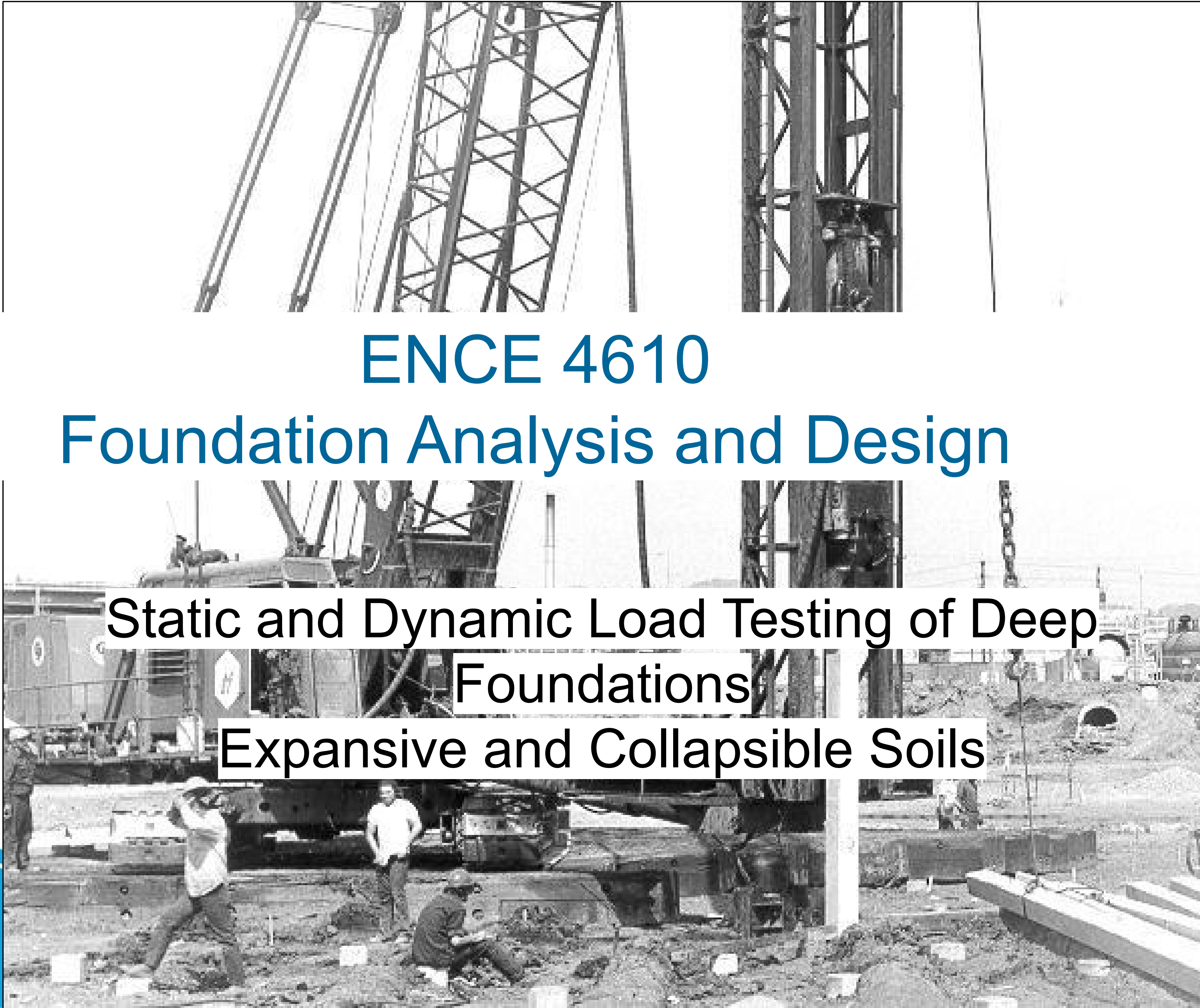
ENCE 4610
Foundation Analysis and Design



Static and Dynamic Load Testing of Deep Foundations
Expansive and Collapsible Soils

Overview of the Section

- Static and Dynamic Testing
  - Overview of the Basics of Design of Deep Foundations
  - Static Load Testing of Deep Foundations
  - Dynamic Load Testing of Deep Foundations
- Expansive and Collapsible Soils
  - Expansive Soils; Swell Potential
  - Collapsible Soils

Credits for Slides and Graphics

Some of the slides and graphics for this lecture courtesy of

GRL

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Pile Dynamics, Inc.
Cleveland, Ohio

Design and Selection of Deep Foundations

Design Parameters

- Service loads and allowable deformations
- Subsurface investigations
- Type of Foundation
- Lateral and Axial Load Capacity
- Driveability and Ability to Install
- Structural Design
- Seismic Design
- Verification and Redesign
- Integrity Testing

Selection and Types

- Type of Foundation is influenced by:
  - Design Loads
  - Subsurface Conditions
  - Constructibility
  - Reliability
  - Cost
  - Availability of materials, equipment and expertise
  - Local experience and precedent

Design of Deep Foundations

- Subsurface Investigations
  - For deep foundations, the subsurface investigations are generally more intensive and expensive
  - Exploratory borings must extend well below the toe elevation, which may have to be altered after initial calculations
  - Investigations assist in determining the type of foundation that should be used
  - Use data from deep foundations that have already been installed in the vicinity (capacity and drivability)
- Service Loads and Allowable Deformations
  - Most important step
  - Loads can include
    - Axial loads (live or dead, tensile or compressive)
    - Lateral loads (live or dead)
    - Moment loads
  - Should be combined both as unfactored loads (ASD) or factored loads (LRFD)
  - Allowable settlements should be determined by structural engineer

Lateral and Axial Load Capacity

- Axial loads
  - Analytic methods should be performed first, although their limitations should be understood
  - Analysis should include considerations of allowable load, allowable settlement, and group effects (including block failure)
  - Ideally, a static load test on test piles should be done; however, if economic or other factors make this impractical, other methods such as Osterberg Tests, CAPWAP, or Statnamic should be considered
  - Blow count specifications using a wave equation analysis are acceptable on smaller projects
- Lateral Loads
  - Frequently dictate the diameter (section modulus) of the foundation, so should be considered first, if present
  - p-y curves are the best way of determining lateral capacity
  - Large diameter piles may also be necessary for scour resistance
  - Batter piles are also a typical way of dealing with lateral loads, but can pose rigidity problems in seismic situations
  - Seismic retrofits are a common application of laterally loaded piles

Driveability

- Piles that cannot be driven cannot be used to support loads; thus, drivability is an important consideration with driven piles
- Wave equation methods are ideal to use for drivability studies
- Dynamic formulae should be avoided except for the simplest projects

●Changes in Soil During Driven Pile Installation

- Driven Piles
  - Interact with the soils differently than drilled piles (drilled shafts, auger cast piles, etc.)
  - This leads to different load transfer characteristics; the effect of driving must be understood
  - Methods are somewhat different, and have been considered separately
- Sands
  - Sands do not experiences the kinds of changes that clays do with driven piles
  - Sands (especially loose ones) experience localised compaction during driving; this increases the capacity
- Clays
  - Distortion – the movement of soil out of the way during driving remoulds the clay and reduces its strength
  - Compression and Excess Pore Water Pressures – these are raised temporarily during driving and thus decrease the driving resistance and the load bearing capacity immediately after driving
  - Loss of contact between pile and soils – also weaken the soil-pile bond along the shaft

Structural and Seismic Design

- Seismic Design
  - In seismically active areas, earthquake occurrence must be taken into consideration
  - Liquefaction of soils is a major cause of failure during seismic events
    - Deep foundations can be installed beyond liquefying soils in some cases
    - Soil improvement is necessary in others
  - Most direct seismic loads on foundations are lateral ones so lateral load analysis is important for seismic resistance
  - Batter piles tend to be too rigid for seismic loads
- Structural Design
  - Once the geotechnical analysis is done, the structural engineer can proceed with the structural design
  - Structural design involves both the structural capacity of the deep foundations themselves and the pile caps and connecting members that are above these members
  - Plans and specifications presented for bid should be complete and unambiguous

Other Considerations

Scour and Downdrag

- Scour
  - Scour is an important consideration for bridges over rivers or other bodies of water with consistent currents
  - Loss of supporting soil at the head of the deep foundation must be considered when designing for scour
- Downdrag
  - Compression of soil by overburden around piles creates downdrag on piles
  - This is dealt with either by design or by treatment of the surface of the pile

Verification and Redesign

- Redesign during construction is to be avoided but is sometimes necessary due to differences in the subsurface conditions between what was estimated and what actually takes place
- Dynamic testing (Case Method, CAPWAP) is a useful tool to evaluate the need for redesign but must be used with good judgement
- Drilled shafts and the cuttings from boring should be inspected before and during the placement of concrete

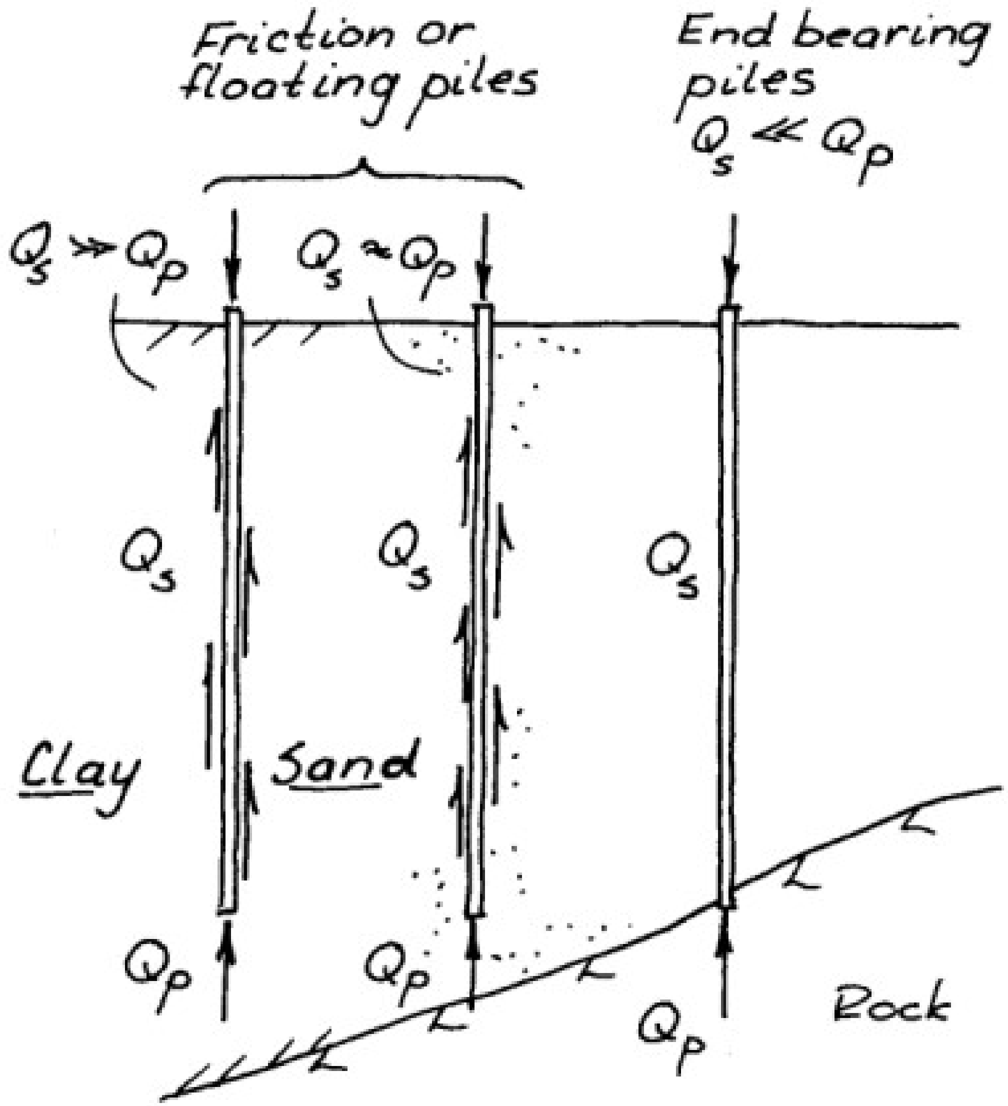
Static Load Testing of Deep Foundations

Evaluation Method for Deep Foundations

- Analytic methods, based on soil properties from laboratory and/or in-situ tests
- Load Testing
  - Full-scale static load tests on test piles
  - Dynamic Methods, based on dynamics of pile driving or wave propagation

Concepts to Review

- Shaft and End Bearing Piles
  - Resistance to load
    - Shaft resistance (Q_s)
    - Toe resistance (Q_t)
  - End-bearing piles, toe resistance predominates
  - Shaft Friction Piles, shaft resistance predominates
  - For tension piles, shaft resistance predominates (toe cannot be in tension)
- Ultimate vs. Allowable Capacity
  - Ultimate capacity is the load required to cause failure, whether by excessive settlement or irreversible movement of the pile relative to the soil
    - For driven piles, one must also consider the resistance to driving, which can be different from the ultimate capacity
  - Allowable capacity is the ultimate capacity divided by a factor of safety (ASD)
  - Important to distinguish between the two



Prediction and Verification

- At what load will the pile fail? (Bearing Capacity)
- How much will pile deflect under service loads? (Settlement)
- The two concepts are not really separated except by custom and code

