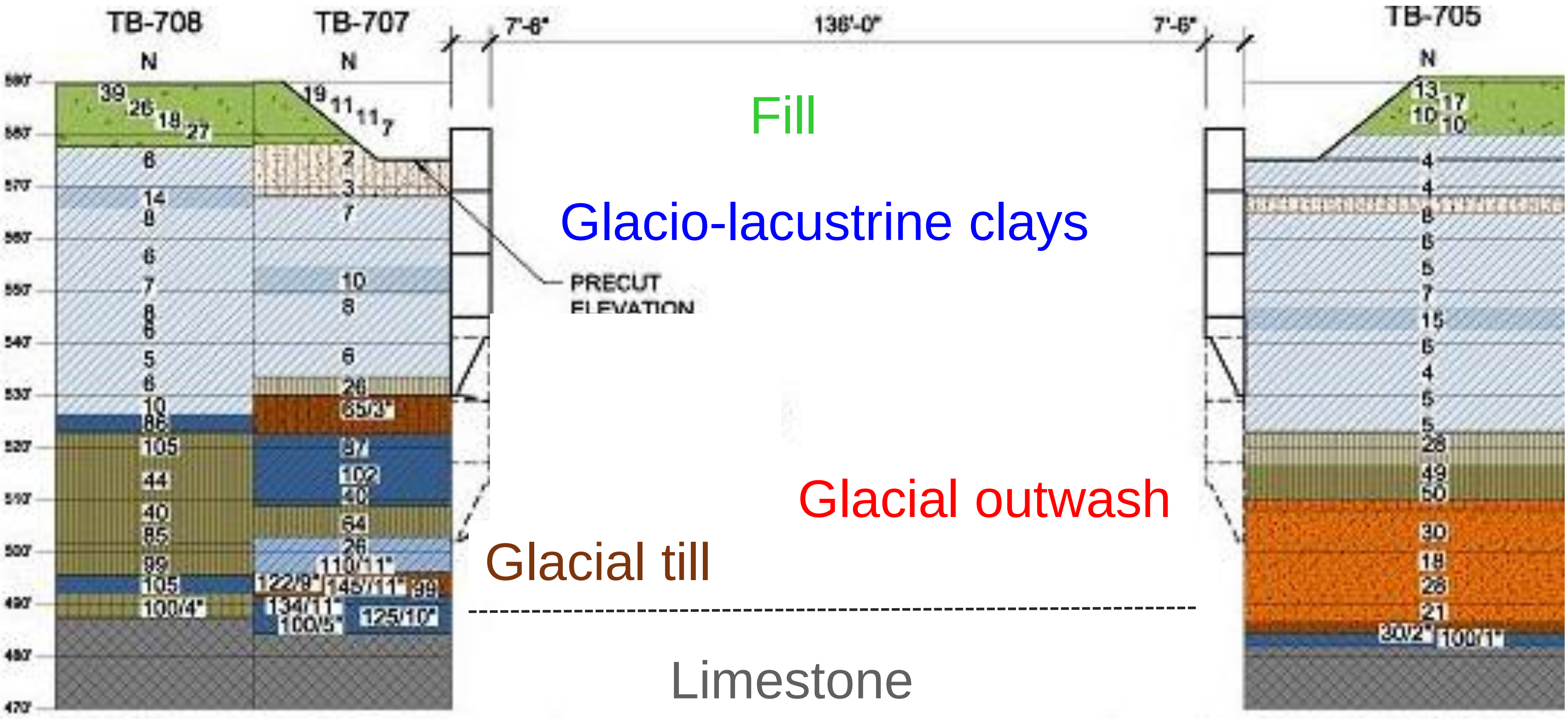
Q_s - Side resistance

Annulus ?

Cutting shoe

Q_t - Tip resistance

Subsurface conditions



Fill

Glacio-lacustrine clays

Glacial outwash

Glacial till

Limestone

Water pressure in outwash and limestone is artesian


9
12
6
3


APR 7 2006

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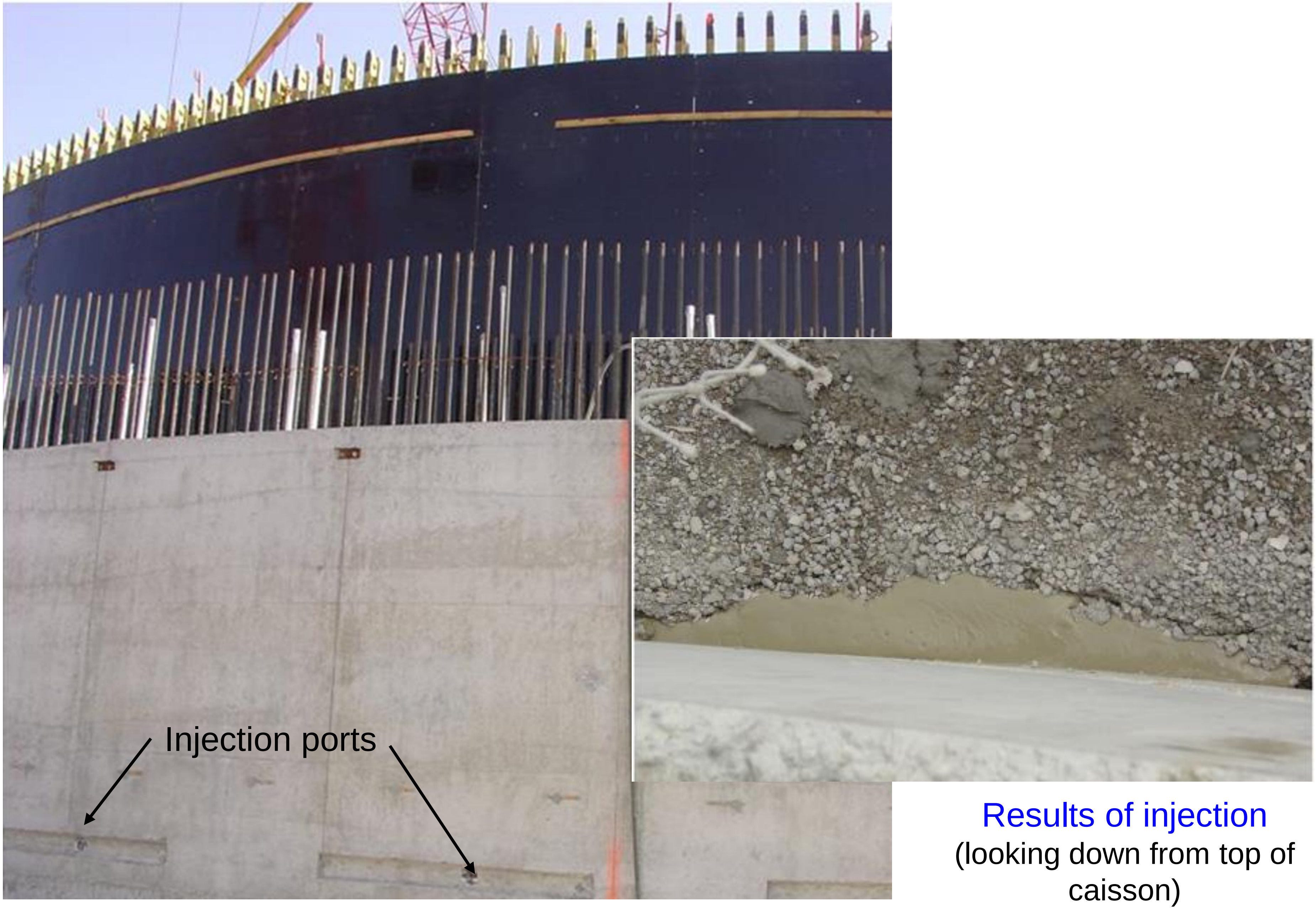
Attempts to sink caisson when glacial till reached

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

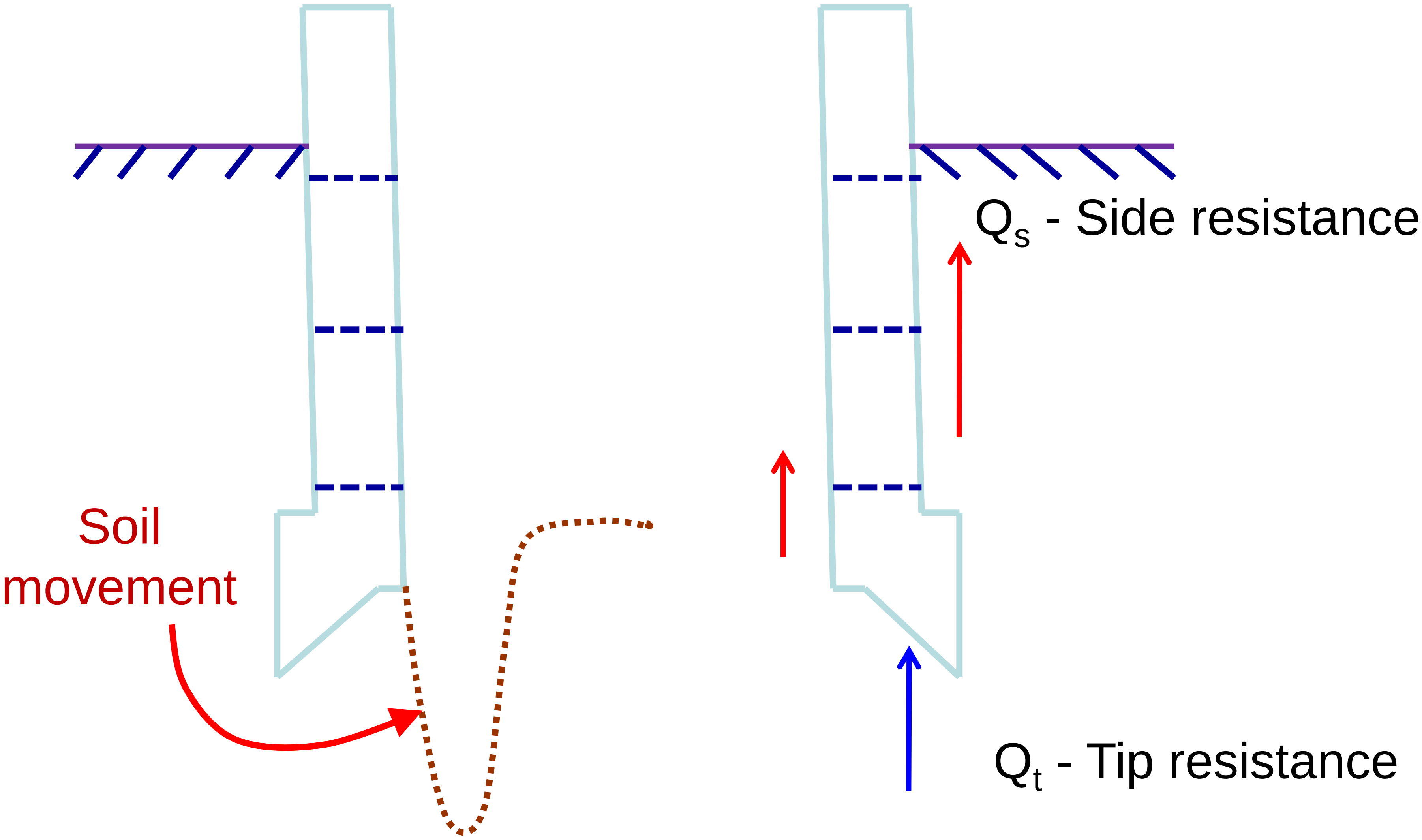
Bentonite injection



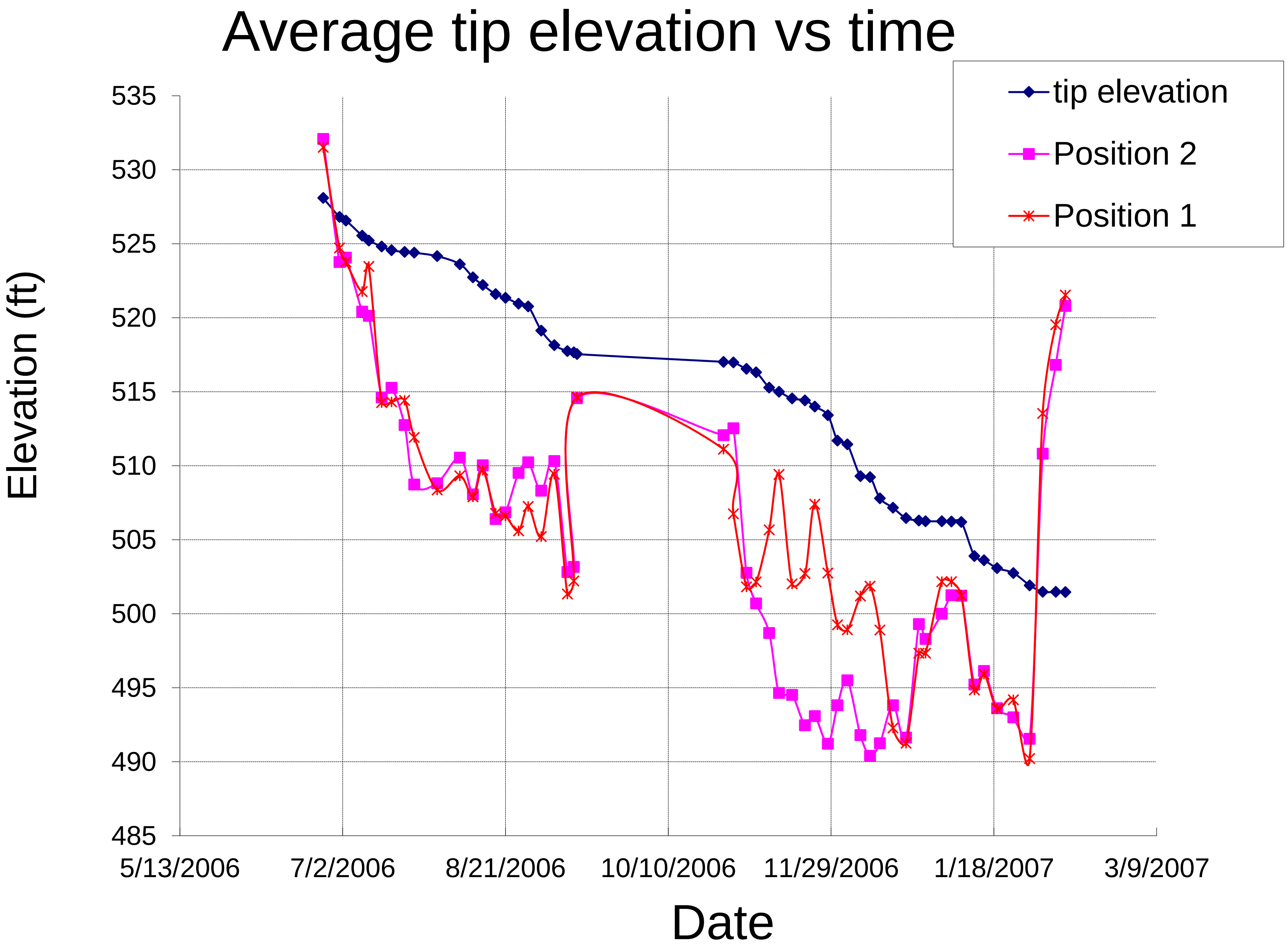
Results of injection
(looking down from top of caisson)