***Prediction* on basis of site investigation, laboratory testing and analytic modeling**

***Verification* by some method of load testing**

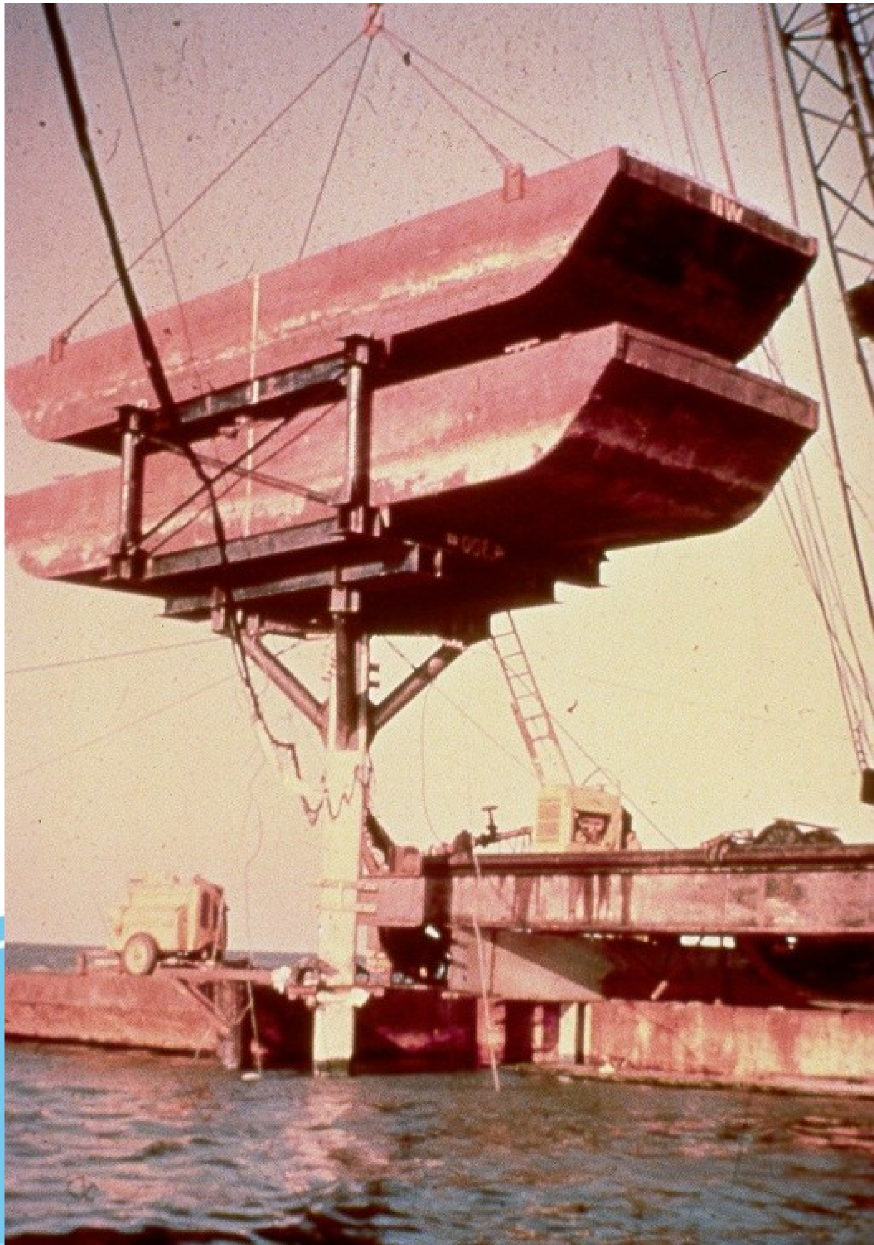
Static Load Tests

Figure 9-65. Load test movement monitoring components (FHWA, 2006a).

- The most precise – if not always the most accurate – method of determining the ultimate upward or downward load capacity of a deep foundation
- Static load tests, however, are time consuming and expensive; must be used judiciously
- Object of the test is to develop a load-displacement curve, from which the load capacity can be determined

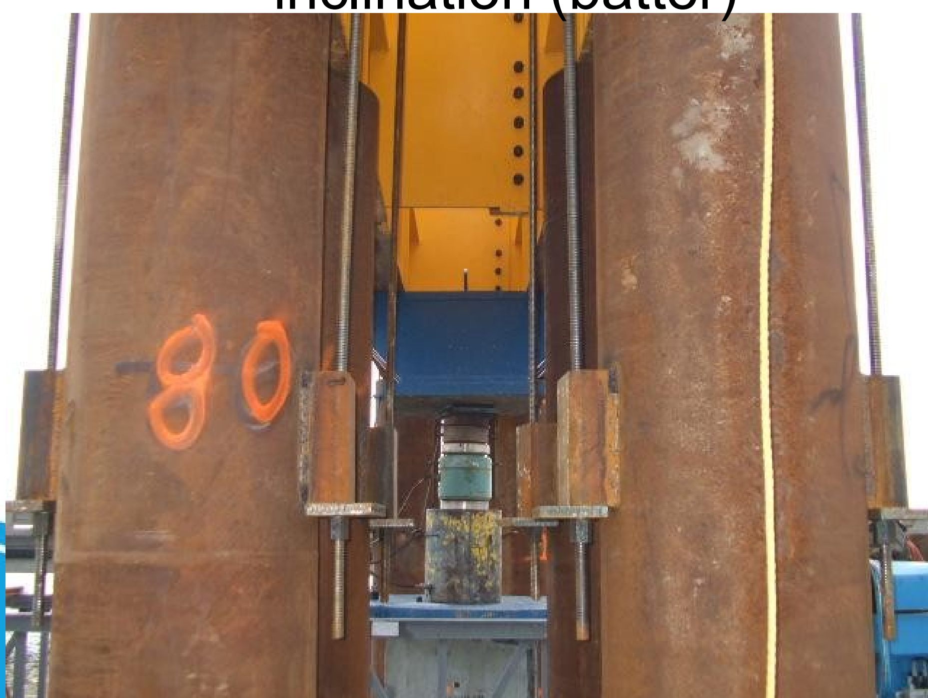
Dead Load Test

- Considered a reliable static test method
- Slow and expensive
- Dangerous when done unsafely
- No longer commonly used in the US; used where labor costs are lower



Reaction Pile Systems

- Advantages
  - Can be installed with same equipment as production piles
  - Test can be done on inclination (batter)
- Disadvantages
  - Reaction piles may pull out
  - If not done properly, reaction pile capacity may result
  - Flexible system stores energy during tests



Static Load Procedure (ASTM D1143)

- Procedure
  - Set up reaction stand
  - Apply a test load to the pile
  - Record the load-settlement history for each load applied
  - Apply the next load
  - Loads are generally applied in increments of 25, 50, 75, 100, 125, 150, 175 and 200% of proposed design load
- Two categories of tests
  - Controlled stress tests – the most common method
  - Controlled strain tests
- For driven piles, need to delay static load test to allow pile set-up
  - Granular soils – 2 days
  - Cohesive soils – 30 days

Static Load Procedure (ASTM D1143)

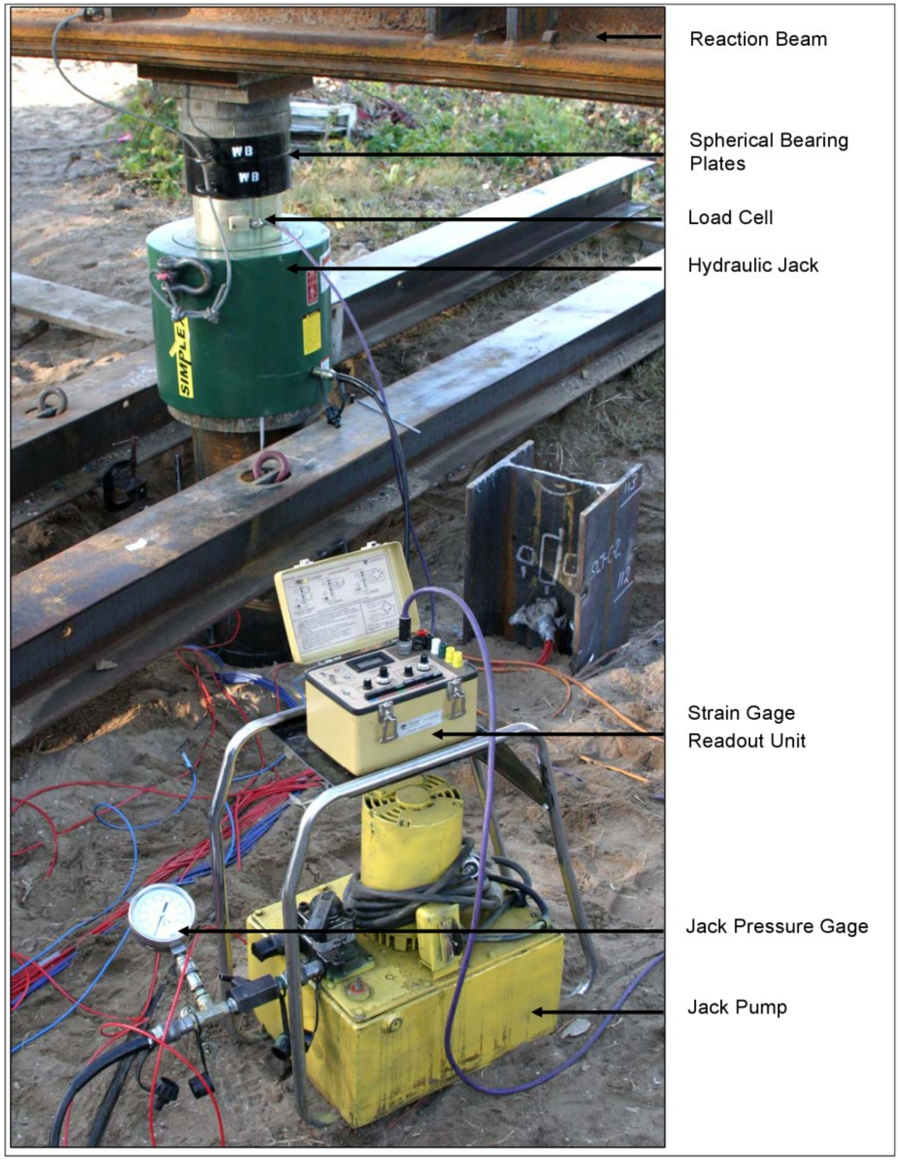


Figure 9-64. Load test load application and monitoring components (FHWA, 2006a).

- Load test increments in time
  - Slow test – maintains load until pile movement is sufficiently small
  - Quick test – each load increment is held for a predetermined length of time, for 2.5 – 15 minutes
    - Generally requires 2-5 hours to complete
    - May be best method for most deep foundations

Static Load Tests: Advantages and Disadvantages

- Advantages
  - Gives reference capacity
  - Relatively slow loading minimises dynamic components
  - Can customise to include creep effects
  - Can be instrumented to yield static resistance distribution & end bearing
- Disadvantages
  - Time consuming
  - Expensive
  - Done on specially designated piles
  - Often done carelessly or inaccurately

Use of Telltales

5. The movement is usually measured only at the pile head. However, the pile can be instrumented to determine movement anywhere along the pile. Telltales (solid rods protected by tubes) shown in Figure 9-62 or strain gages may be used to obtain this information.

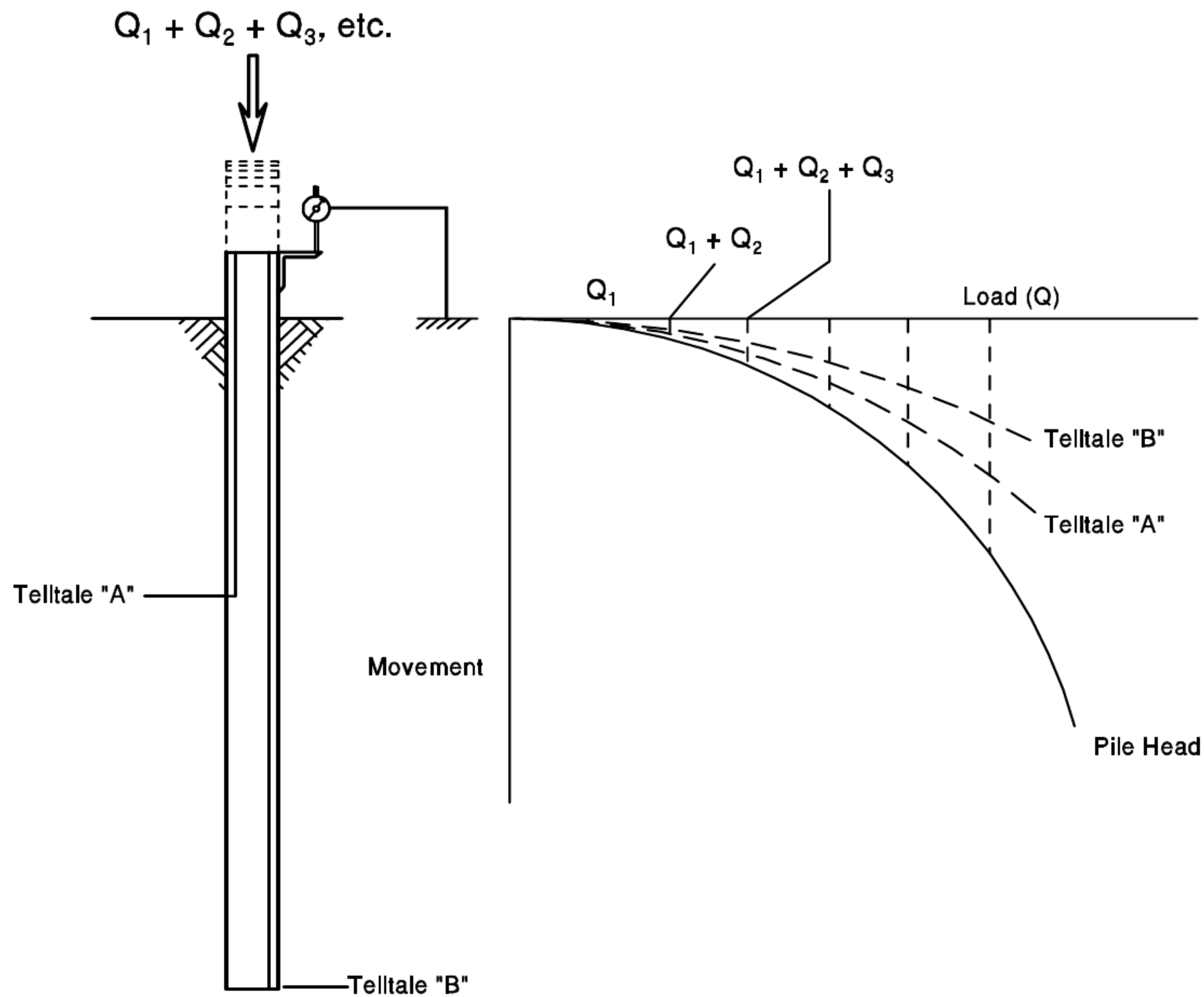


Figure 9-62. Basic mechanism of a compression pile load test (FHWA, 2006a).

Static Load Test Interpretation

At what point of the measured pile top load vs deflection curve do we define the failure load R_u?

Loading method and curve interpretation can make significant differences in result.

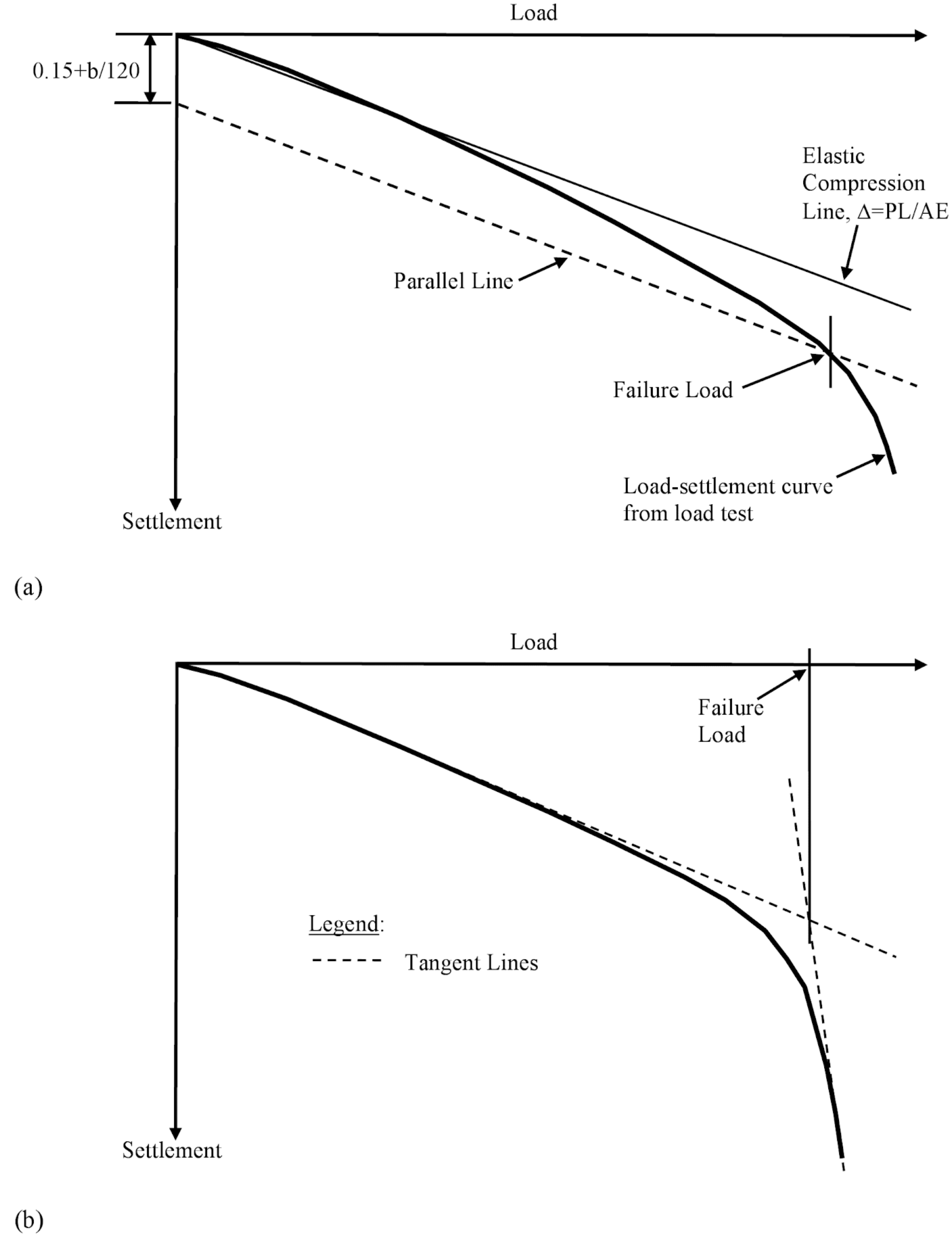


Figure 9-68. Presentation of typical static pile load-movement results, (a) Davisson's method, (b) Double-tangent method.

● Davisson's Method

9.15.7.3 Presentation and Interpretation of Compression Test Results

The results of load tests should be presented in a report conforming to the requirements of ASTM D 1143. A load-movement curve similar to the one shown in Figure 9-68 should be plotted for interpretation of test results.

The literature abounds with different methods of defining the failure load from static load tests. Methods of interpretation based on maximum allowable gross movements, which do not take into account the elastic deformation of the pile shaft, are not recommended. These methods overestimate the allowable capacities of short piles and underestimate the allowable capacities of long piles. Methods that account for elastic deformation and are based on a specified failure criterion provide a better understanding of pile performance and provide more accurate results.

AASHTO (2002) and FHWA (1992c) recommend pile compression test results be evaluated by using an offset limit method as proposed by Davisson (1972). The "double-tangent" is more commonly used for drilled shafts. These methods are shown in Figure 9-68 and are discussed in the following sections.

9.15.7.4 Plotting the Failure Criteria

Figure 9-68a shows the load-movement curve from a typical pile load test. To facilitate the interpretation of the test results, the scales for the loads and movements are selected so that the line representing the elastic deformation Δ of the pile is inclined at an angle of about 20° from the load axis. The elastic deformation Δ is computed from:

$$\Delta = \frac{QL}{AE} \qquad \text{9-51}$$

Where:
- Δ = elastic deformation in inches (mm)
- Q = test load in kips (kN)
- L = pile length in inches (mm)
- A = cross sectional area of the pile in in^2 (m^2)
- E = modulus of elasticity of the pile material in ksi (kPa)

9.15.7.5 Determination of the Ultimate (Failure) Load

For pile diameters less than 24 in (610 mm), the ultimate or failure load Q_f of a pile is that load which produces a movement of the pile head equal to:

In US Units

$$s_f = \Delta + \left(0.15 + \frac{b}{120}\right) \qquad \text{9-52}$$

where:
- s_f = settlement at failure in inches
- b = pile diameter or width in inches
- Δ = elastic deformation of total pile length in inches

A failure criterion line parallel to the elastic deformation line is plotted as shown in Figure 9-68a. The point at which the observed load-movement curve intersects the failure criterion is by definition the failure load. If the load-movement curve does not intersect the failure criterion line, the pile has an ultimate capacity in excess of the maximum applied test load.

For pile diameters greater than 24 in (610 mm), additional pile toe movement is necessary to develop the toe resistance. For pile diameters greater than 24 in (610 mm), the failure load can be defined as the load that produces at movement at the pile head equal to:

In US Units

$$s_f = \Delta + \left(\frac{b}{30}\right) \qquad \text{9-53}$$

For drilled shafts, the failure load is commonly determined based on the "double-tangent" method shown in Figure 9-68b. Alternatively, the failure load is often defined as the test load corresponding to 5% of the shaft diameter because such a movement represents a large movement given that the drilled shafts are often much larger in diameter than driven piles.

Example of Davisson's Method

An axial compression static load test has been performed and the results must be interpreted to determine if the pile has an ultimate capacity in excess of the required ultimate capacity. The load - movement curve from the static load on a 356 mm square prestressed concrete pile is presented on the following page. The pile has a cross sectional area, A, of 0.127 m^2 and a length, L, of 24 m. The concrete compression strength, f'_c is 34.5 MPa. The pile has a required ultimate pile capacity of 2200 kN.

Recommended Procedure:

First determine, the elastic modulus, E, of the pile from the concrete compressive strength using $E = 4700\sqrt{f'_c}$ where f'_c must be in MPa.

$$E = 4700\sqrt{34.5} = 27606 \text{ MPa}$$

Next, calculate and plot the elastic deformation line using zero and any other load. However, for consistency between solutions and ease in plotting, calculate the elastic deformation using a load of 2500 kN from $\Delta = QL / AE$. Make sure the units for the terms in this equation are as required in the equation description provided in Section 19.7.4.

$\Delta = QL / AE = [2500 \text{ kN} (24 \text{ m})(1000 \text{ mm / m})]/[0.127 \text{ m}^2 (27606000 \text{ kPa})] = 17.1 \text{ mm}$

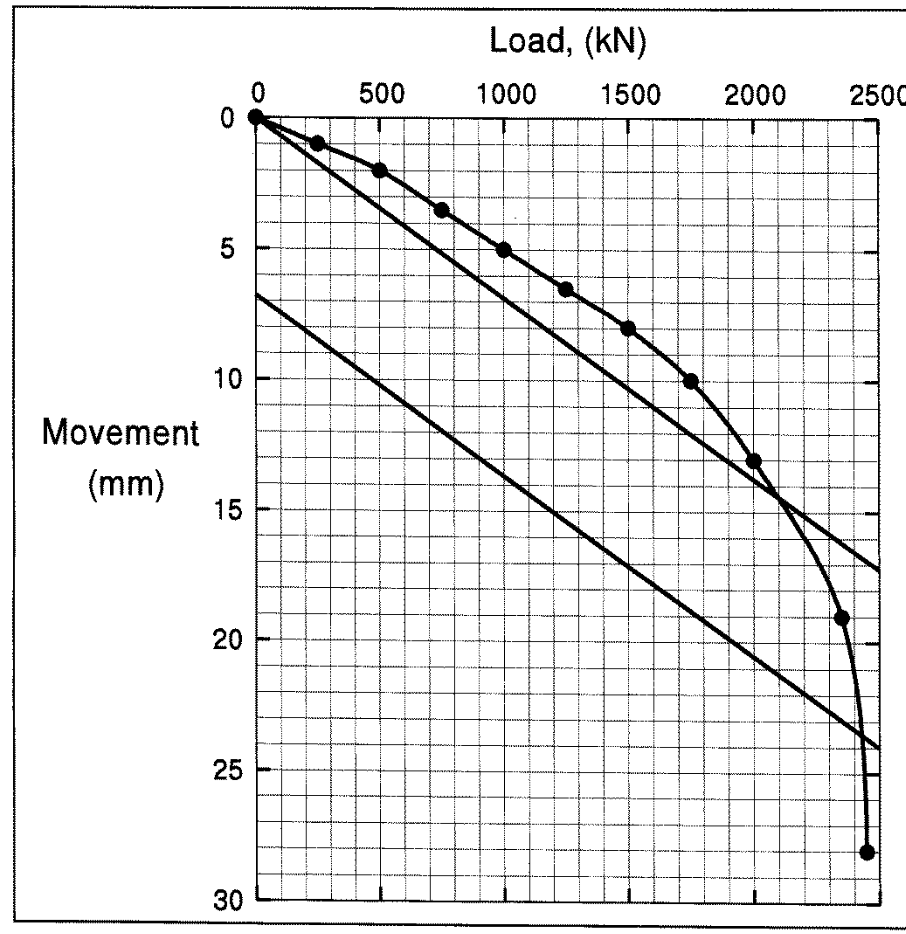
Then calculate the failure criterion line for the 356 mm pile from $s_f = \Delta + (4.0 + 0.008b)$ as described in Section 19.7.5. Remember at zero load, the failure criterion line will start at a movement equal to (4.0 + 0.008b) and at 2500 kN, the failure criterion line will be equal to a movement of $s_f = \Delta + (4.0 + 0.008b)$.

At 0 kN, $s_f = (4.0 \text{ mm} + 0.008b) = 6.8 \text{ mm}$.