Jetting below tip of caisson

Undercutting tip



Progress through hard strata




2006 9 6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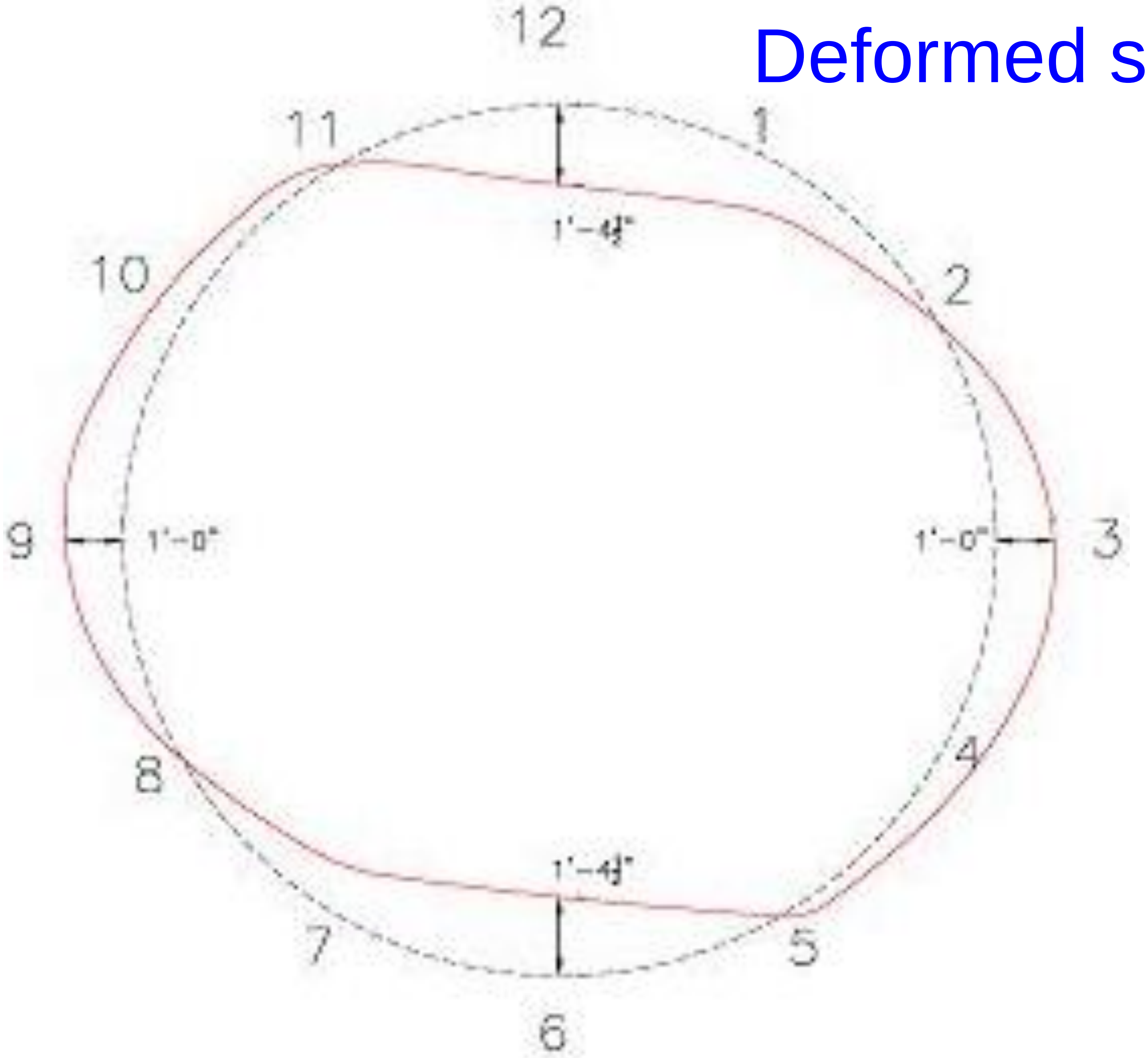
Deformed shape



12:00 position

6:00 position

2007 3 6

3:00 position

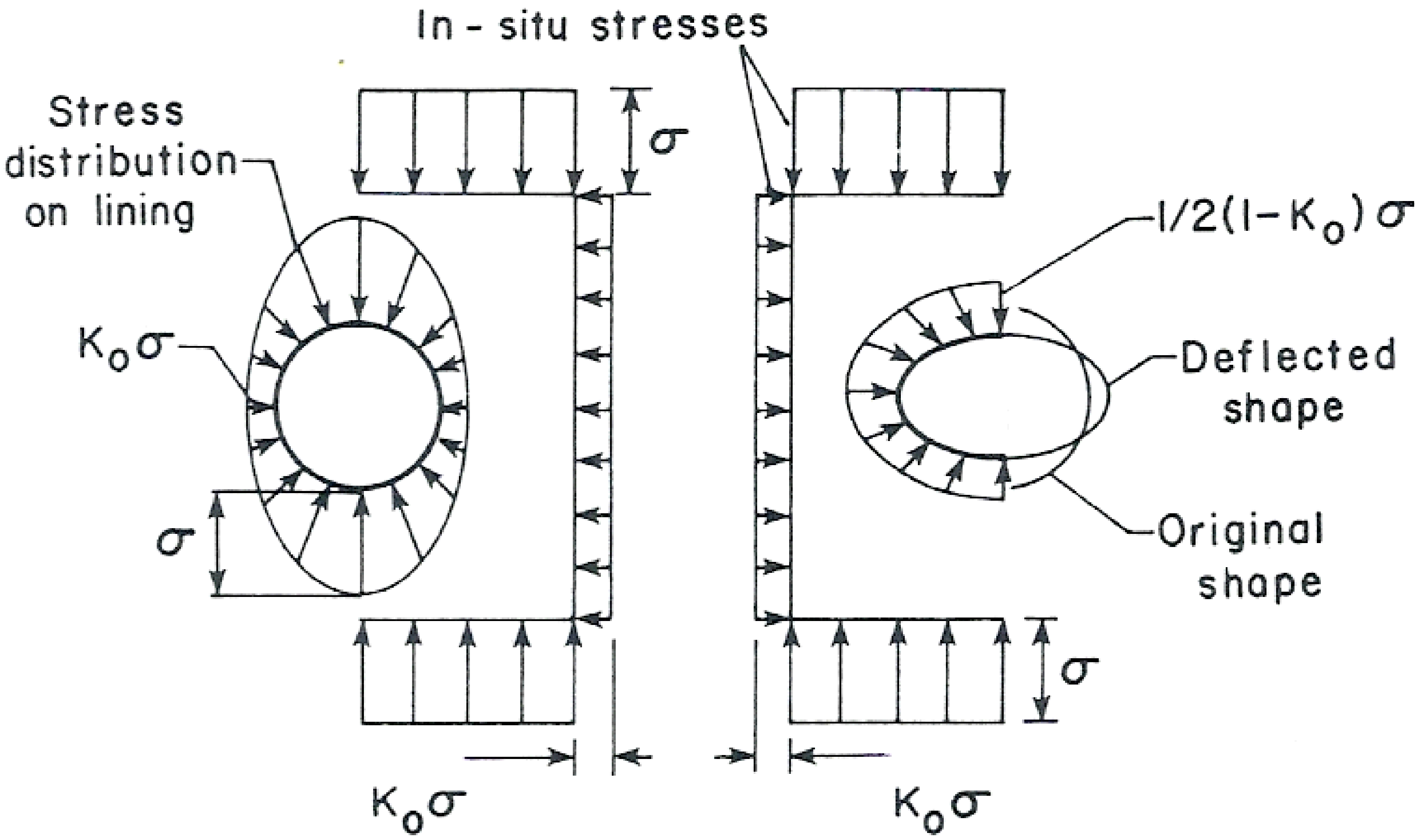
Hinge

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"



Rigid caisson

Flexible caisson

Comments on design

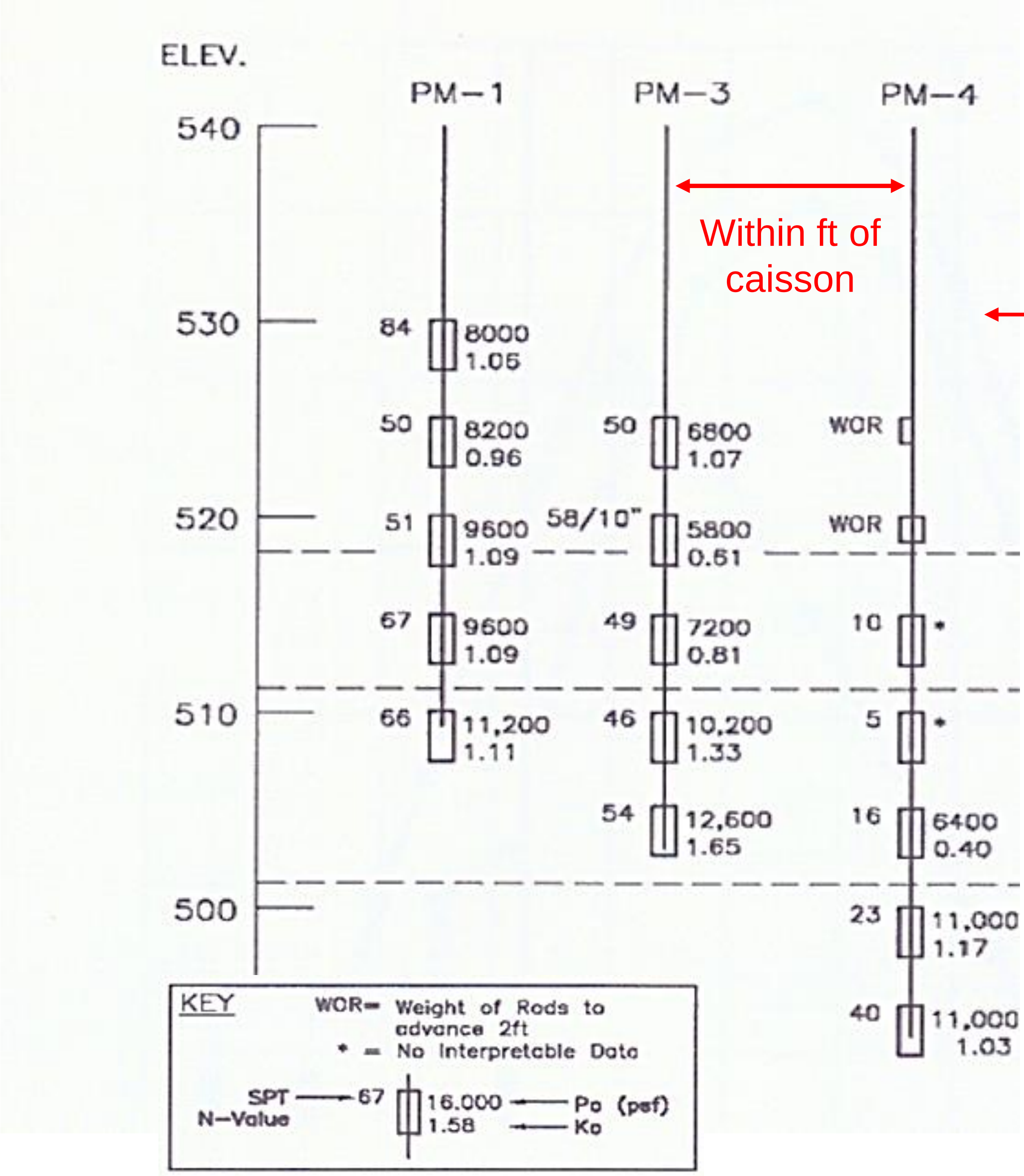
- Uniform pressures consisting of at-rest pressures representative of fully dewatered case after construction
- No consideration of construction-induced lateral stresses
- Apparently considered caisson as a deep structure – but B/D ratio ~ 0.5
- Sinking plan

  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° 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

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

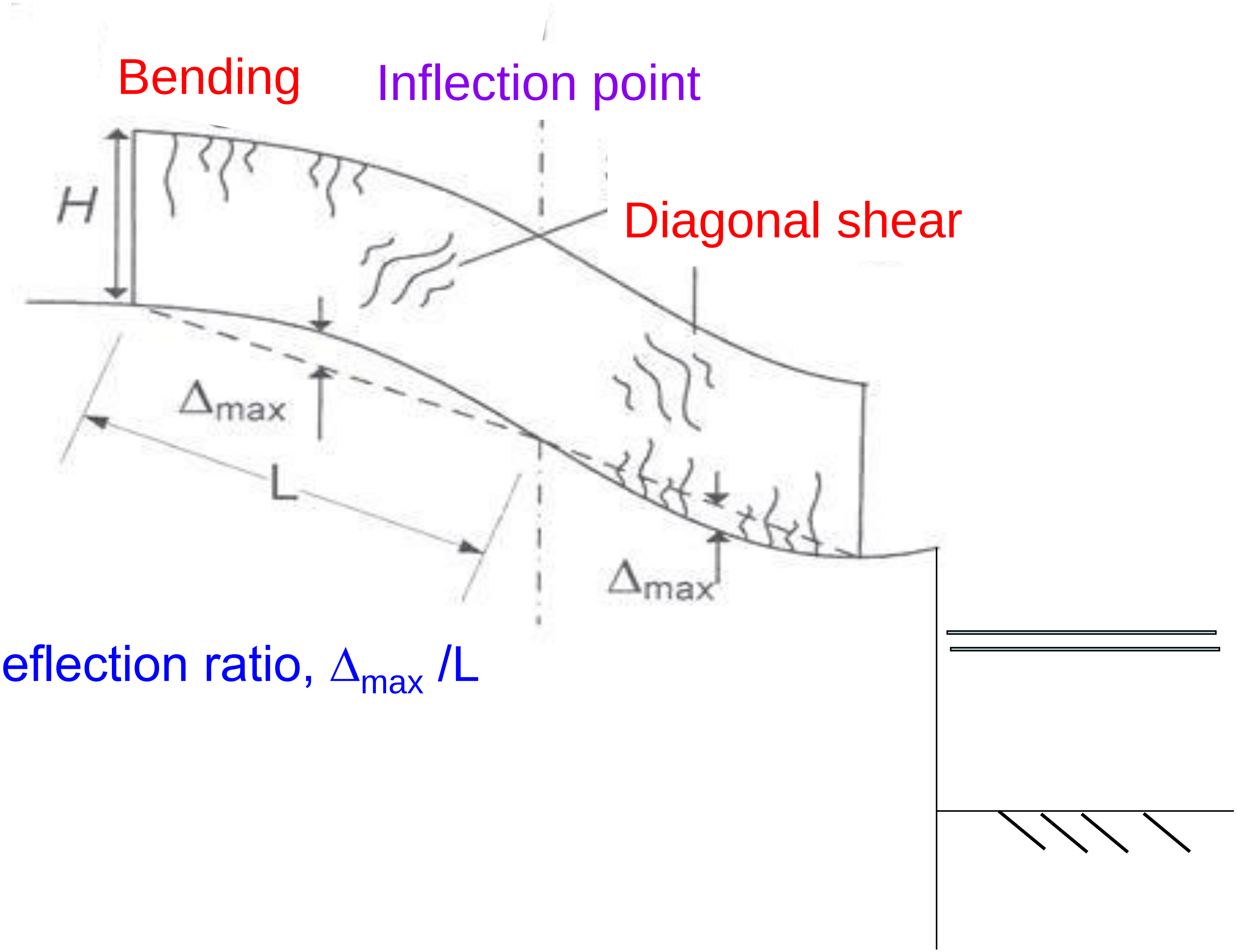
“Do not design on paper what must be wished into place”