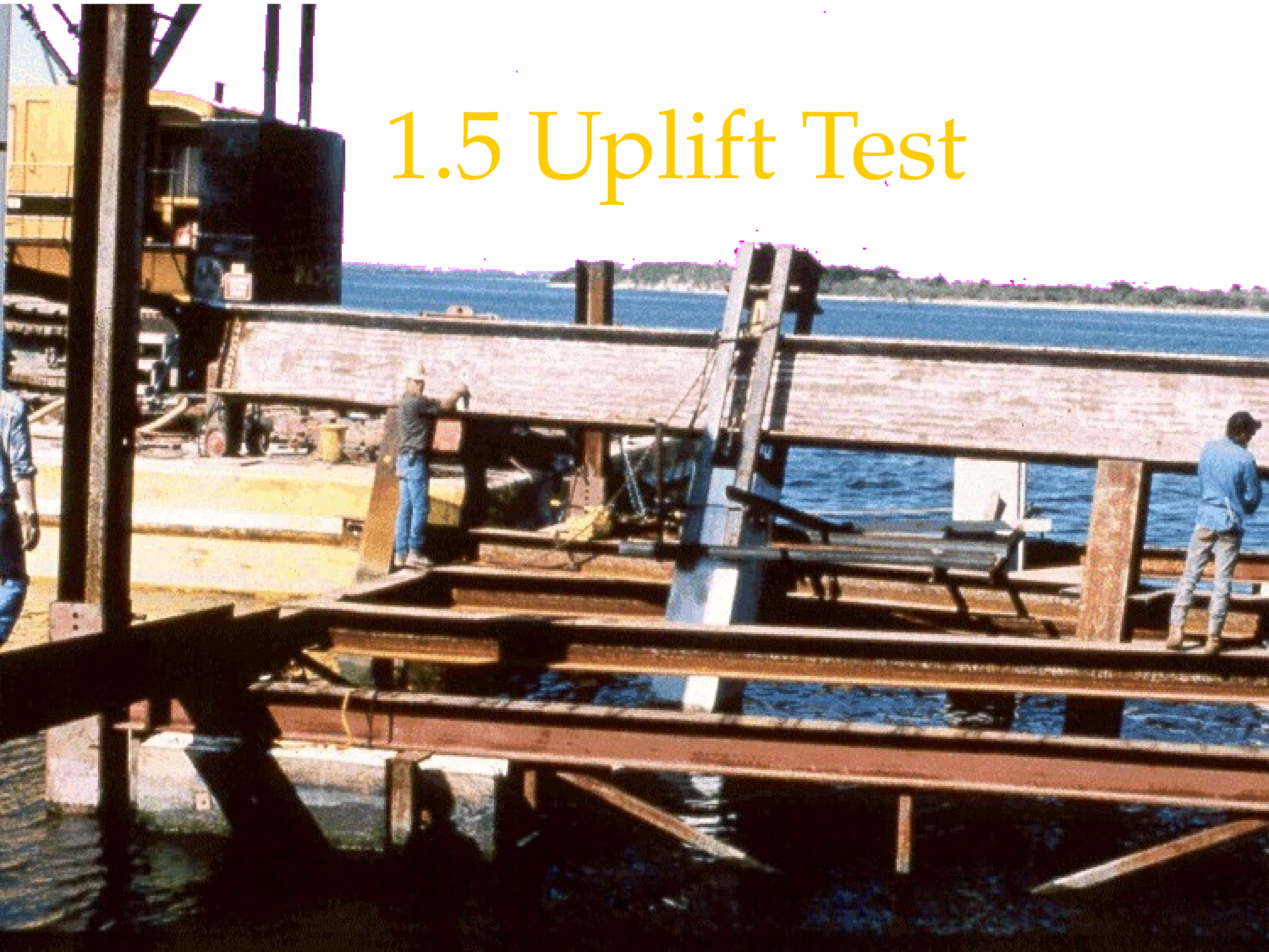
At 2500 kN, $s_f = \Delta + (4.0 \text{ mm} + 0.008b) = 17.2 + (4.0 \text{ mm} + 0.008(356)) = 23.9 \text{ mm}$

Last, plot the failure criterion line on the load-movement curve and determine whether the failure load is greater than the required ultimate pile capacity of 2200 kN.

Based on the attached plot, the failure load is 2425 kN which is greater than the required ultimate capacity of 2200 kN.



1.5 Uplift Test



Load Transfer Evaluations

9.15.7.7 Load Transfer Evaluations

FHWA (1992c) provides a method for evaluation of the soil resistance distribution from telltales embedded in a load test pile. The average load in the pile, Q_{avg}, between two measuring points can be determined as follows:

$$Q_{avg} = A\ E\ \frac{R_1 - R_2}{\Delta L} \qquad 9\text{-}54$$

Where:
- ΔL = length of pile between two measuring points under no load condition
- A = cross sectional area of the pile
- E = modulus of elasticity of the pile
- R_1 = deflection readings at upper of two measuring points
- R_2 = deflection readings at lower of two measuring points

If the R_1 and R_2 readings correspond to the pile head and the pile toe respectively, then an estimate of the shaft and toe resistances may be computed. For a pile with an assumed constant uniform soil resistance distribution, Fellenius (1990) states that an estimate of the toe resistance, R_t, can be computed from the applied pile head load, Q_h by the following equation.

$$R_t = 2\,Q_{avg} - Q_h \qquad 9\text{-}55$$

The applied pile head load, Q_h, is chosen as close to the failure load as possible. For a pile with an assumed linearly increasing triangular soil resistance distribution, the estimated toe resistance may be calculated by using the following equation:

$$R_t = 3\,Q_{avg} - 2\,Q_h \qquad 9\text{-}56$$

The estimated shaft resistance can then be calculated from the applied pile head load minus the toe resistance.

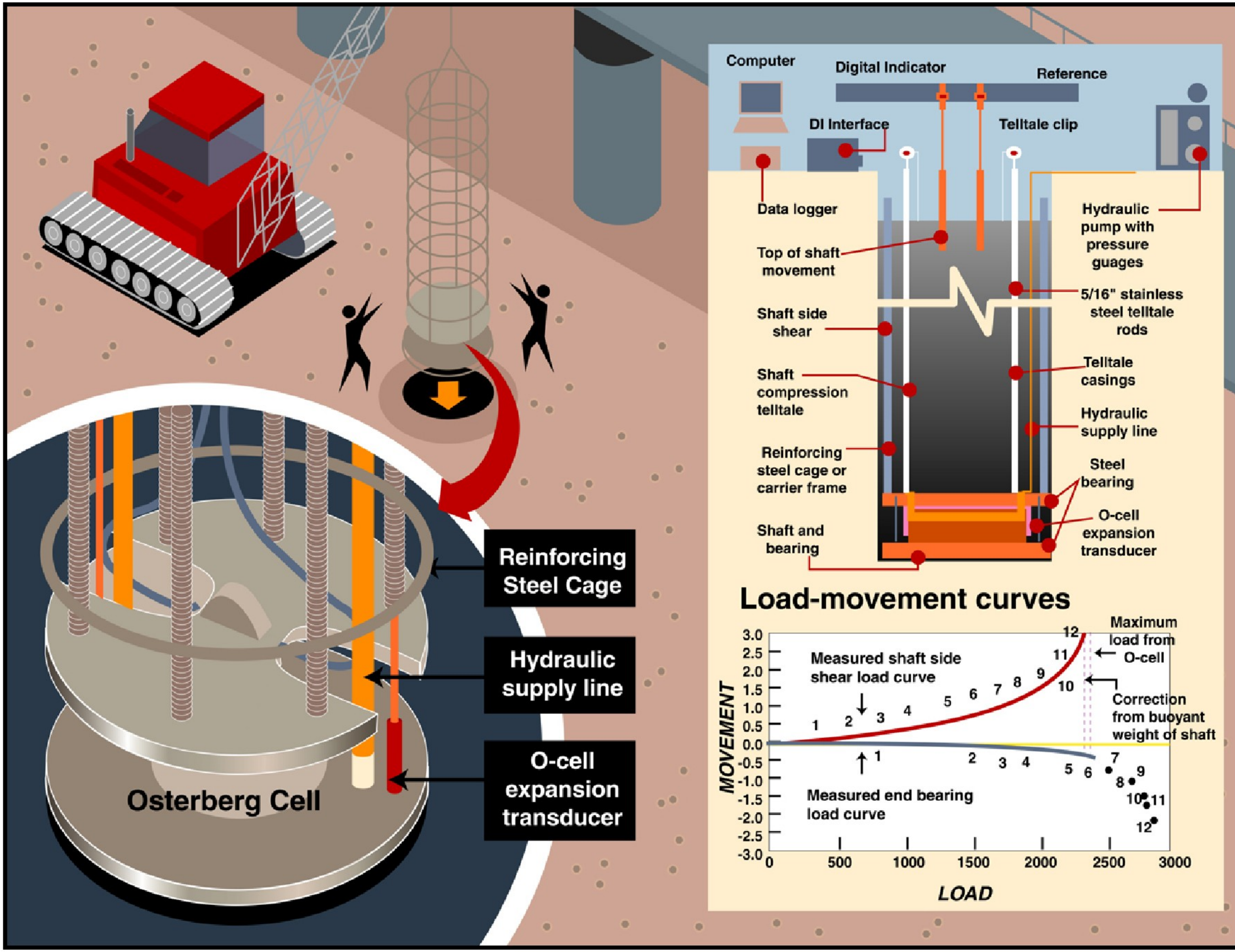
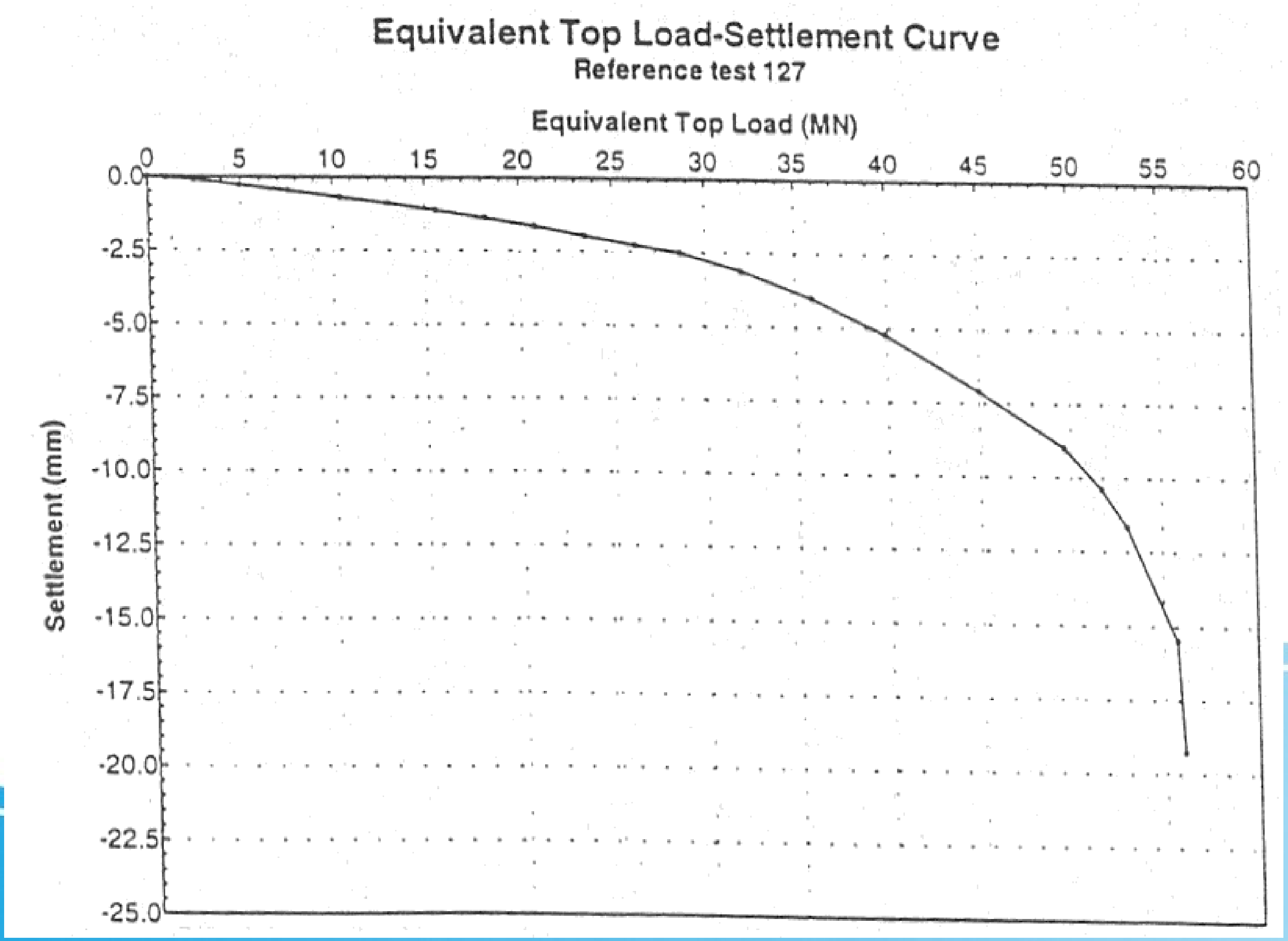


Figure 9-73. Some details of the O-Cell® test (after www.bridgebuildermagazine.com).