-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

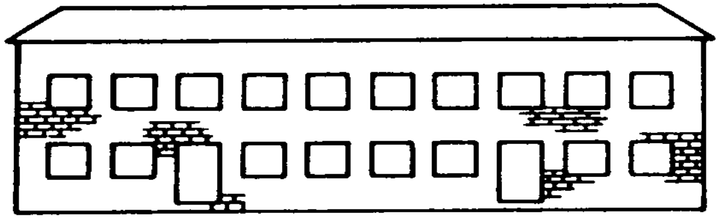
Deflection ratio, Δ_{max} /L

Methods to evaluate when tensile cracking develops

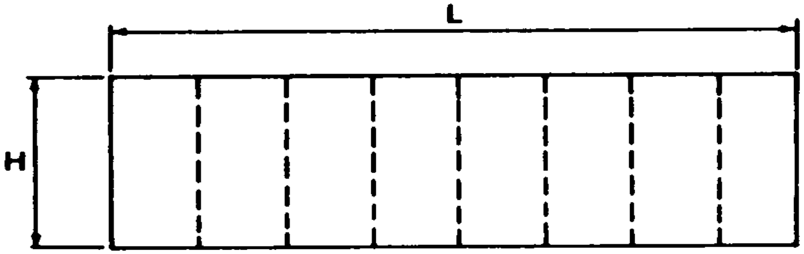
Reference	Method type	Limiting parameter	Applicability
Burland and Wroth (1975)	Deep beam model of building	$\Delta /(L\ \varepsilon_{crit})$	Load bearing wall (E/G = 2.6), framed structures (E/G = 12.5), and masonry building (E/G = 0.5) with no lateral strain
Boscardin and Cording (1989)	Extended deep beam model	β, ε_h	L/H = 1 and assumption horizontal ground and building strains are equal
Son and Cording (2005)	Semi-empirical	Average strain	Masonry structures; need relative soil/structure stiffness; use average strain in distorting part of structure
Finno et al (2005)	Laminate beam model	$\Delta /(L\ \varepsilon_{crit})$	Load bearing walls, framed structures, masonry buildings, need bending and shear stiffness of components of walls and floors
Boone (1996)	Detailed analysis of structure	crack width	general procedure that considers bending and shear stiffness of building sections, distribution of ground movements, slip between foundation and grade and building configuration

Burland and Wroth (1975)

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

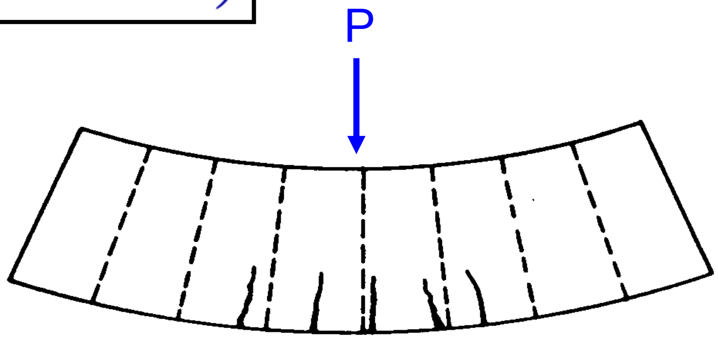


Actual building

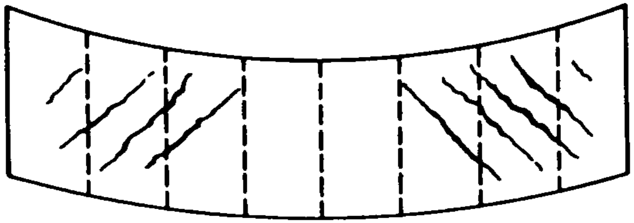


Beam – Simple idealization of building

Deflected shape of soffit of beam



Bending deformation with cracking due to direct tensile strain

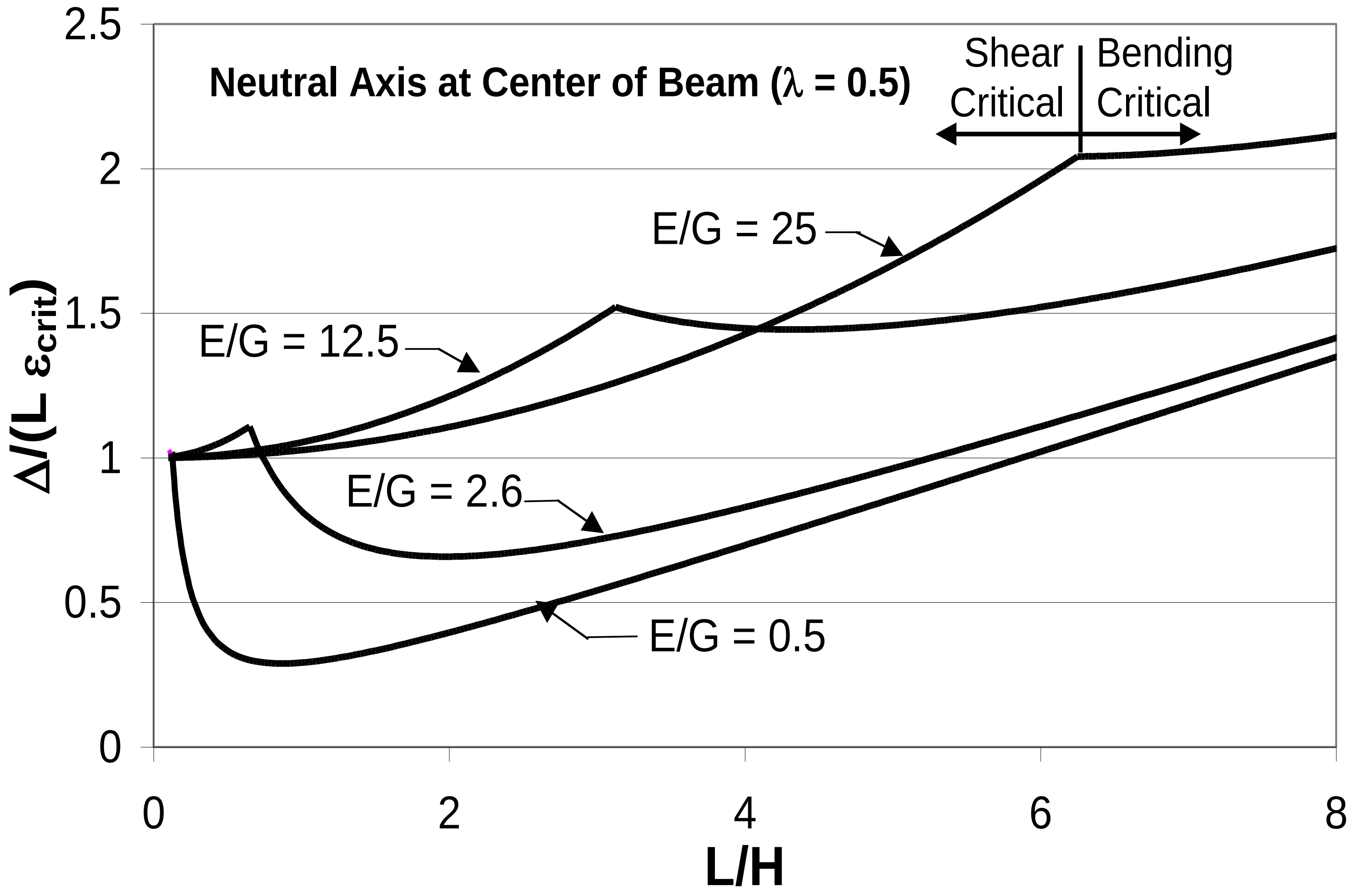


Shear deformation with cracking due to diagonal tensile strain

Burland and Wroth approach

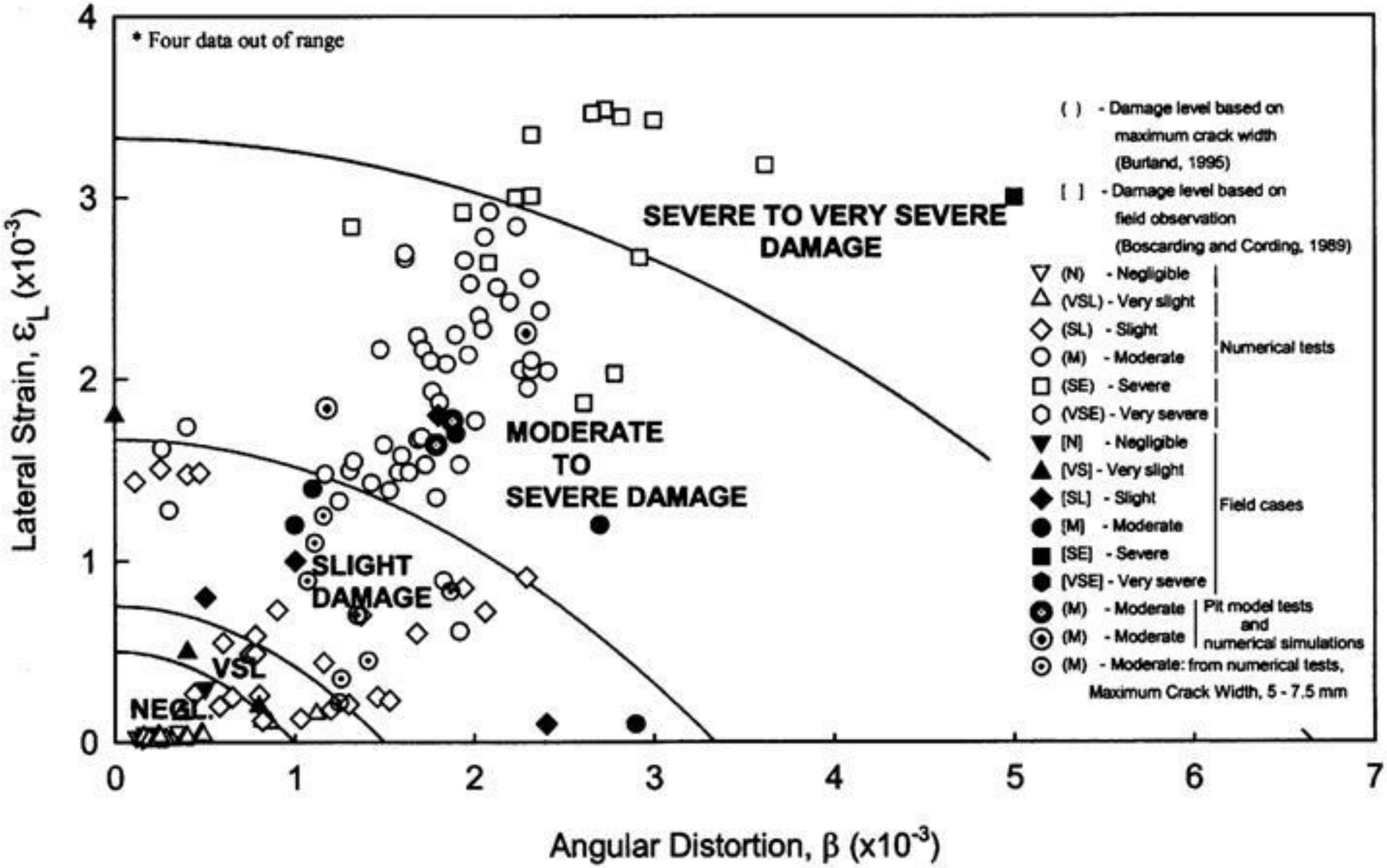
- Relate tensile strains in beam to onset of cracking
- Use E/G to define characteristic of building
- E/G = 2.6 (theoretical value for $\upsilon = 0.3$)
- E/G = 0.5 for buildings with little tensile restraint
- E/G = 12.5 for buildings very flexible in shear
- Beam of unit thickness – implication is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

Example of range of distortions to cause damage



from Voss 2005

Alternate approach based on field performance data



Son and Cording 2005

DAMAGE CLASSIFICATION AFTER BURLAND AND WROTH (1975)

Category	**Description of Damage**	**Crack Width**
Negligible	Hairline Crack.	< 0.1 mm
Very Slight	Fine cracks which can easily be treated during normal decoration. Cracks in exterior brickwork visible on close inspection.	1 mm
Slight	Cracks that can be easily filled. Redecoration probably required. Several slight fractures showing inside building. Cracks are visible externally.	5 mm
Moderate	Cracks may require cutting out and patching. Repointing of external brickwork. Doors and windows sticking. Service pipes may be fracture. Weather tightness often impaired.	5 mm to 15 mm or several cracks > 3 mm
Severe	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.	15 mm to 25 mm, depends on number of cracks
Very Severe	Major repair required involving partial or complete reconstruction. Beams lose bearing; walls lean badly and require shoring. Danger of instability.	Usually > 25 mm, depends on number of cracks

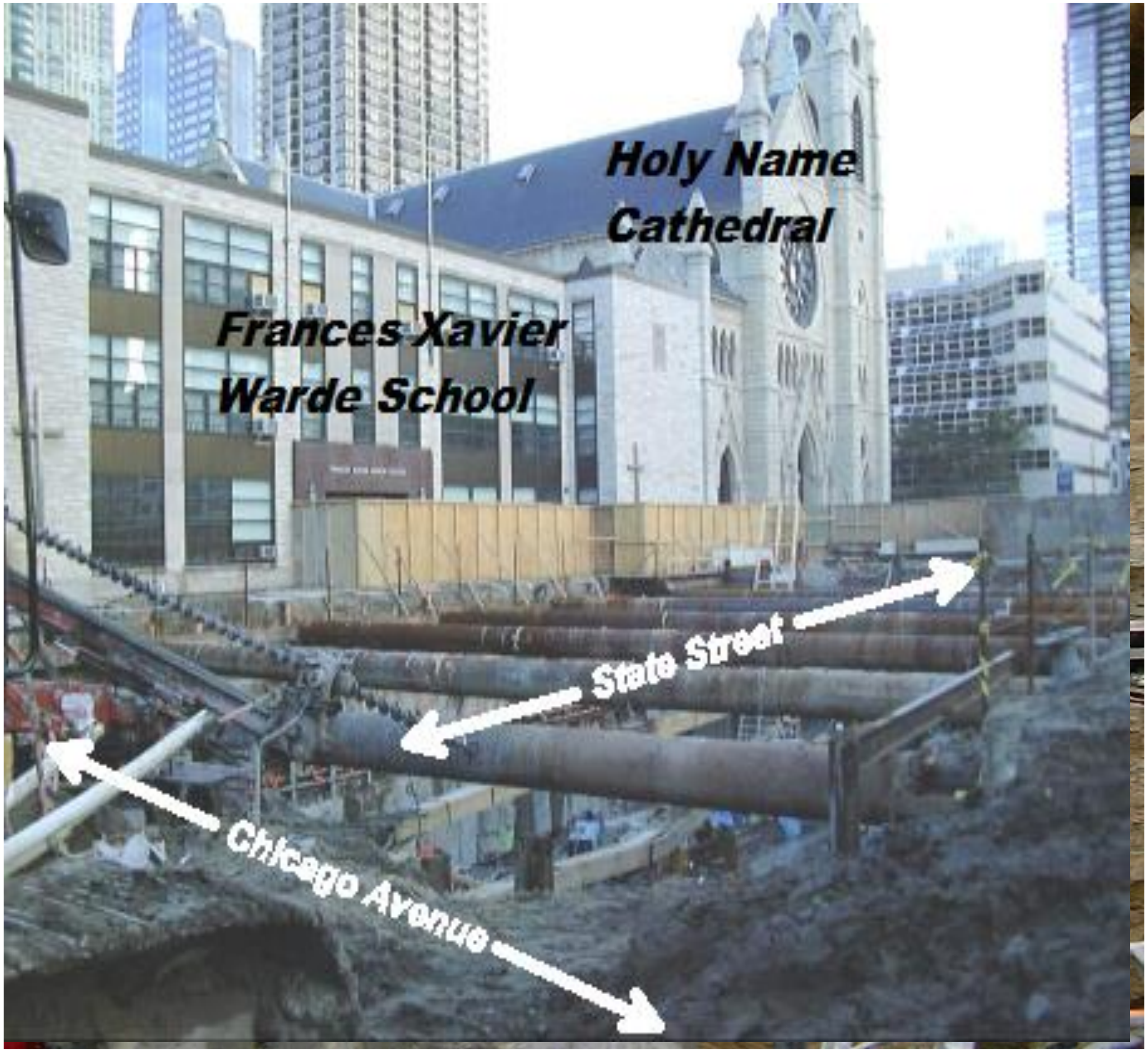
Limitations for quantitative evaluation of framed structures

- 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?