Equivalent Curve from Osterberg Data



Osterberg Test

Advantages and Disadvantages

- Shaft is loaded upward rather than in downward direction
- Tensile vertical strains near toe will cause cracking in soil
- Maximum movement is at the pile toe rather than pile top
- Only for specially prepared piles

- No reaction load needed
- Requires jack load only half of test load

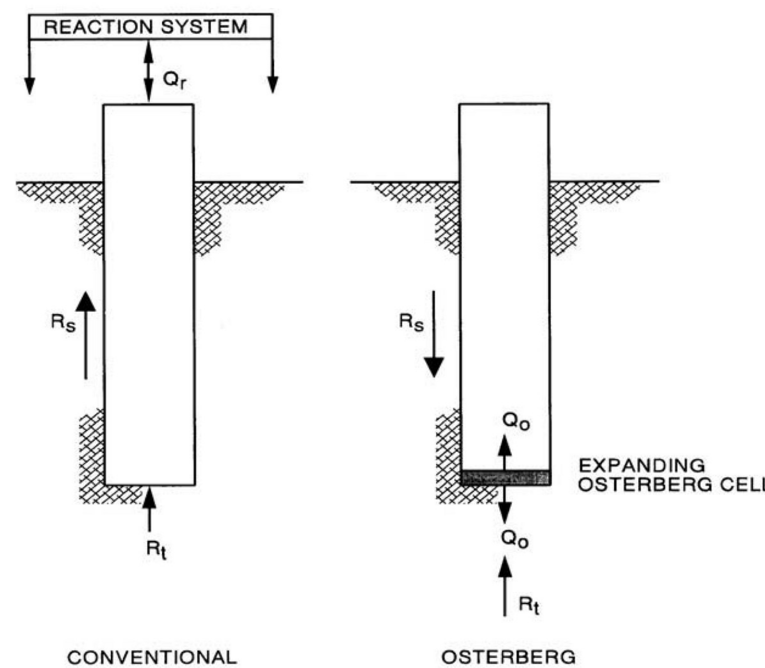


Figure 9-72. Comparison of reaction mechanism between Osterberg Cell® and Static test.

Weaknesses in Static Load Capacity Approach

- Uncertainties in soil-pile interface
- Multiple interpretations of load-deflection curve
  - Pile failure and plunging failure not the same
- Existence of downdrag in piles
- Pile capacity must, in the end, be settlement based

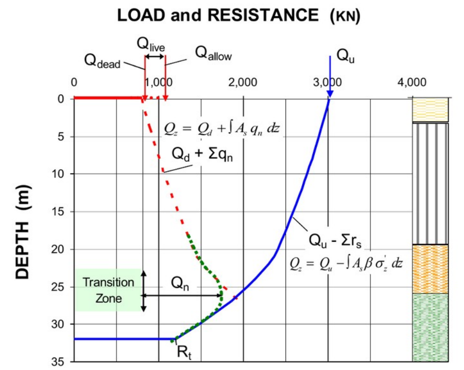


Fig. 7.6a Load-Transfer and Resistance Curves

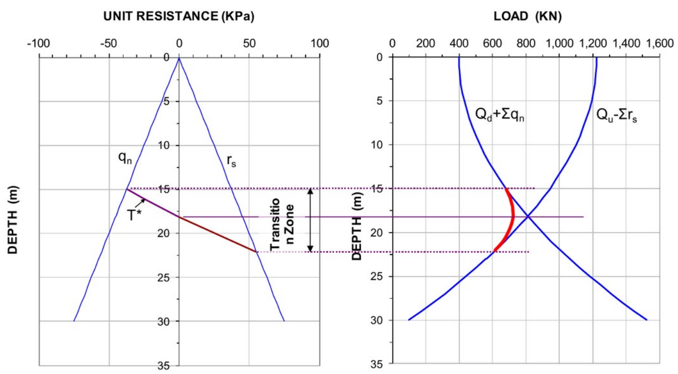
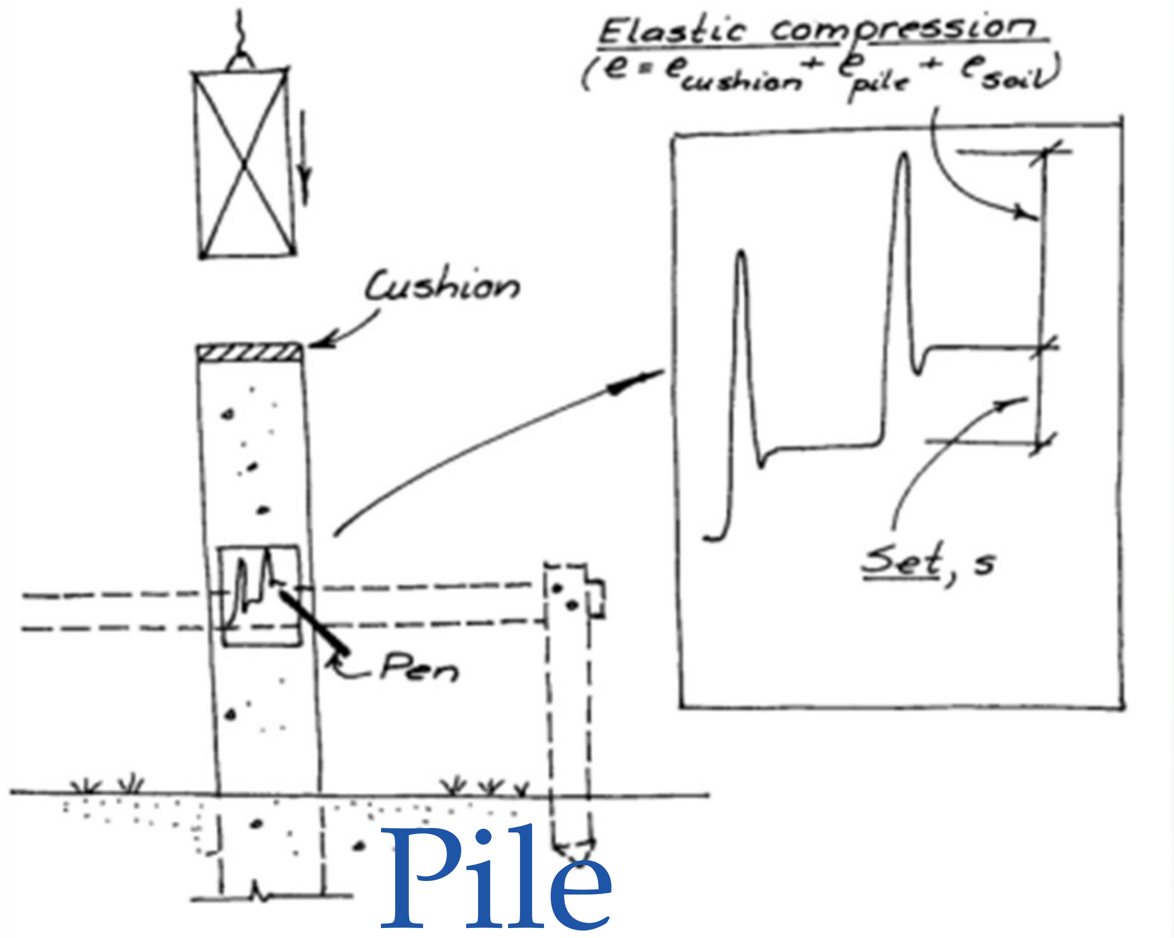


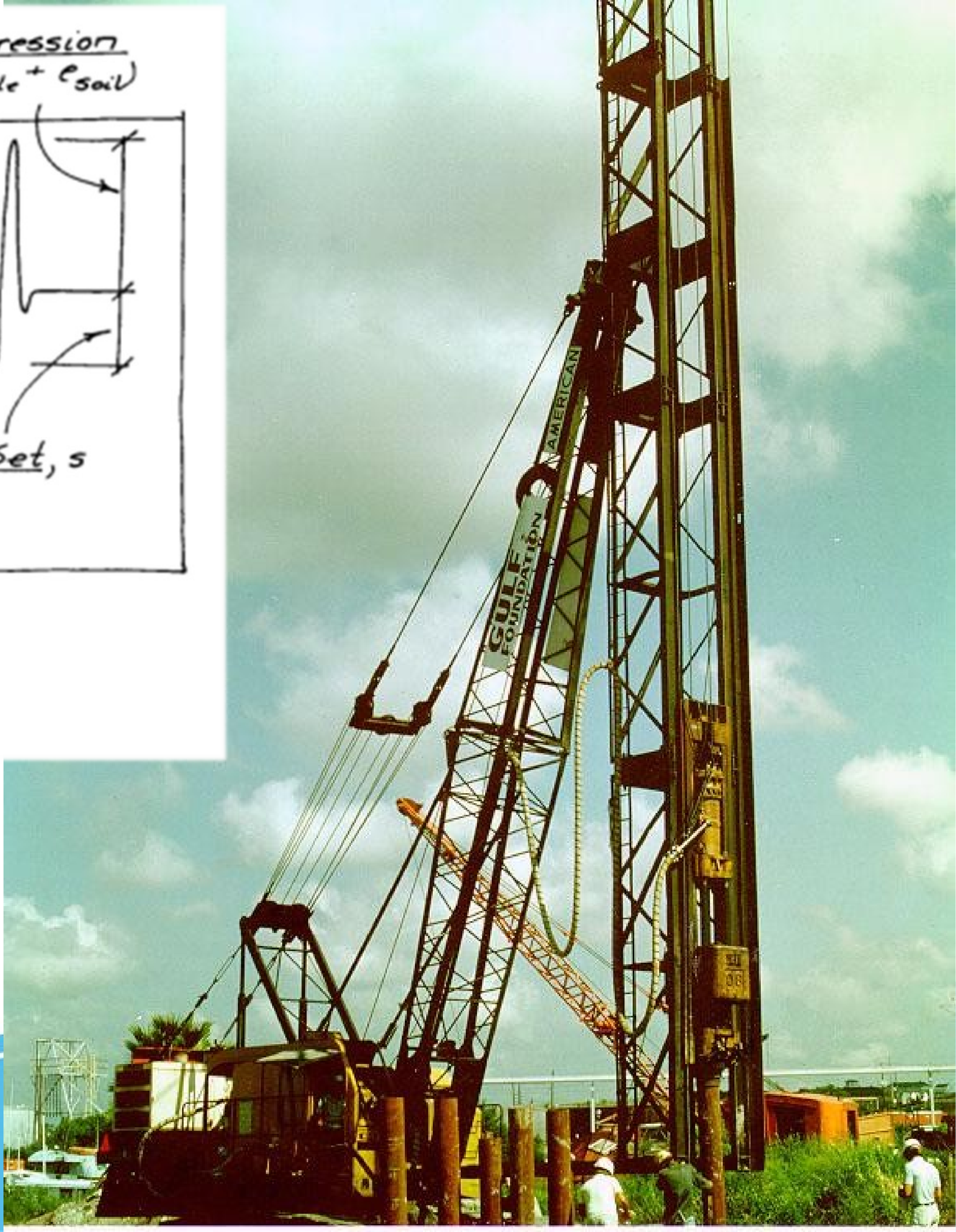
Fig. 7.6b The Principles of Transition from Unit Negative Skin Friction to Unit Positive Shaft Resistance and from Increasing Load to Decreasing Load Distribution (*not the same example as in 7.6a*)

Overview of Pile Dynamics

- Background and the Dynamic Formulae
- Development of the Wave Equation
- Application of the Wave Equation to Piles
- Use of the Wave Equation in field monitoring
- Statnamic Testing



Pile Blow Counts



Dynamic Formulae

- The original method of estimating the relationship between the blow count of the hammer and the "capacity" of the pile
- Use Newtonian impact mechanics

Mass of hammer

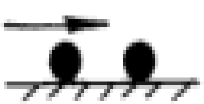
Penetration resistance of pile

$$Q_h h_e g = Rs + \underbrace{0.5Re_{soil} + 0.5Re_{cushion} + 0.5Re_{pile}}_{0.5Re}$$

Set

Effective height of fall

Elastic compression of soil, cushion and soil



$$+ \frac{Q_h h_e g Q_p (1-k^2)}{Q_h + Q_p}$$

Coefficient of restitution

Mass of hammer

Mass of pile

Engineering News Formula

$$P_a = \frac{2E_r}{s + 0.1}$$

- Developed by A.M. Wellington in 1888
- The most common dynamic formula
- Assumes a factor of safety of 6

- Variables
  - E_r = Rated striking energy of the hammer, ft-kips
  - s = set of the hammer per blow, in.
  - P_a = allowable pile capacity, kips

Other Dynamic Formulae (after Parola, 1970)

Table A.1

TABULATION OF PILE ENERGY FORMULAS

1. Engineering News: $R_u = \frac{12\,E_r}{s + \bar{e}}$

2. Modified Engineering News: $R_u = \frac{12\,E_r}{s + \bar{e}} \cdot \frac{W_1 + n^2 W_p}{W_1 + W_p}$

3. Gow: $R_u = \frac{12\,e_f\,E_r}{s + \bar{e}(W_p/W_1)}$

4. Vulcan Iron Works: $R_u = \frac{120\,E_r}{10s + 1}$

5. Bureau of Yards and Docks: $R_u = \frac{12\,W_1 h}{s + 0.3}$

6. Rankine: $R_u = \frac{AEs}{6L}\left[\sqrt{1 + \frac{144\,E_r L}{s^2\,AE}} - 1\right]$

7. Dutch: $R_u = \frac{12\,E_r}{s} \cdot \frac{W_1}{W_1 + W_p}$

8. Ritter: $R_u = \frac{12\,E_r}{s} \cdot \frac{W_1}{W_1 + W_p} + W_1 + W_p$

9. Eytelwein: $R_u = \frac{12\,E_r}{s + 0.1\,W_p/W_1}$ Single-Acting Hammers

$R_u = \frac{12\,(E_r + A_p\,P)}{s + 0.1\,W_p/W_1}$ Double-Acting Hammers

10. Navy-Mckay: $R_u = \frac{12\,E_r}{s(1 + 0.3\,W_p/W_1)}$

11. Sanders: $R_u = \frac{12\,E_r}{s}$

Table A.1 (continued)