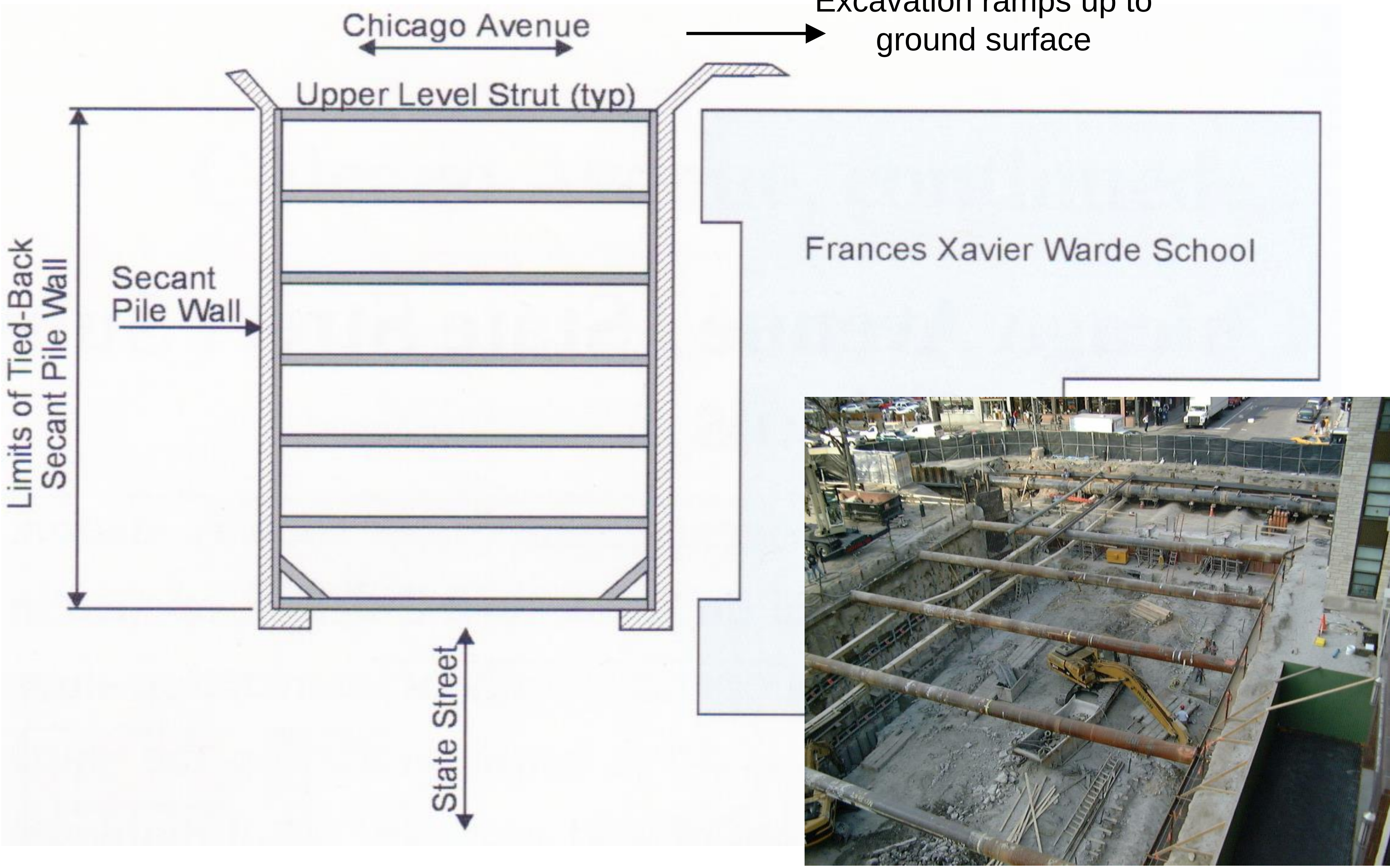
But all methods rely on knowing the distribution of excavation-induced ground movements

Example of cracking

Excavation for Chicago-State Subway renovation

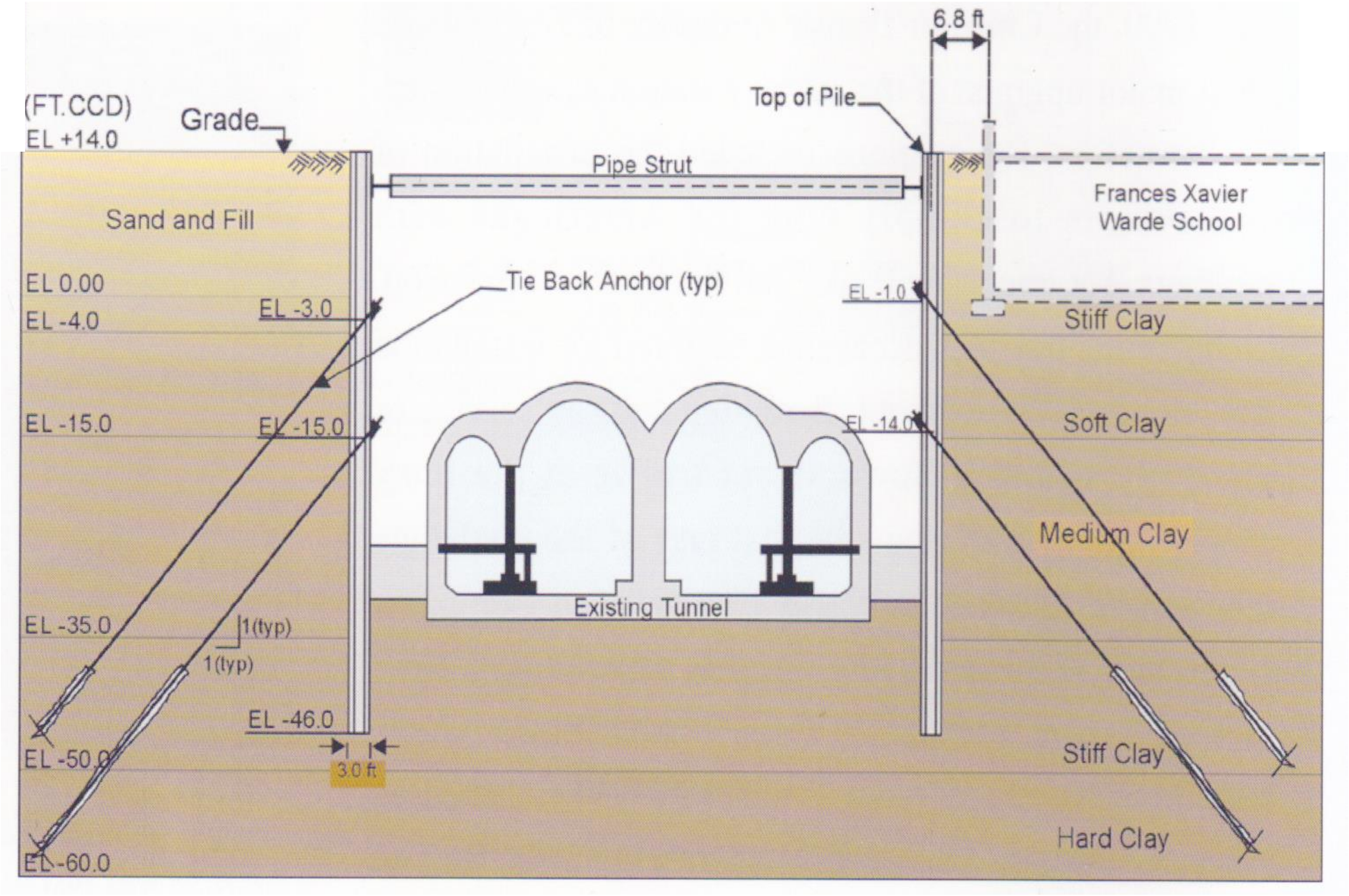


Plan view of excavation support

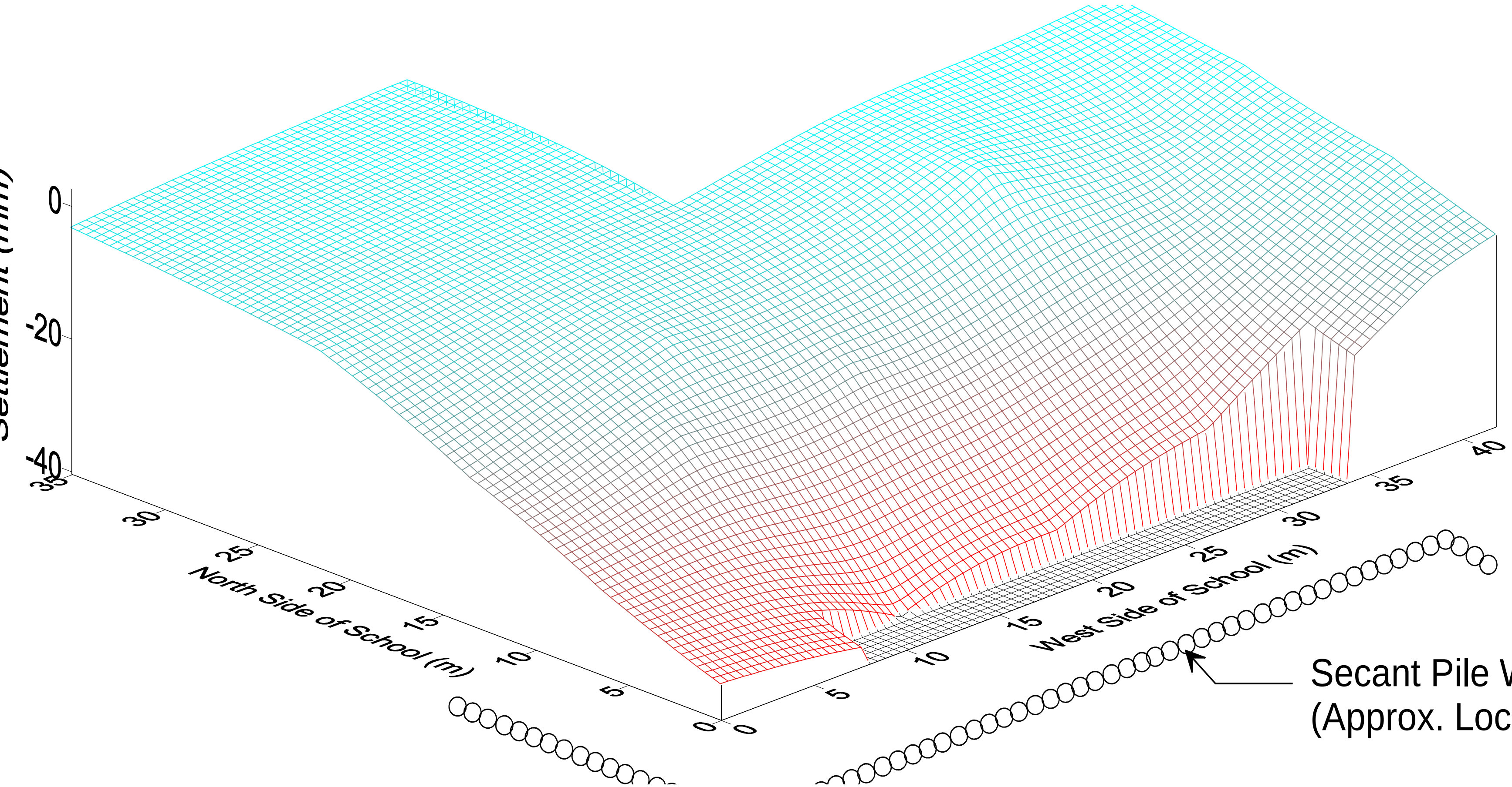


Excavation ramps up to ground surface

Section view of excavation support

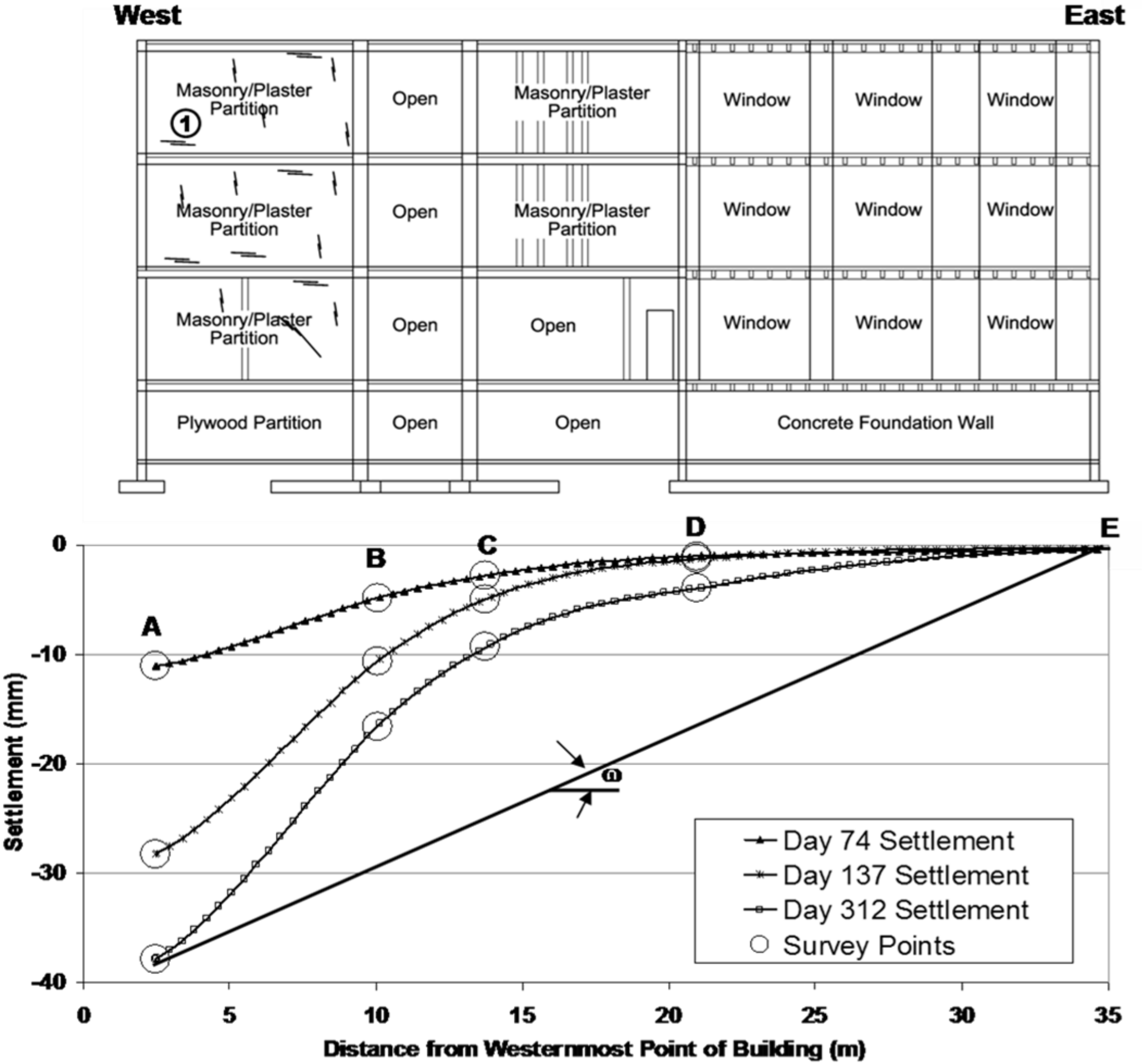


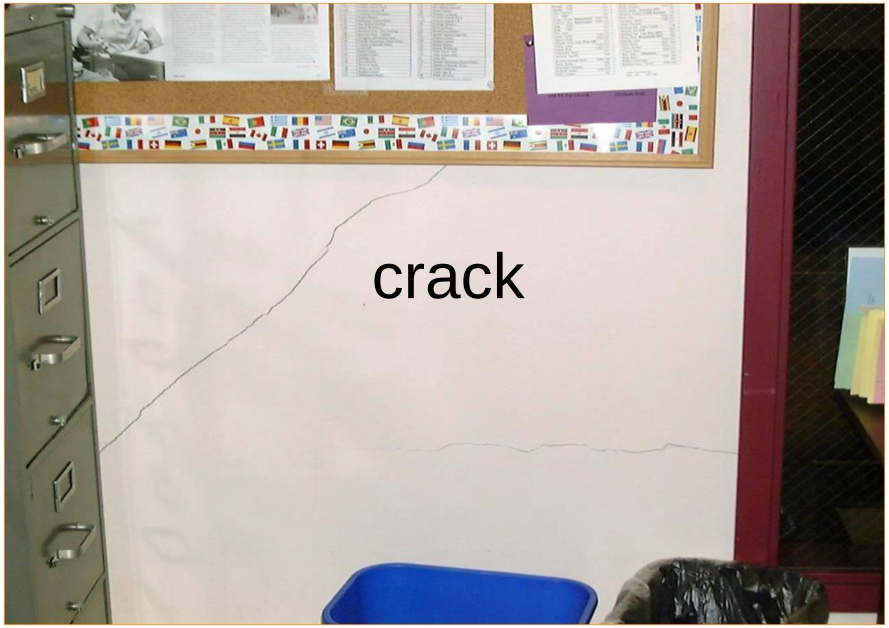
Settlements of Warde School at end of excavation