Gates: $R_u = 990\,\sqrt{e_f\,E_r}\,\log(10/s)$

Danish: $R_u = \frac{e_f\,E_r}{\frac{s}{12} + \sqrt{\frac{e_f E_r L}{2\,AE}}}$

Janbu: $R_u = \frac{12\,E_r}{k_u\,s}$

$k_u = C_d(1 + \sqrt{1 + \lambda_e/C_d})$

$\lambda_e = 144\,E_r L/AE\,s^2$

$C_d = 0.75 + 0.15(W_p/W_1)$

Hiley: $R_u = \frac{12\,e_f\,E_r}{s + 1/2(C_1 + C_2 + C_3)} \cdot \frac{W_1 + n^2 W_p}{W_1 + W_p}$

Redtenbacker: $R_u = \frac{AE\,s}{12\,L}\left[\sqrt{1 + \frac{288\,W_r^2\,h}{AEs^2(W_1 + W_p)}} - 1\right]$

Pacific Coast Uniform Building Code:

$$R_u = \frac{AE}{24L}\left[-s + \sqrt{s^2 + \frac{576\,e_f E_r L}{AE} \cdot \frac{W_1 + n^2 W_p^2}{W_1 + W_p}}\right]$$

Canadian National Building Code:

$$R_u = \frac{12\,e_f\,e_1\,E_r}{s + \bar{e}/2}$$

$e_1 = \frac{W_1 + n^2 W_p}{W_1 + W_p}$ Friction Piles

Table A.1 (continued)

$e_1 = \frac{W_1 + 0.5n^2\,W_p}{W_1 + W_p}$ Point Bearing Piles

$\bar{e} = \frac{R_u}{A}\left(\frac{12\,\bar{L}}{E} + 0.0001\right)$

19. Statistical Adjustments of Gates Formula (Olson and Flaate, 1967):

$$R_u = C_4\,\sqrt{e_f\,E_r}\,\log(10/s) - C_5$$

Key:

R_u	ultimate pile capacity (lbs)
E_r	rated hammer energy (ft-lbs)
s	set (in.)
$\bar{e}$	temporary set representing losses (in.)
W_p	weight of the pile (lbs)
W_1	weight of the ram (lbs)
h	height of ram fall (ft)
A	cross-sectional area of pile (in.^{2})
E	Young's modulus of pile (psi)
L	length of pile (ft)
$\bar{L}$	length of pile, measured from head to center of driving resistance resistance (ft)
e_f	hammer efficiency (%)
A_p	effective area of piston (in.^{2})
p	mean pressure of steam or air (psi)

Gates Formula

The WSDOT and FHWA studies resulted in both organizations replacing EN in their specifications with the Gates dynamic formula. However, the Gates dynamic formula, which was originally developed based on correlations with static load test data, is usually restricted to piles that have driving capacities less than 600 kips. The Gates formula, was modified by FHWA for driving capacity as shown below:

$$R_u = 1.75\sqrt{E_r}\,\log_{10}(10N_b) - 100 \qquad \text{9-26a}$$

where: R_u = the ultimate pile capacity (kips)

E_r = the manufacturer's rated hammer energy (ft-lbs) at the **field observed ram stroke**

N_b = the number of hammer blows per 1 inch at final penetration

The number of hammer blows per foot of pile penetration required to obtain the ultimate pile capacity is calculated as follows:

$$N/ft = 12\ (10^x)$$

9-26b

where: $x = [(R_u + 100)/(1.75\sqrt{E_r})] - 1$

Weaknesses of Dynamic Formulae

It is not possible to devise a simple, yet accurate, formula for determining the load bearing capacity of a driven pile and incorporate, therein, allowances for all the modifying conditions that arise. The piles may be long or short, light or heavy, of different cross-sectional areas and materials, and may have been driven with or without a cushion block. Soils very greatly. Diverse styles of pile hammers are employed for driving; some may have, in comparison to the total weight of hammer, or with respect to the weight of the pile that is to be driven, a relatively light ram, others a relatively heavy ram, and may strike few or many blows per minute at widely different velocities.

A simple formula cannot take into account the existing ratio of the weight of the striking part (or ram) to the weight of the pile, though both this and the velocity at the moment of impact are factors which greatly affect the efficiency of the blow and, consequently, the bearing capacity of the pile. The effectiveness of the cushioning, elasticity of the pile and quake of the soil are other variable factors of vital importance not possible of inclusion in a simple formula. (Vulcan product literature, 1930's)

The Wave Equation

$$(9.3) \qquad \sigma = \frac{E}{c} v \quad \text{derived from the "Wave Equation":} \quad \rho \frac{\partial^2 u}{\partial t^2} = E \frac{\partial^2 u}{\partial x^2}$$

where

σ = stress

E = Young's modulus

c = wave propagation speed $\qquad c = \sqrt{\dfrac{E}{\rho}}$

v = particle velocity

- When applied to piling, must be expanded to include damping and spring of shaft resistance
- Closed form solution possible but limited in application
- Solved numerically for real piling problems

- Variables
  - u = displacement, m
  - x = distance along the length of the rod, m
  - t = time, seconds
- Hyperbolic, second order differential equation

Semi-Infinite Pile Theory

- From this, $u_x = -\frac{u_t}{c}$
- This relates pile particle velocity to pile displacement
- Define pile impedance:

- Assumes pile:
  - Has no resistance of any kind along pile shaft
  - Starts at x = 0 and goes to infinity
  - Has no reflections back to the pile head
- For semi-infinite piles,

$$Z_P = \frac{E_P A_P}{c_P} \quad (9.4)$$

where
Z_P = pile impedance
E_P = Young's modulus of the pile material
A_P = pile cross section area
c_P = wave propagation speed (= speed of sound in the pile)

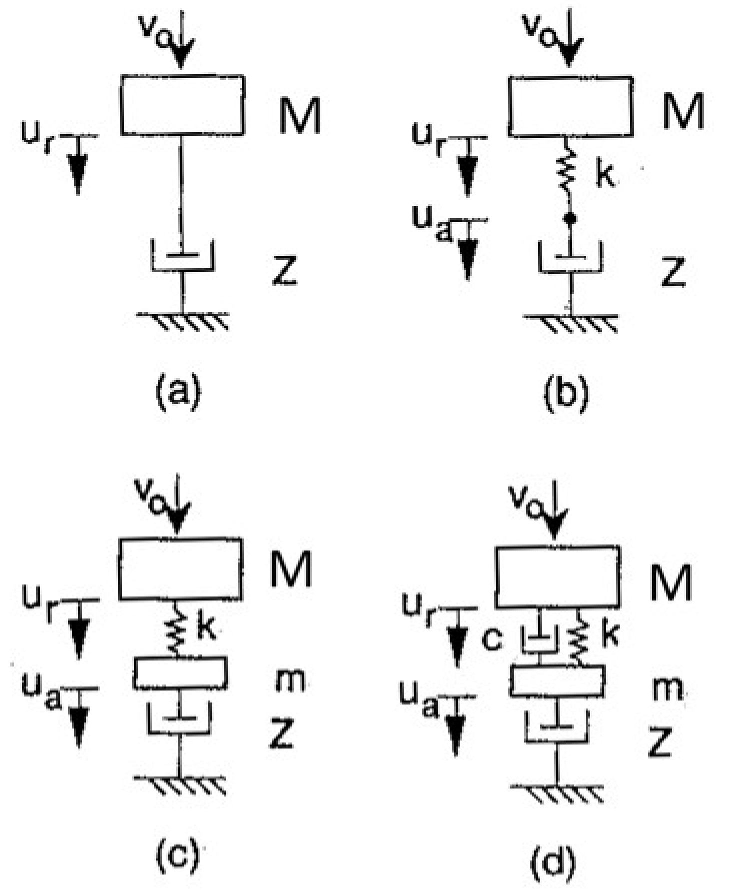
$$\sigma A = F = Z_P \, v_P \quad (9.5)$$

where
σ = axial stress in the pile
A = pile cross section area
F = force in the pile
Z_P = pile impedance
v_P -= pile particle velocity

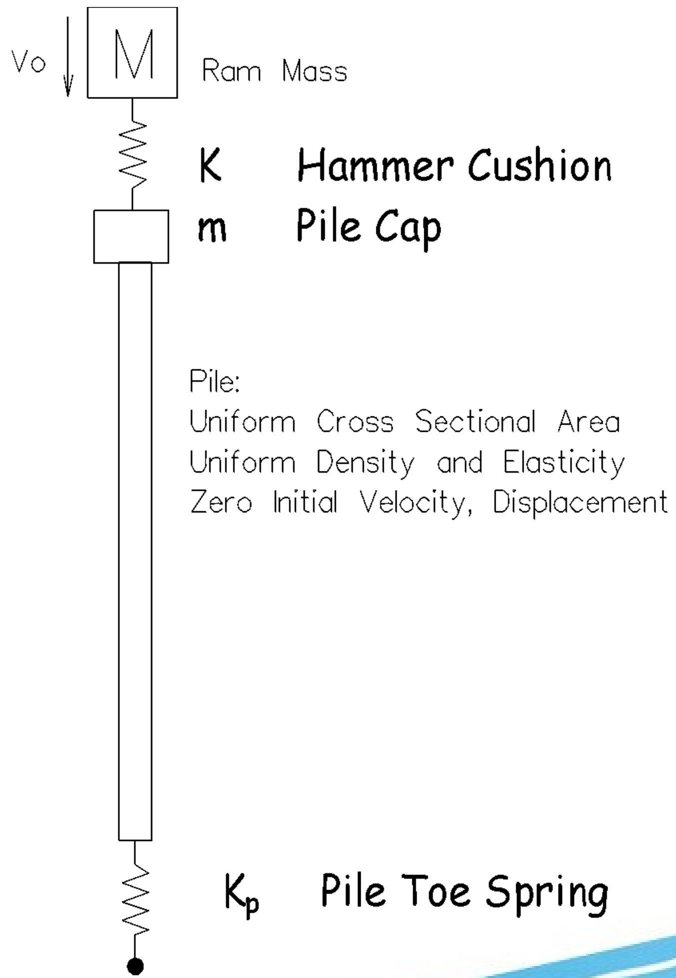
Modeling the Pile Hammer

- With semi-infinite pile theory, pile is modeled as a dashpot
- Ram and pile top motion solved using methods from dynamics and vibrations
- For cushionless ram:

$$F(t)=ZV_oe^{-\frac{Zt}{M}}$$



Closed Form Solution Finite Undamped Pile

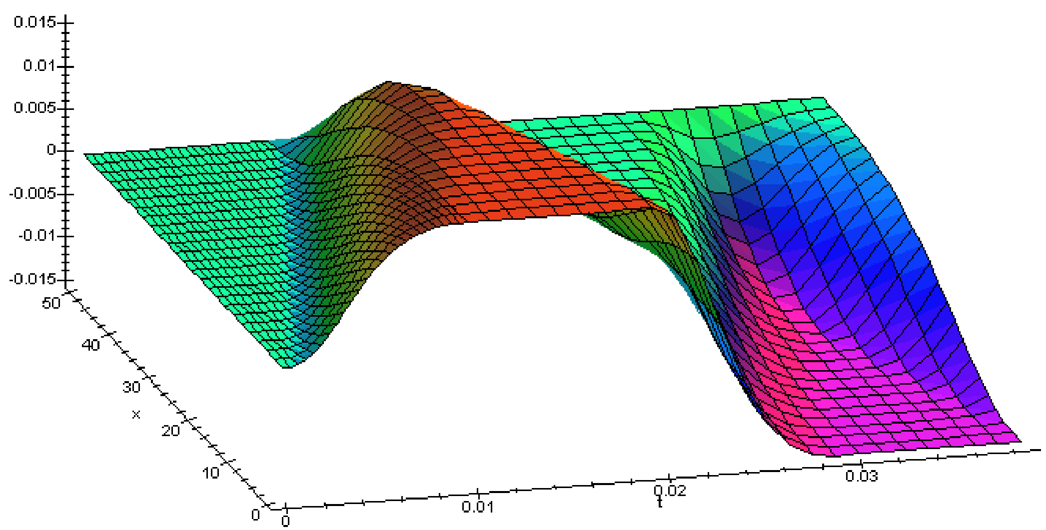


- Simple hammer-pile-soil system
- We will use this to analyze the effect of the variation of the pile toe
- Pile toe spring stiffness can vary from zero (free end) to infinite (fixed end) and an intermediate condition

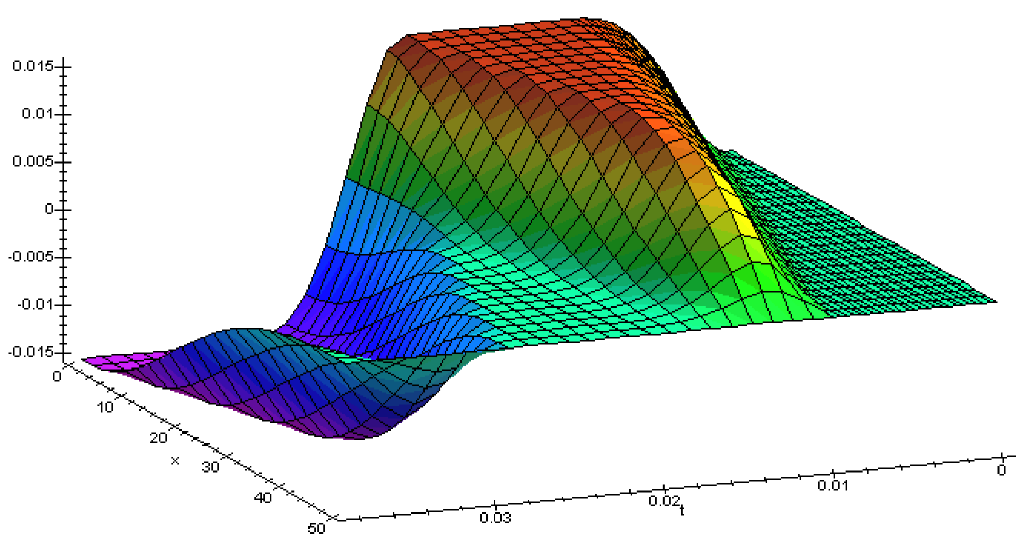
Pile Period: $t_p = \frac{L}{c}$

Fixed End Results

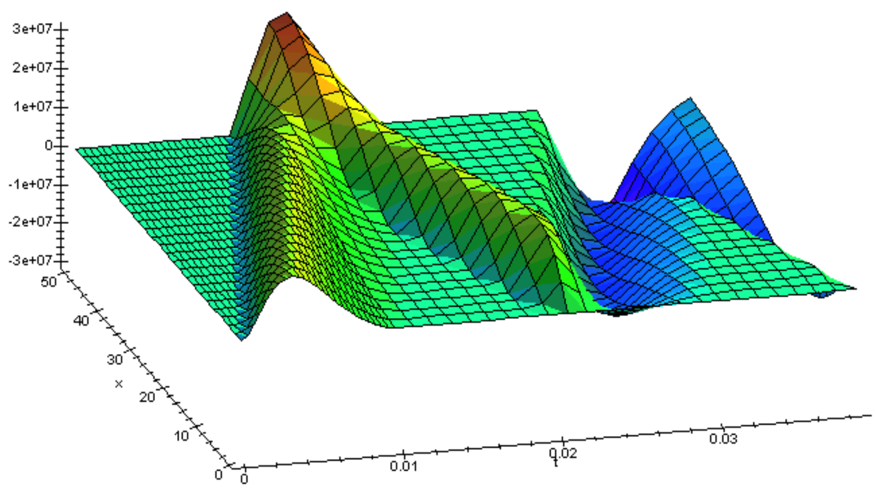
Pile Displacements, 0 < t < 4L/c



Pile Displacements, 0 < t < 4L/c

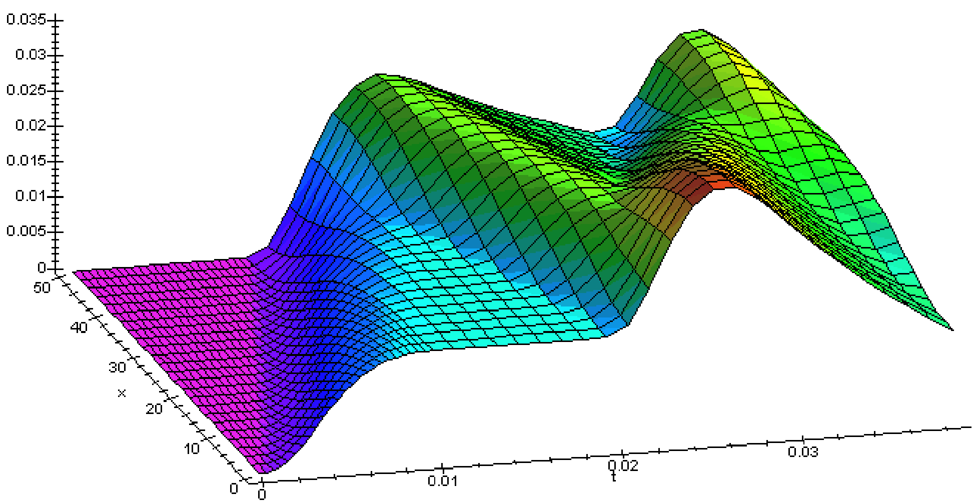


Pile Stresses, 0 < t < 4L/c

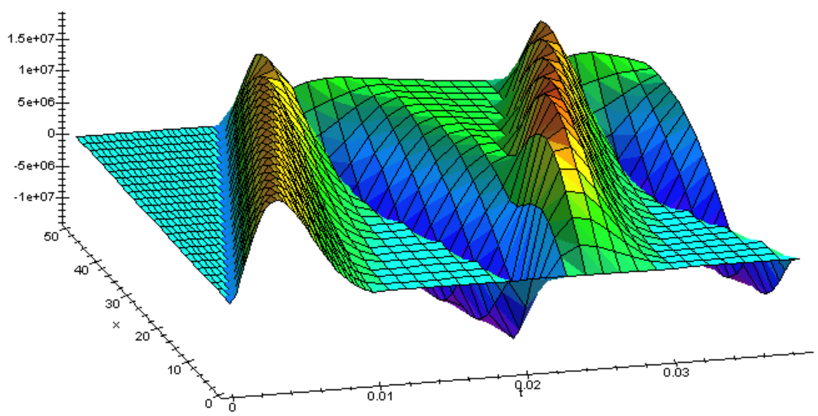


Intermediate Case Results

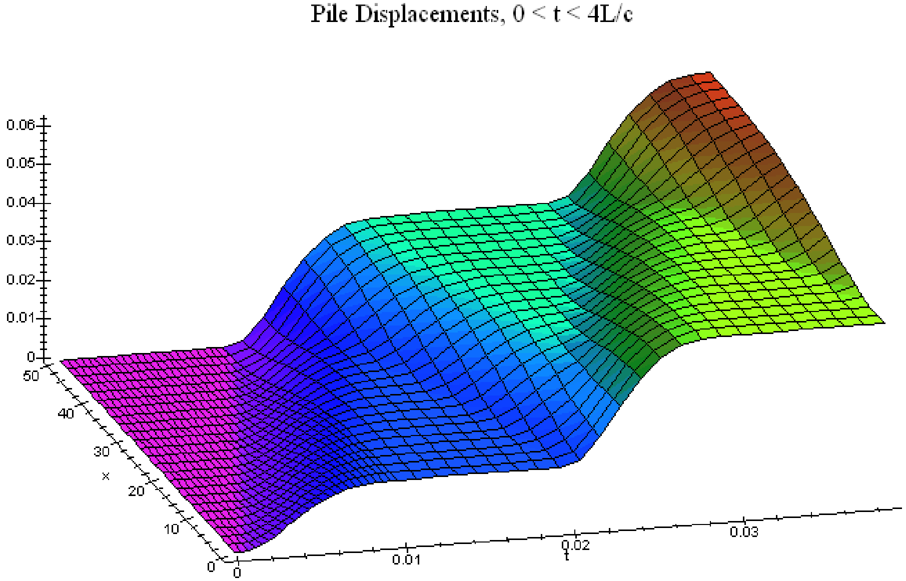
Pile Displacements, 0 < t < 4L/c

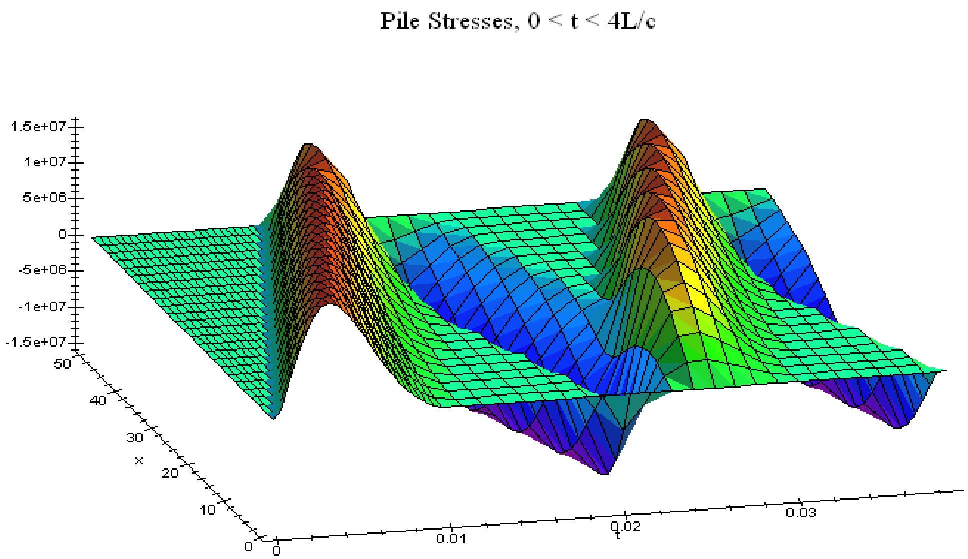


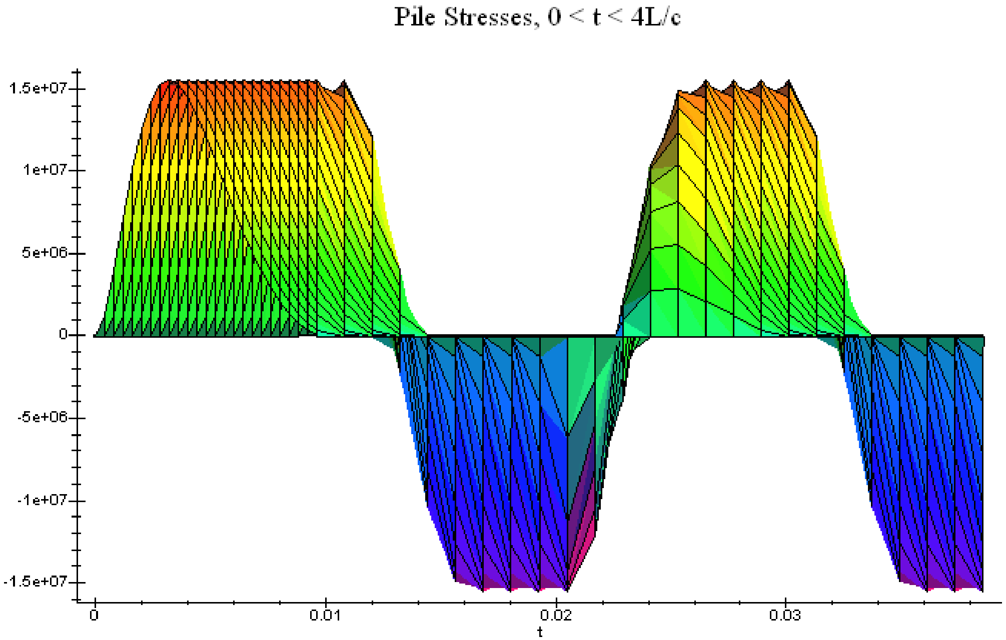
Pile Stresses, 0 < t < 4L/c



Free End Results







Numerical Solutions

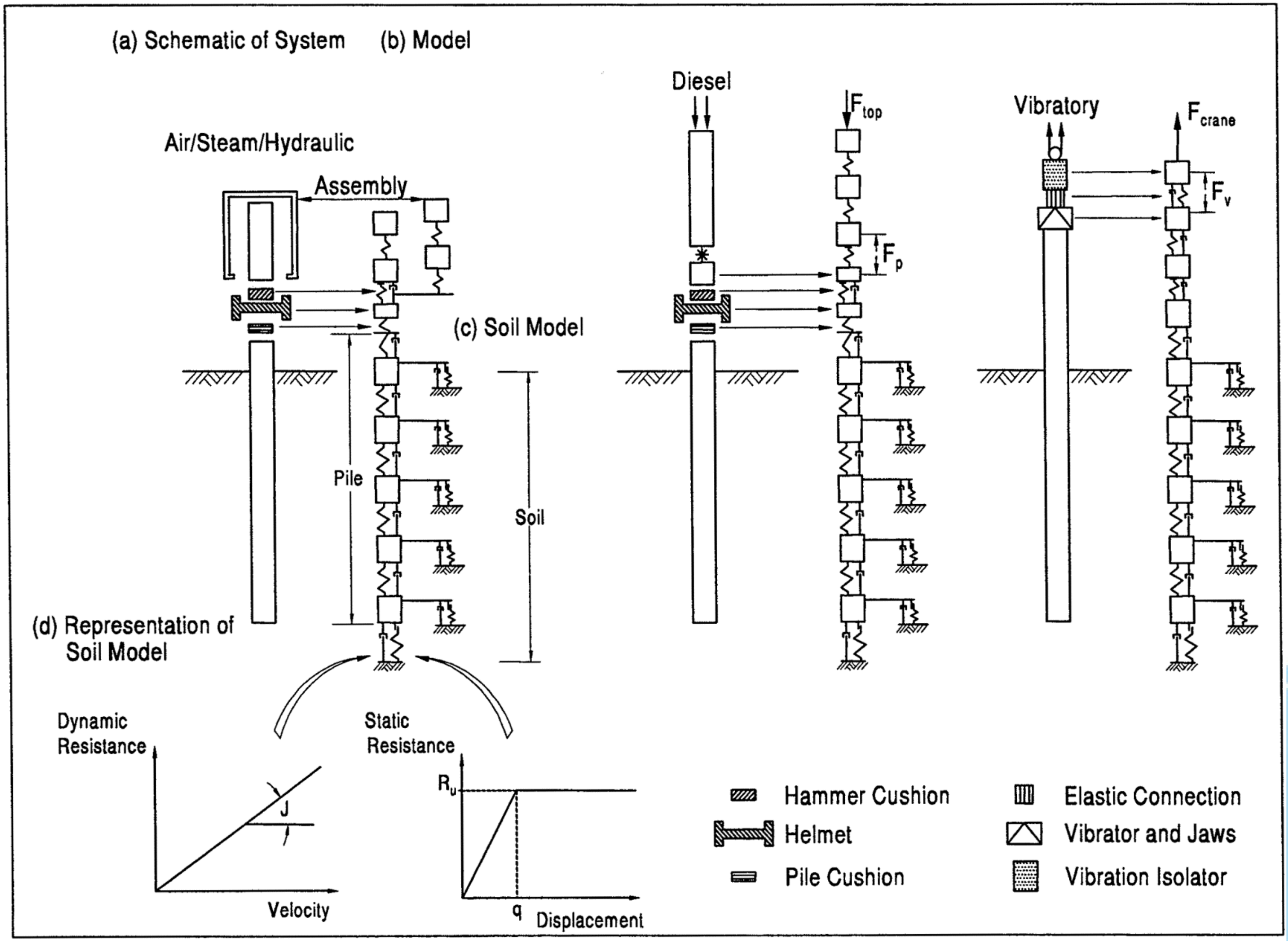
- Subsequent Solutions
  - TTI (Texas Transportation Institute) – late 1960's
    - Very similar to Smith's solution
  - GRL/Case – 1970's and 1980's
    - Added adequate modelling of diesel hammers
    - Added convenience features
  - TNO

- First developed at Raymond Concrete Pile by E.A.L. Smith (1960)
- Solution was first done manually, then computers were involved
- One of the first applications of computers to civil engineering

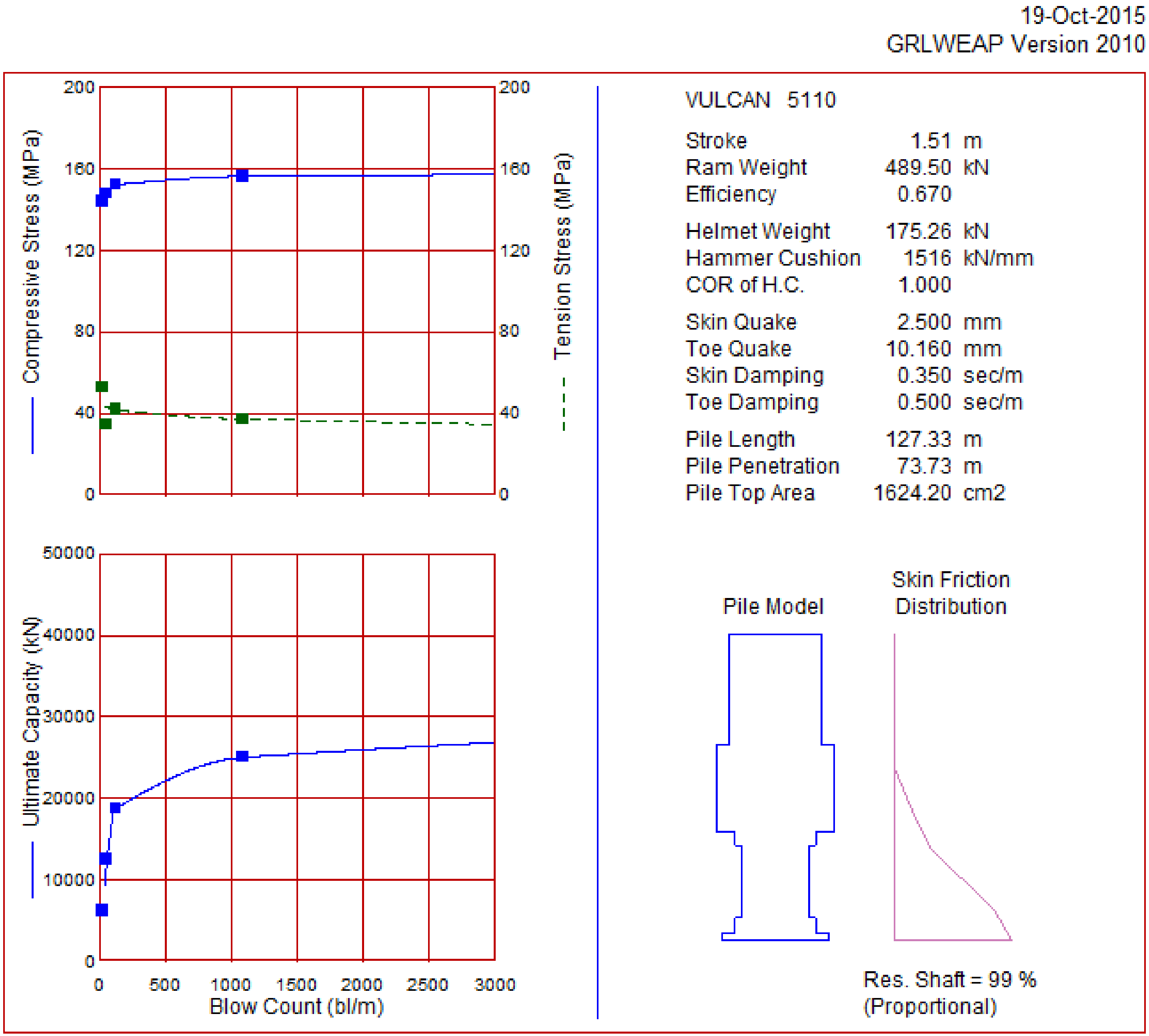
Necessity for Numerical Solution

- Non-uniformity of the pile cross section along the length of the pile, and in some cases the pile changes materials.
- Slack conditions in the pile. These are created by splices in the pile and also pile defects.
- With diesel hammers, the force-time characteristics during combustion are difficult to simulate in closed form. (It actually took around fifteen years, until the first version of WEAP was released, to do a proper job numerically.)
- Unusual driving conditions, such as driving from the bottom of the pile or use of a long follower between the hammer and the pile head.
- Existence of dampening, both at the toe, along the shaft, and in all of the physical components of the system. In theory, inclusion of distributed spring constant and dampening along the shaft could be simulated using the Telegrapher's wave equation, but other factors make this impractical also.
- Non-linear force-displacements along the toe and shaft, and in the cushion material. Exceeding the "elastic limit" of the soil is in fact one of the central objects of pile driving.
- Non-uniformity of soils along the pile shaft, both in type of soil and in the intensity of the resistance.
- Inextensibililty of many of the interfaces of the system, including all interfaces of the hammer-cushion-pile system and the pile toe itself.

Wave Equation for Piles in Practical Solution



"Bearing Graph" Result of Wave Equation



Allowable Axial Stresses in Driven Piles

Table 9-11. Maximum allowable stresses in pile for top driven piles (after AASHTO, 2002; FHWA, 2006a)

Pile Type	**Maximum Allowable Stresses** (f_y = yield stress of steel; f'_c = 28-day compressive strength of concrete; f_{pe} = pile prestress)