Secant Pile Wall
(Approx. Location)

Settlements measured in basement of school at bottom of columns


West
East
Masonry/Plaster Partition
①
Open
Masonry/Plaster Partition
Window
Window
Window
Masonry/Plaster Partition
Open
Masonry/Plaster Partition
Window
Window
Window
Masonry/Plaster Partition
Open
Open
Window
Window
Window
Plywood Partition
Open
Open
Concrete Foundation Wall
A
B
C
D
E
ω
Settlement (mm)
0
-10
-20
-30
-40
Distance from Westernmost Point of Building (m)
0
5
10
15
20
25
30
35
Day 74 Settlement
Day 137 Settlement
Day 312 Settlement
Survey Points


crack


crack


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)

1/400

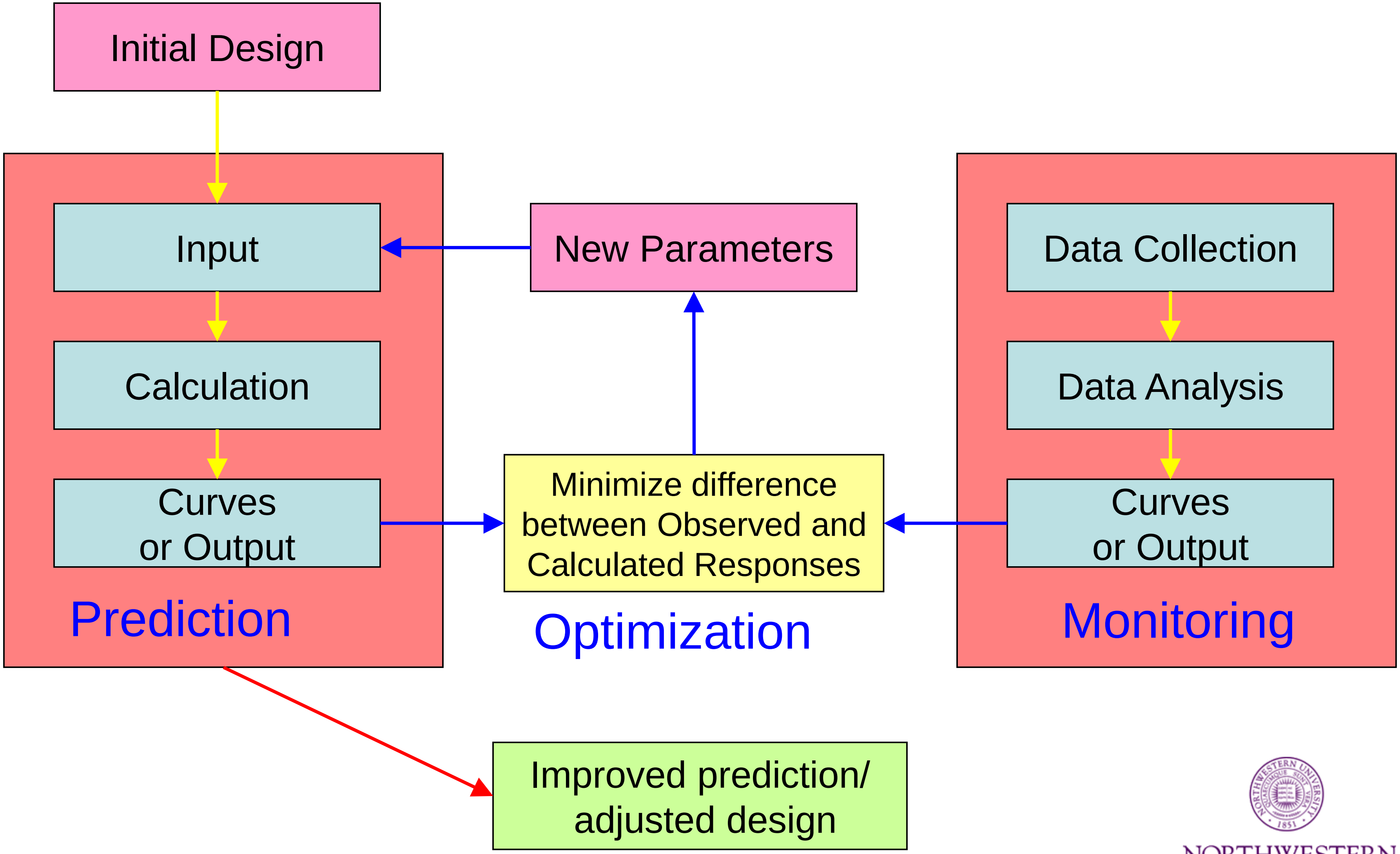
1/1000

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

Prediction

New Parameters

Minimize difference
between Observed and
Calculated Responses

Optimization

Data Collection

Data Analysis

Curves
or Output

Monitoring

Improved prediction/
adjusted design

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Movement predictions

- Depend on soil conditions, retention system stiffness and construction procedures
- Two step process
  - Precedent
  - Site specific (numerical method)

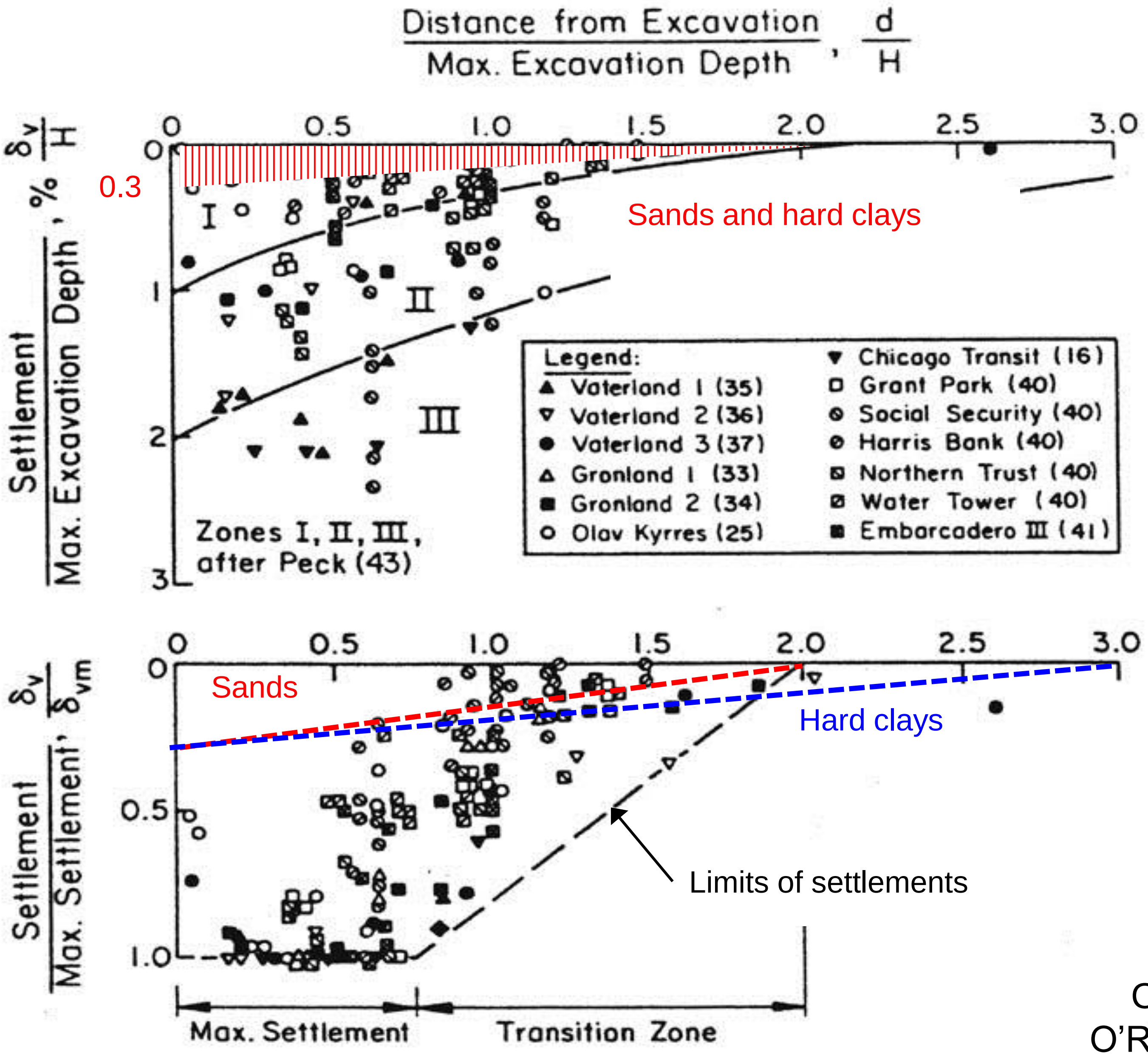
“Accurate predictions in geotechnical engineering are a results of compensating errors”

Dr. Elio D’Appolonia

Movement predictions based on precedent

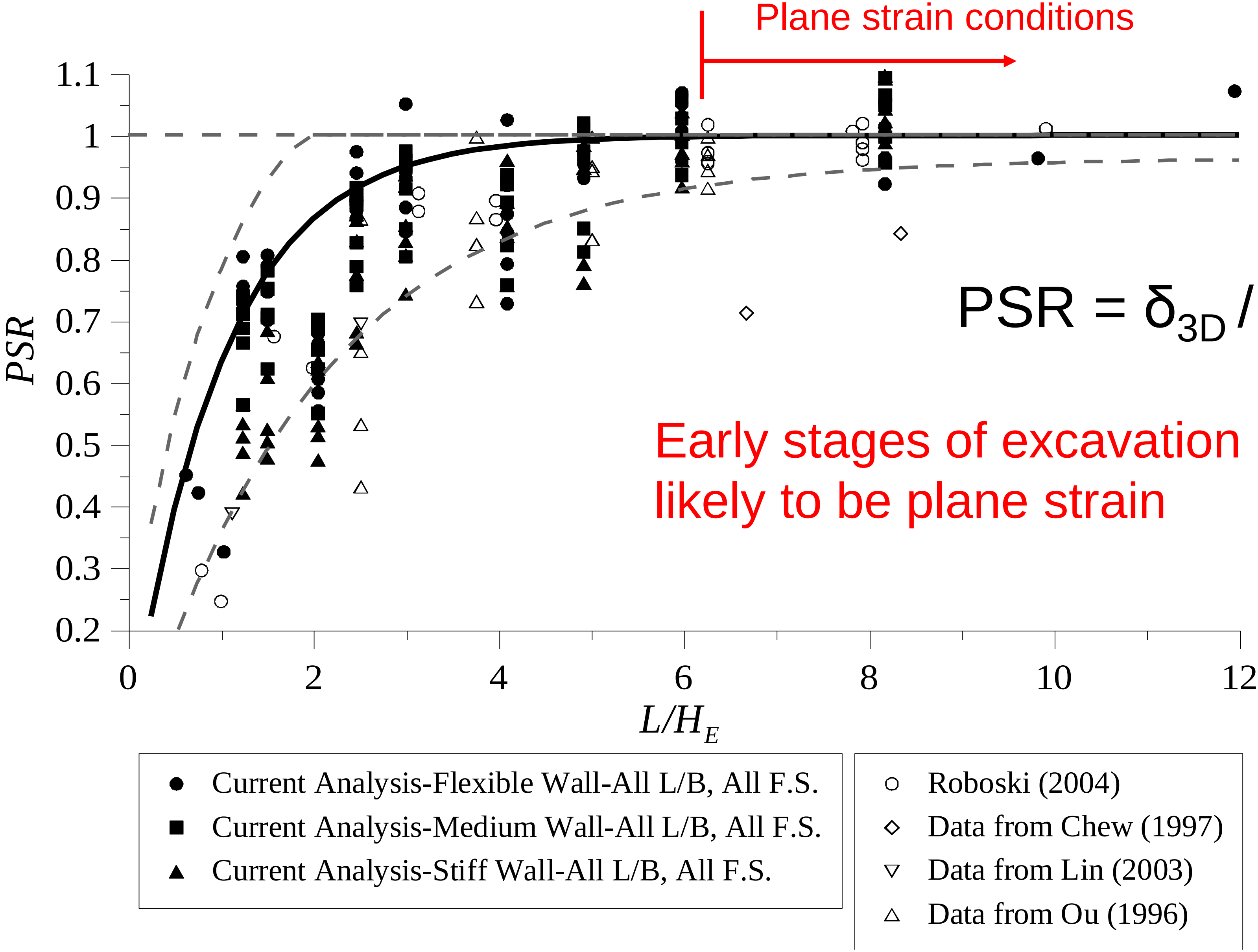
- Empirical
  - Peck (1969) and Goldberg et al. (1975)
- Semi-empirical
  - Excavation and bracing cycles
    - Maximum movement
      - Clough and O’Rourke (1990) ~ lateral wall movement and settlement
      - Clough et al (1989) ~ lateral wall movement in clays
      - 3-D adjustments (Finno et al 2007)
    - Distribution of movements
      - Hsieh and Ou (1999) ~ perpendicular to wall
      - Roboski and Finno (2005) ~ parallel to wall

Normalized movements: summary



Clough and O’Rourke (1990)

Corner effects

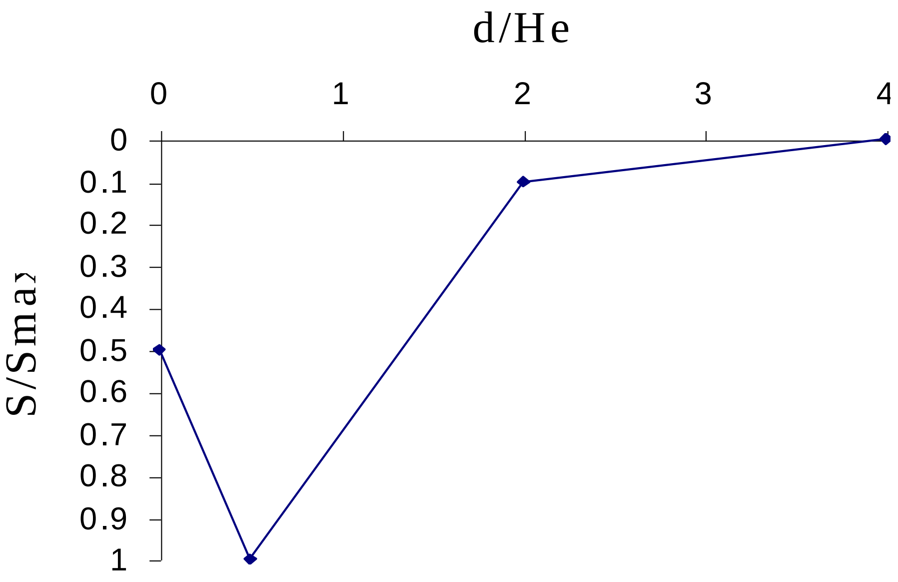


$PSR = \delta_{3D} / \delta_{ps}$

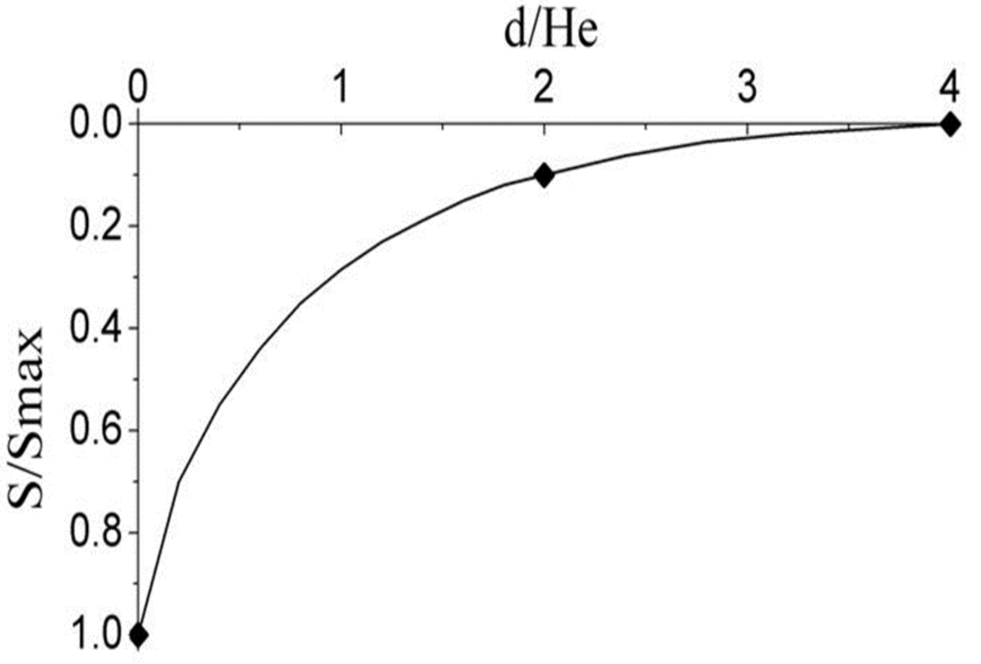
Early stages of excavation are likely to be plane strain

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

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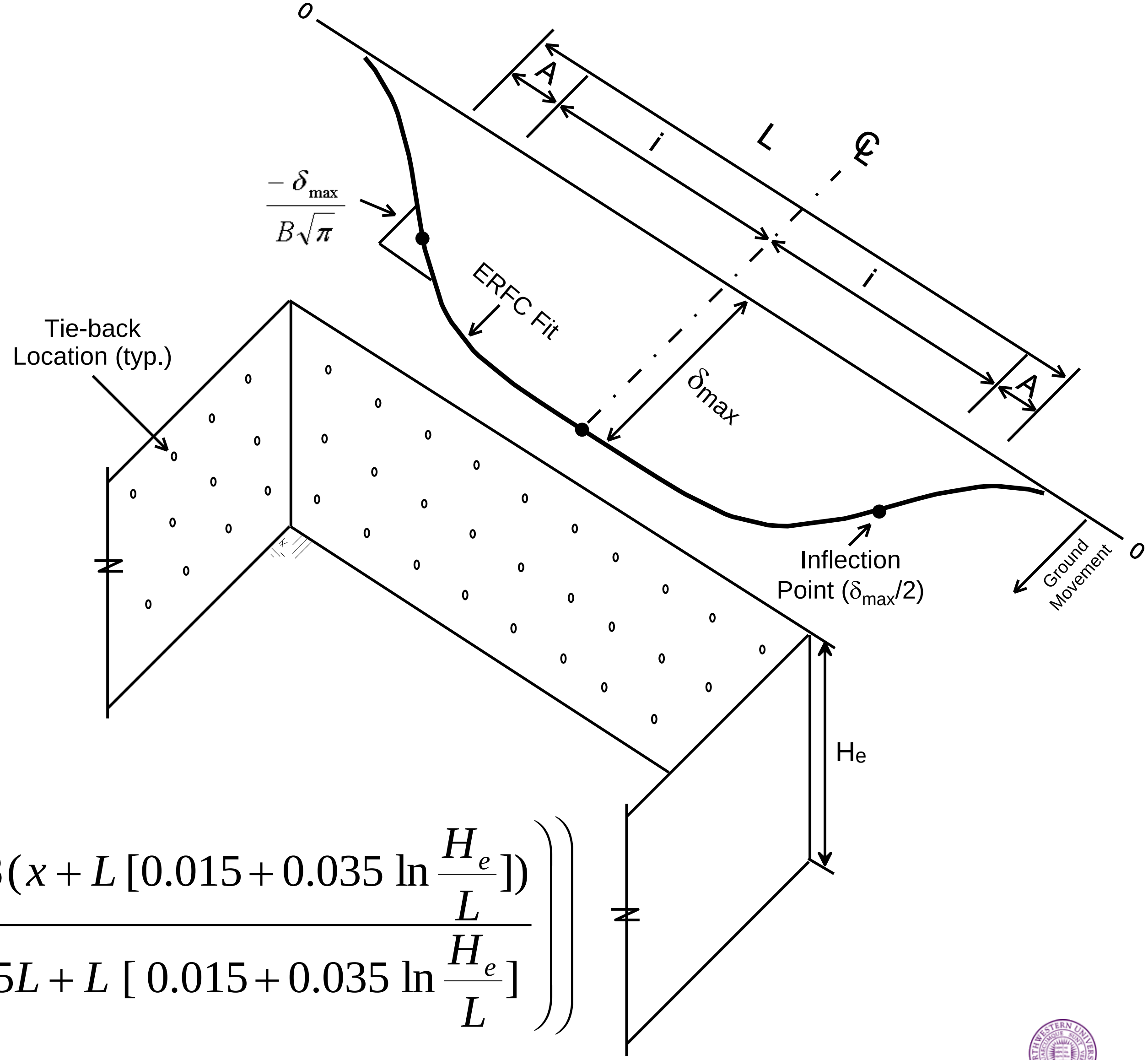
Movements parallel to wall



$$\delta(x)=\delta_{\max}\left(1-\frac{1}{2}*erfc\left(\frac{2.8(x+L\,[0.015+0.035\,\ln\frac{H_e}{L}])}{0.5L+L\,[\,0.015+0.035\,\ln\frac{H_e}{L}]}\right)\right)$$

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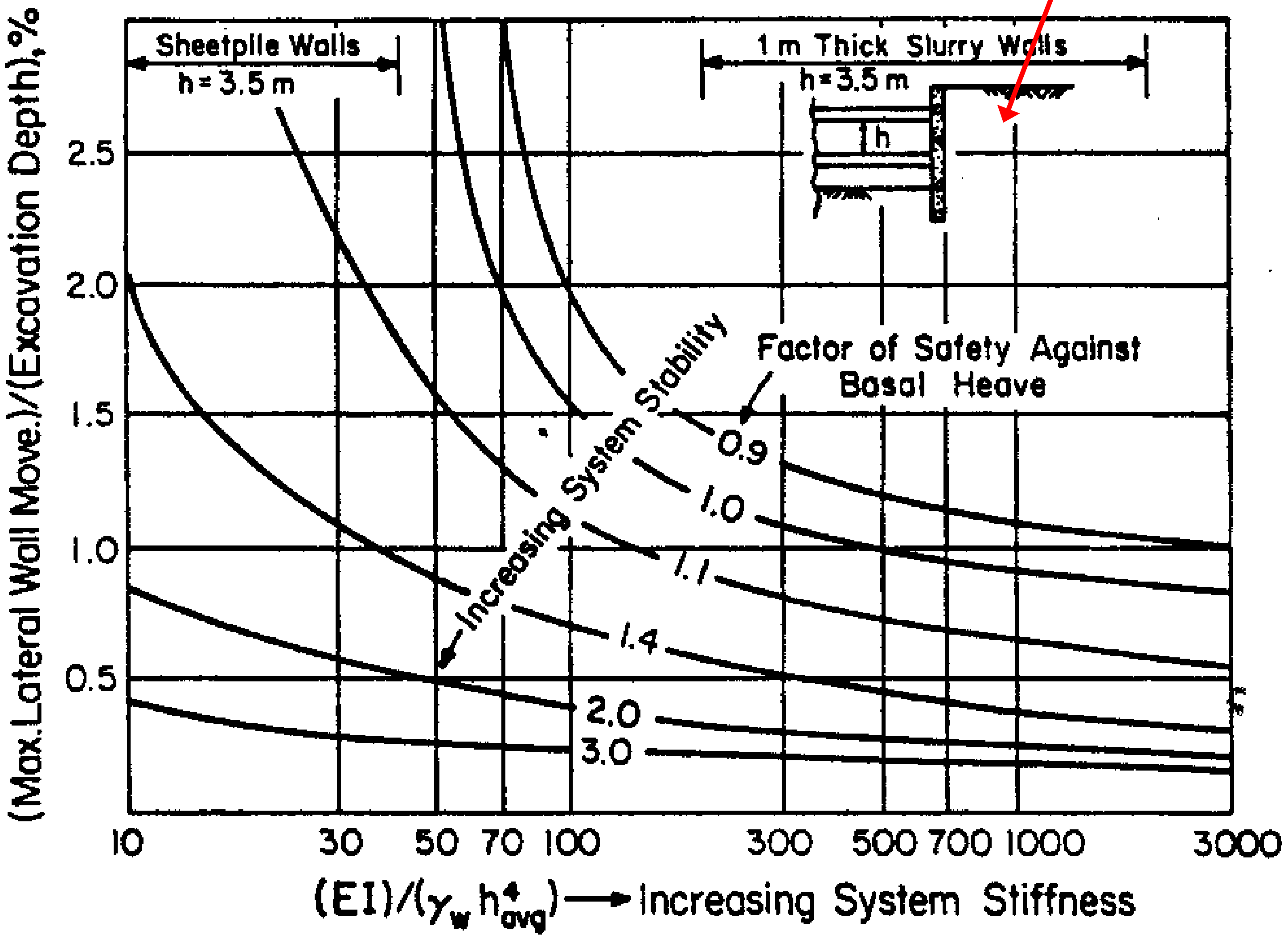
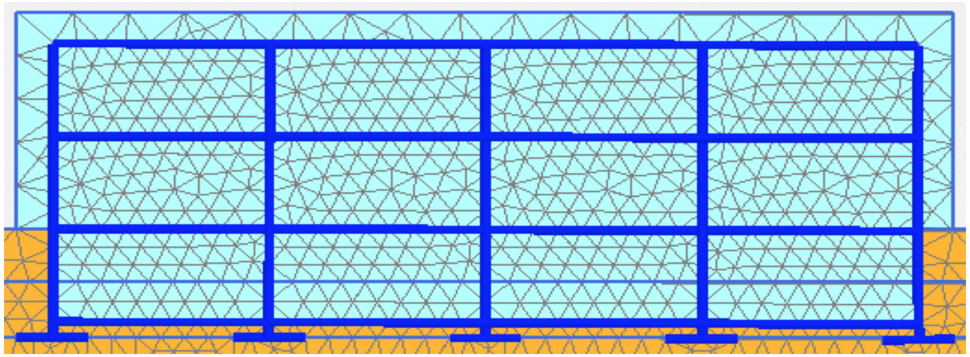
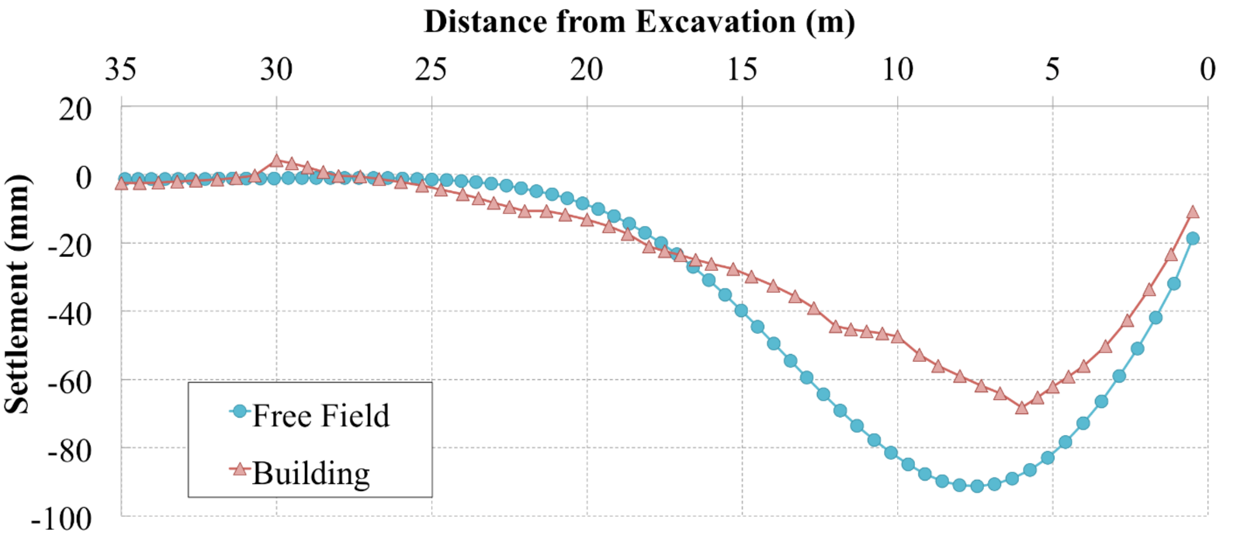
Estimate lateral movements in clays – semi-empirical (Clough et al. 1989)

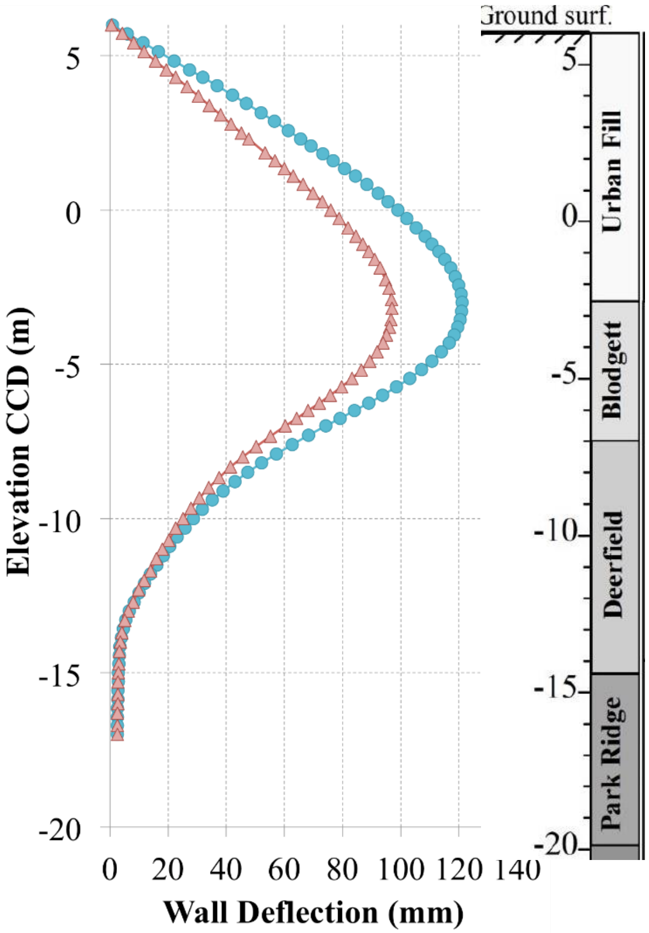
Free field movements



Presence of building adjacent to excavation affects movements







25% reduction of maximum free field settlement

two factors: lower stress from basement stiffness of building

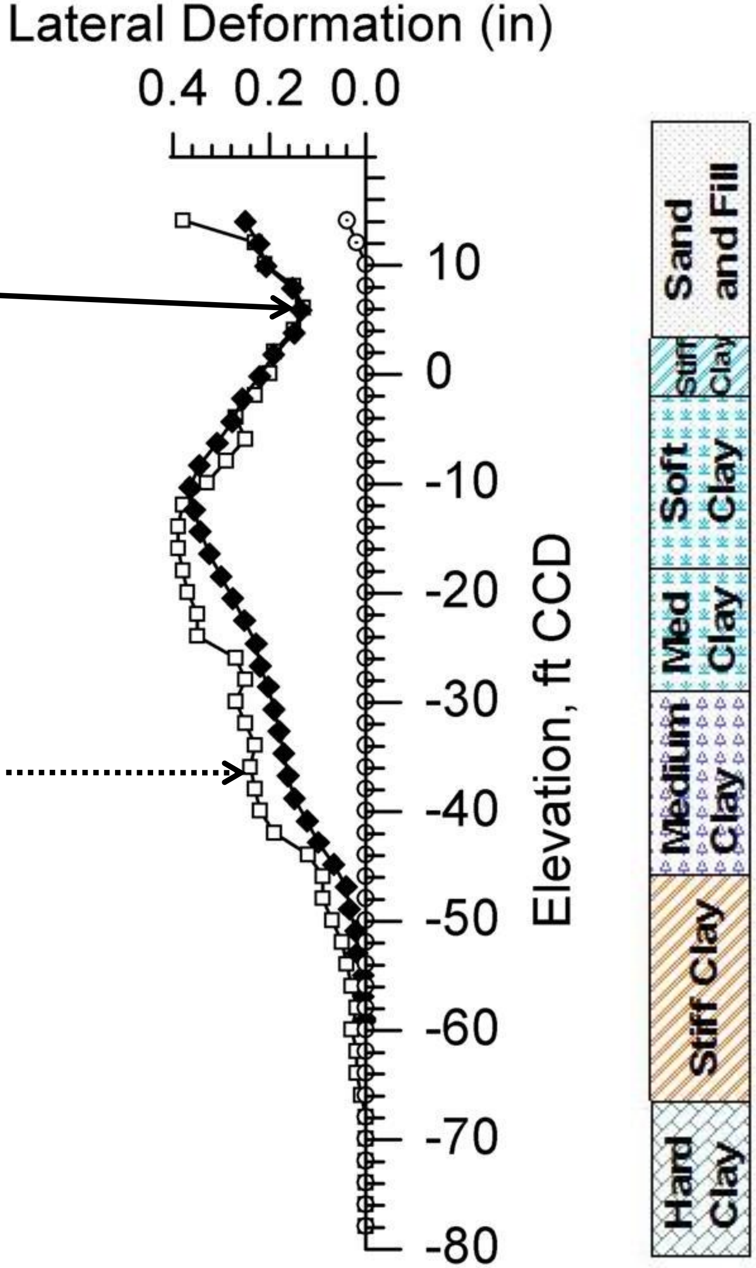
Empirical methods mostly developed by 1990

- 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

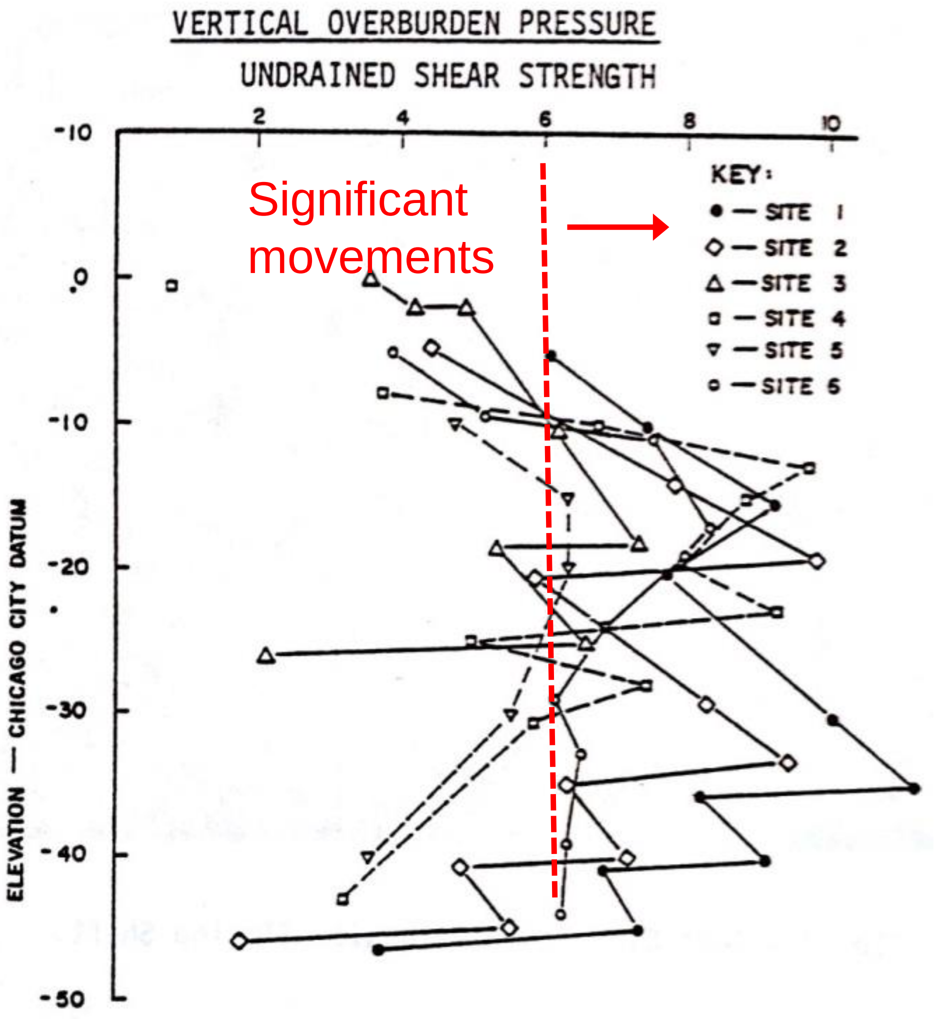
- Removal of existing foundations
- Wall installation
  - Densification of sands from vibrations
  - Displacements arising during installation
    - Slurry or secant pile wall
    - Sheet-pile wall
- Deep foundation installation
- Concrete shrinkage during top-down construction

Secant pile wall installation



Drilled shaft installation

Baker and Lukas (1978)



Significant movements

Baker and Gill (1985)

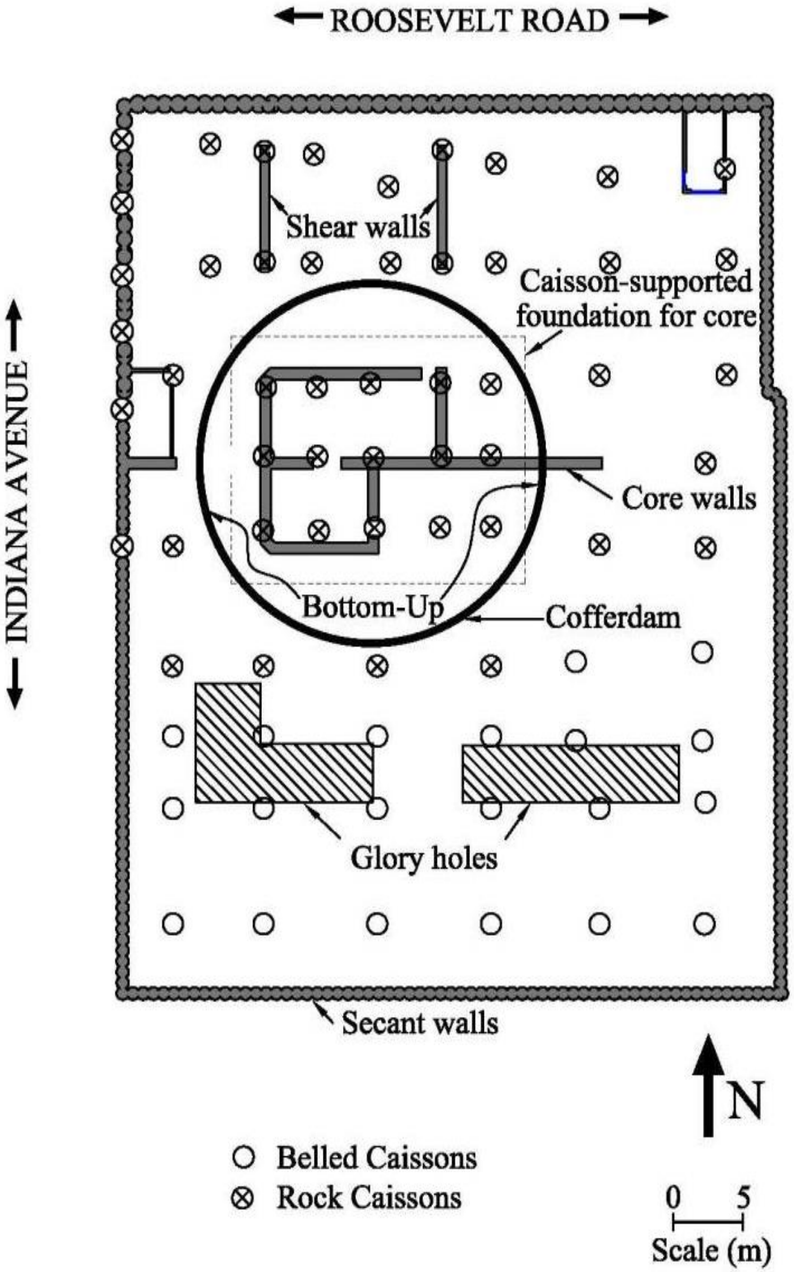
One Museum Park West project

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

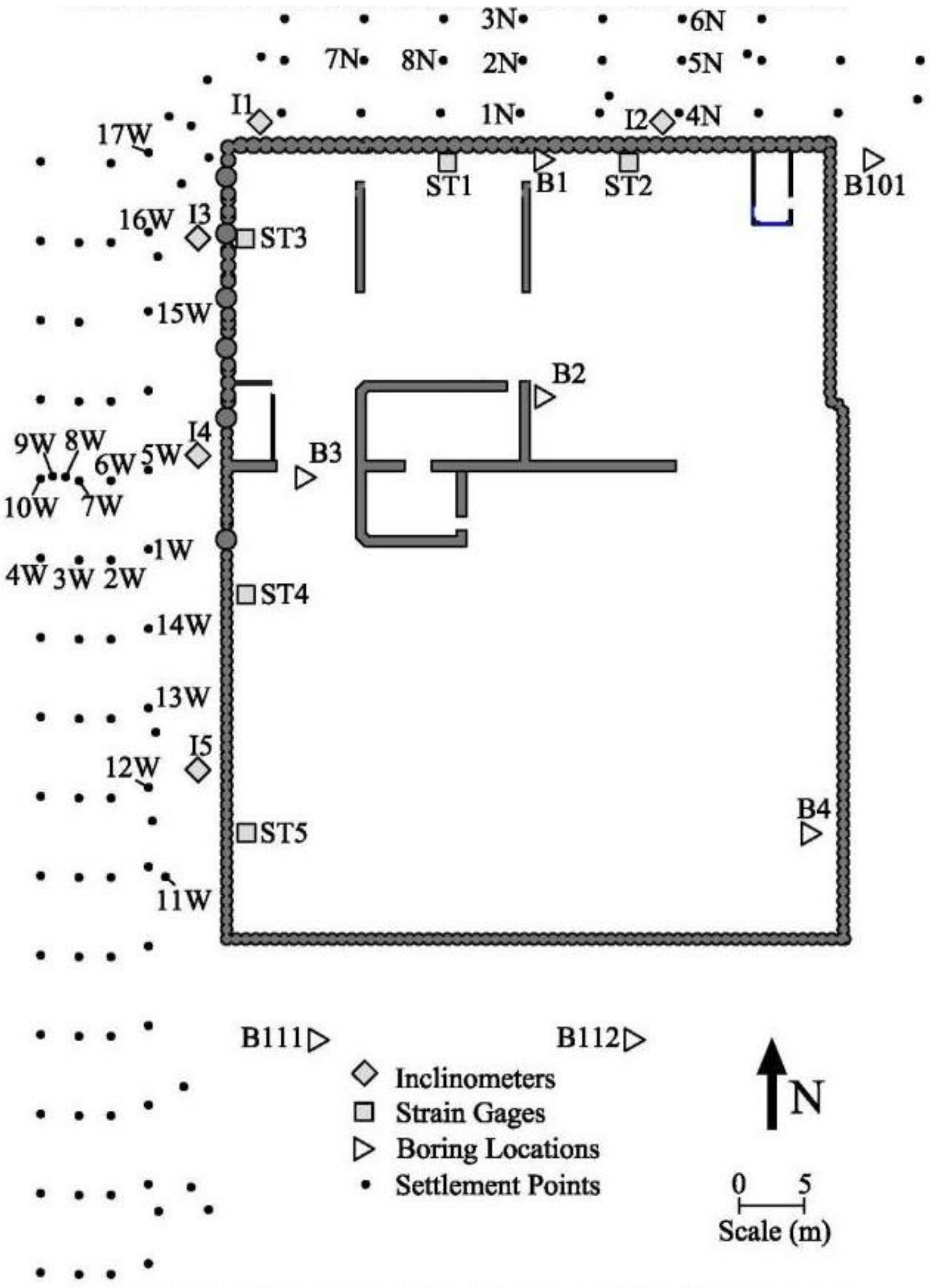
"Nominally" top down construction

1. Excavation removed from critical path
2. Overexcavation is prevented
3. Relatively high stiffness
4. Temporary support system is also permanent

One Museum Park West Excavation



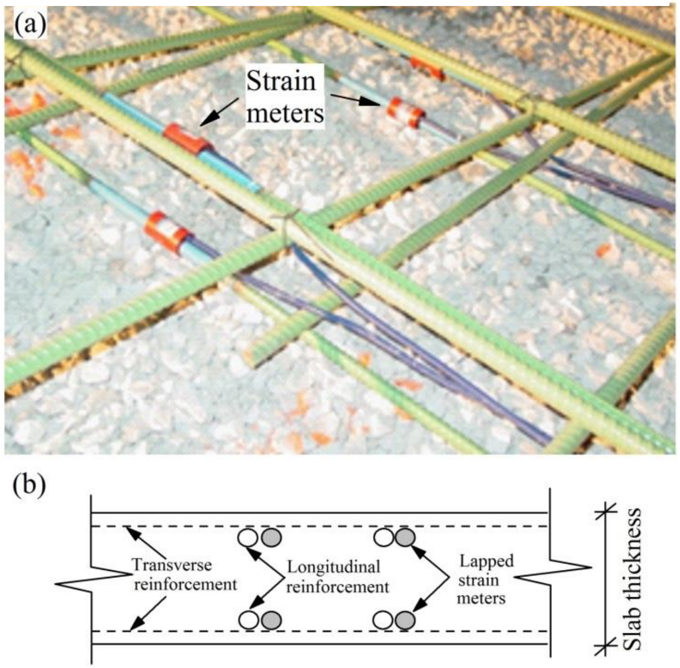
Stage	Activity	Description
1	Perimeter pile wall and foundation installation	Level site
		Install perimeter pile walls
		Install caissons
2	Central core construction (Bottom-up)	Install sheet pile wall
		Cycles of excavation and bracing
		Place reinforced concrete mat
		Construct core
3	Basement construction	Top-down construction



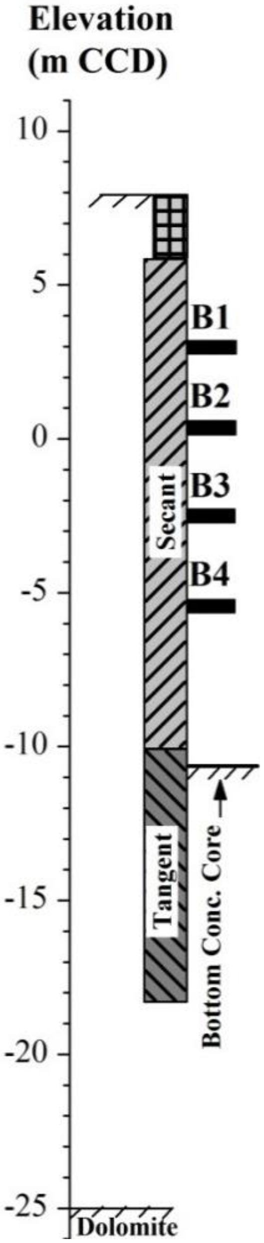
- 5 inclinometers

- 101 settlement points

- 5 Strain gage stations: total of 72 strain gages



Perimeter wall and drilled shaft construction

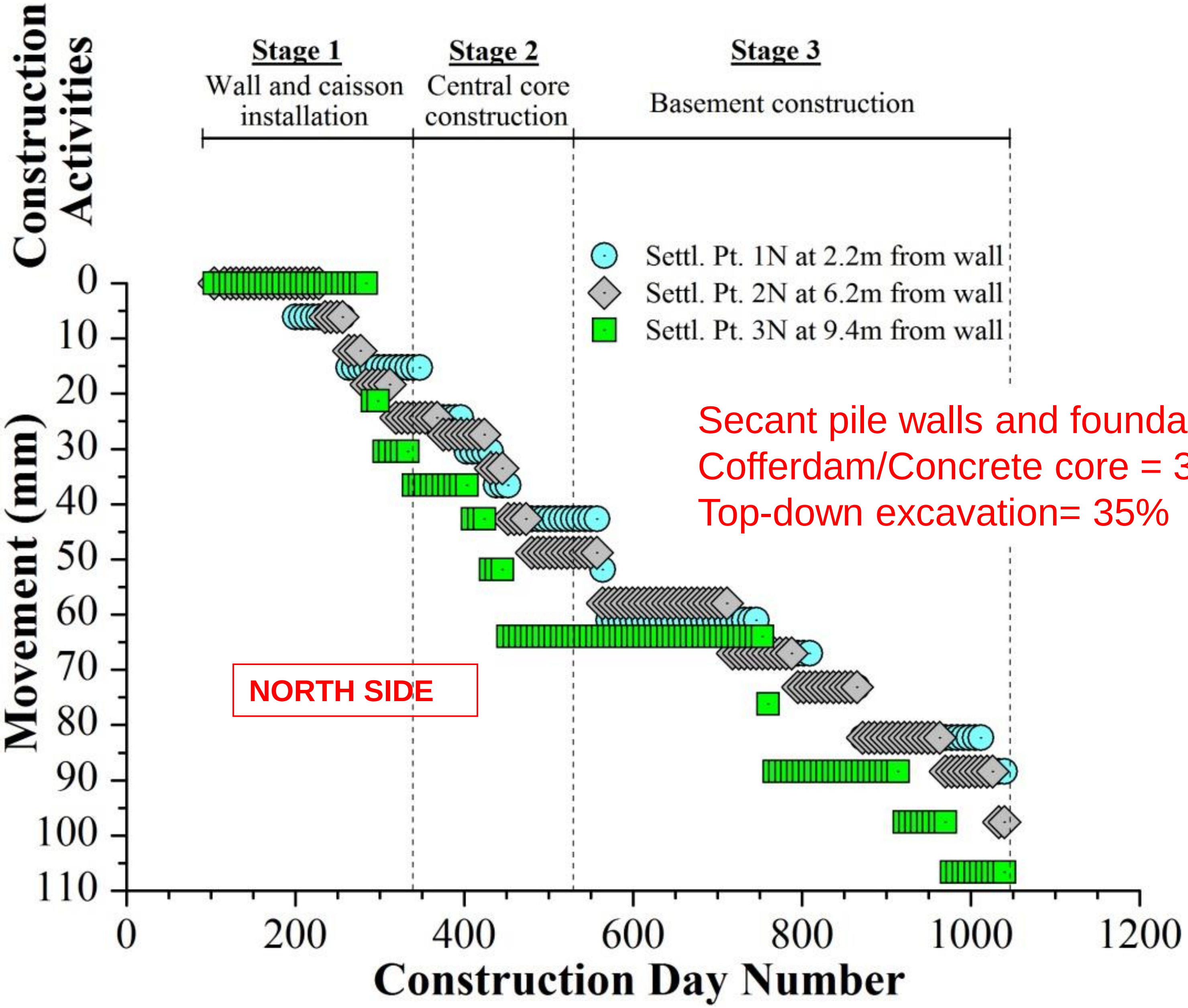


Fully cased until tangent section reached

Central core construction

Top down construction

Summary of observed settlements

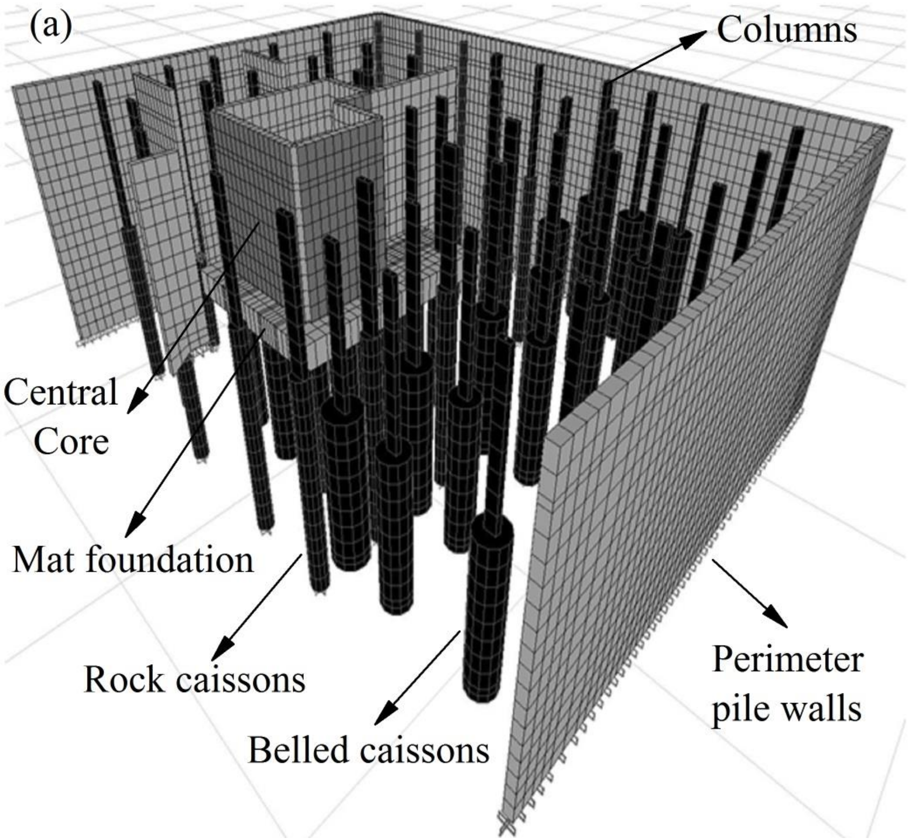


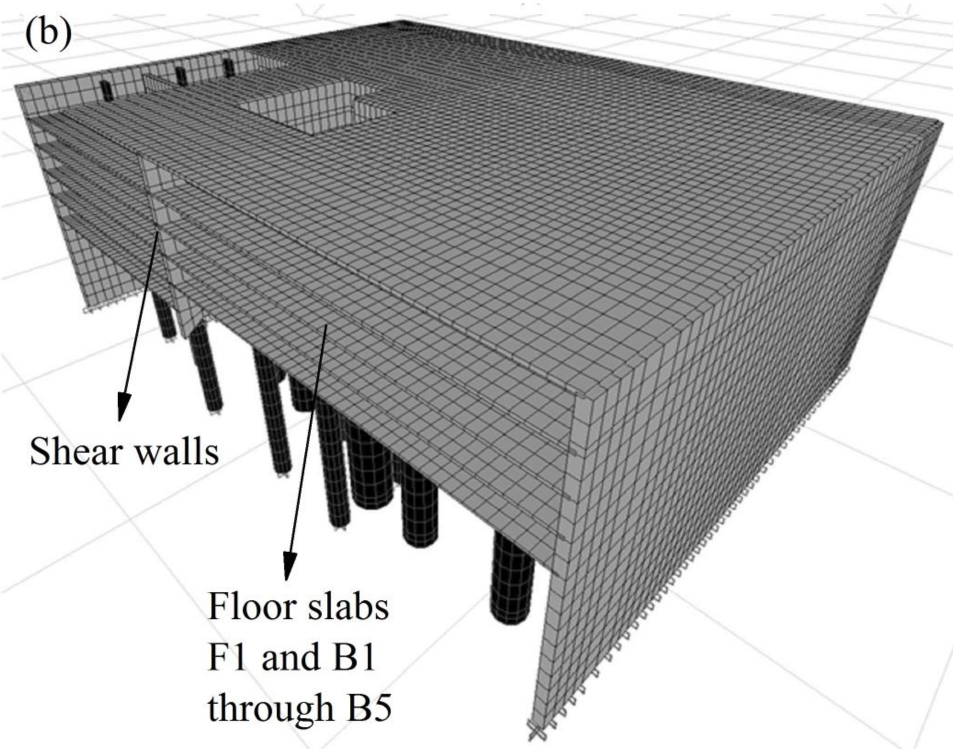
Secant pile walls and foundations = 35%
Cofferdam/Concrete core = 30%
Top-down excavation= 35%

NORTH SIDE

Concrete material time-dependence at One Museum Park West project

To quantify: FE analysis of below grade structural components

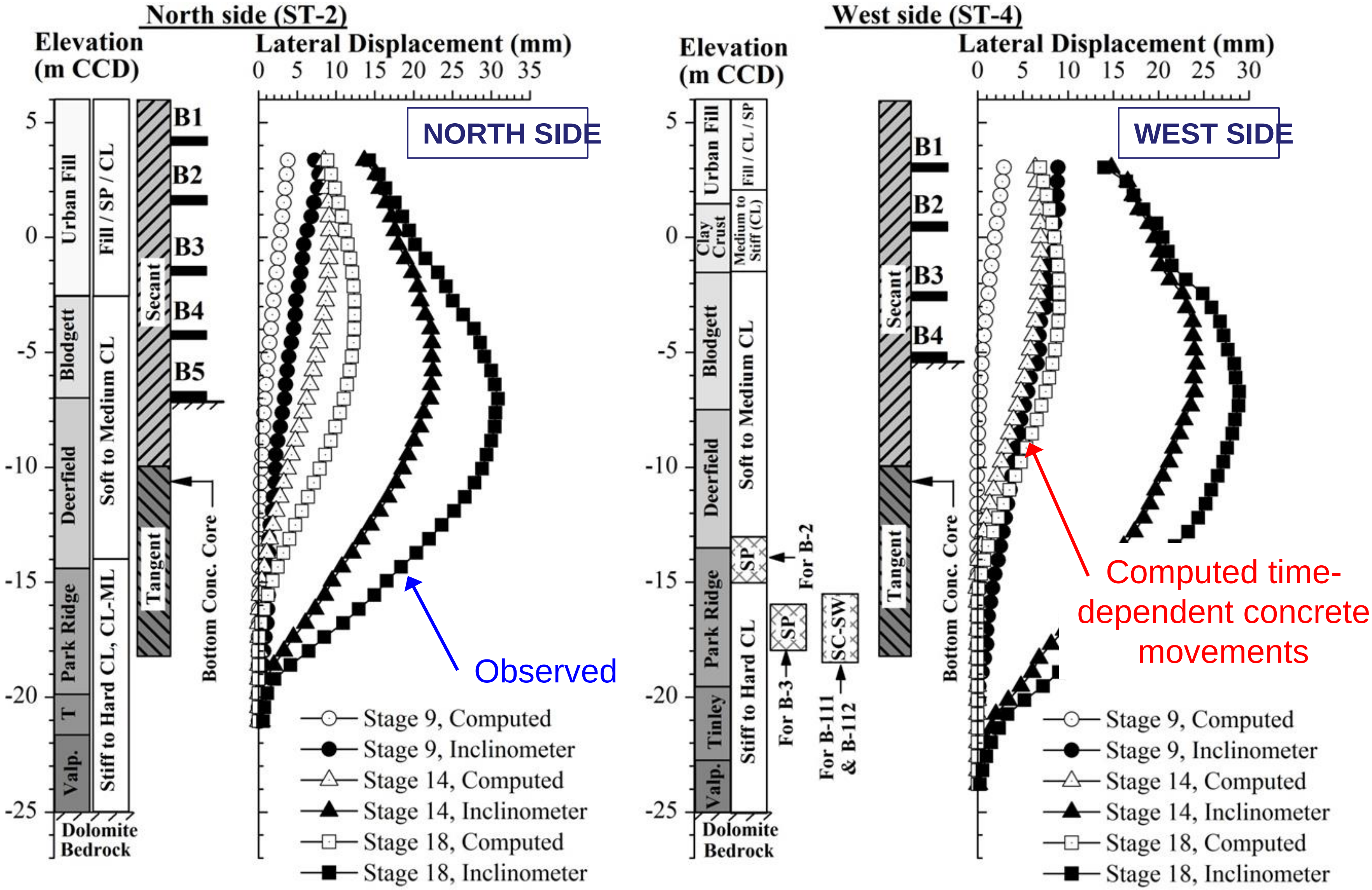




Creep

Aging

Shrinkage



NORTH SIDE

Observed

WEST SIDE

Computed time-dependent concrete movements

Concrete effects = 30% of the max. lateral displacement

After Finno et al (2015)

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

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- Hayward Baker
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- WJE & Associates
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