Steel H-Piles	**<u>Design Stress</u>** 0.25 f_y 0.33 f_y If damage is unlikely, and confirming static and/or dynamic load tests are performed and evaluated by engineer. **<u>Driving Stress</u>** 0.9 f_y
	32.4 ksi (223 MPa) for ASTM A-36 (f_y = 36 ksi; 248 MPa) 45.0 ksi (310 MPa) for ASTM A-572 or A-690, (f_y = 50 ksi; 345 MPa)
Unfilled Steel Pipe Piles	**<u>Design Stress</u>** 0.25 f_y 0.33 f_y If damage is unlikely, and confirming static and/or dynamic load tests are performed and evaluated by engineer. **<u>Driving Stress</u>** 0.9 f_y
	27.0 ksi (186 MPa) for ASTM A-252, Grade 1 (f_y = 30 ksi; 207 MPa) 31.5 ksi (217 MPa) for ASTM A-252, Grade 2 (f_y = 35 ksi; 241 MPa) 40.5 ksi (279 MPa) for ASTM A-252, Grade 3 (f_y = 45 ksi ; 310 MPa)
Concrete filled steel pipe piles	**<u>Design Stress</u>** 0.25 f_y (on steel area) ***<u>plus</u>*** 0.40 f'_c (on concrete area) **<u>Driving Stress</u>** 0.9 f_y
	27.0 ksi (186 MPa) for ASTM A-252, Grade 1 (f_y = 30 ksi; 207 MPa) 31.5 ksi (217 MPa) for ASTM A-252, Grade 2 (f_y = 35 ksi; 241 MPa) 40.5 ksi (279 MPa) for ASTM A-252, Grade 3 (f_y = 45 ksi ; 310 MPa)
Precast Prestressed Concrete Piles	**<u>Design Stress</u>** 0.33 f'_c - 0.27 f_{pe} (on gross concrete area) ; f'_c minimum of 5.0 ksi (34.5 MPa) f_{pe} generally > 0.7 ksi (5 MPa)
	<u>Driving Stress</u> Compression Limit $< 0.85\ f'_c - f_{pe}$ (on gross concrete area) Tension Limit (1) $< 3\ (f'_c)^{1/2} + f_{pe}$ (on gross concrete area) **<u>US Units</u>*** $< 0.25\ (f'_c)^{1/2} + f_{pe}$ (on gross concrete area) **<u>SI Units</u>** * Tension Limit (2) $< f_{pe}$ (on gross concrete area) (1) - Normal Environments ; (2) - Severe Corrosive Environments ***Note:** f'_c and f_{pe} must be in psi and MPa for US and SI equations, respectively.
Conventionally reinforced concrete piles	**<u>Design Stress</u>** 0.33 f'_c (on gross concrete area) ; f'_c minimum of 5.0 ksi (34.5 MPa)
	<u>Driving Stress</u> Compression Limit $< 0.85\ f'_c$; Tension Limit $< 0.70\ f_y$ (of steel reinforcement)
Timber Pile	**<u>Design Stress</u>** 0.8 to 1.2 ksi (5.5 to 8.3 MPa) for pile toe area depending upon species
	<u>Driving Stress</u> Compression Limit $< 3\ \sigma_a$ Tension Limit $< 3\ \sigma_a$ σ_a - AASHTO allowable working stress

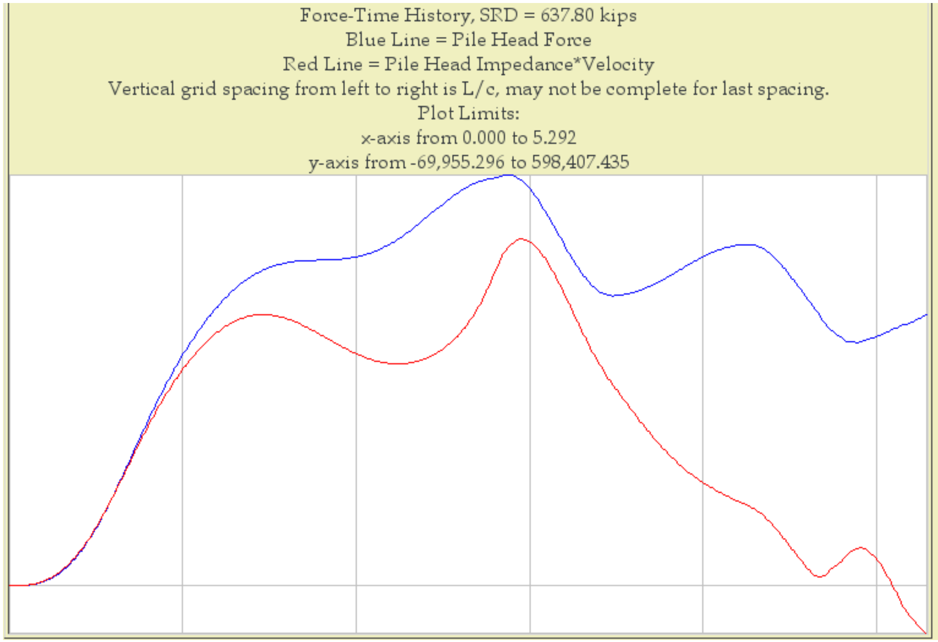
TAMWAVE

- Originally developed in 2005; recently extensively revised
- With simple soil and pile input, capable of the following for single piles:
  - Axial load-deflection analysis
  - Lateral load-deflection analysis
  - Wave Equation Drivability Analysis
- Uses method presented earlier to estimate static capacity
- Uses ALP method for axial load-deflection analysis
- Uses CLM 2 method for lateral load-deflection analysis
- Hammer database (in ascending energy order) and initial hammer selection estimate available
- Includes estimate of soil set-up in clays

16” Concrete Pile Example

General Output for Wave Equation Analysis
2018-04-08T17:02:24-04:00

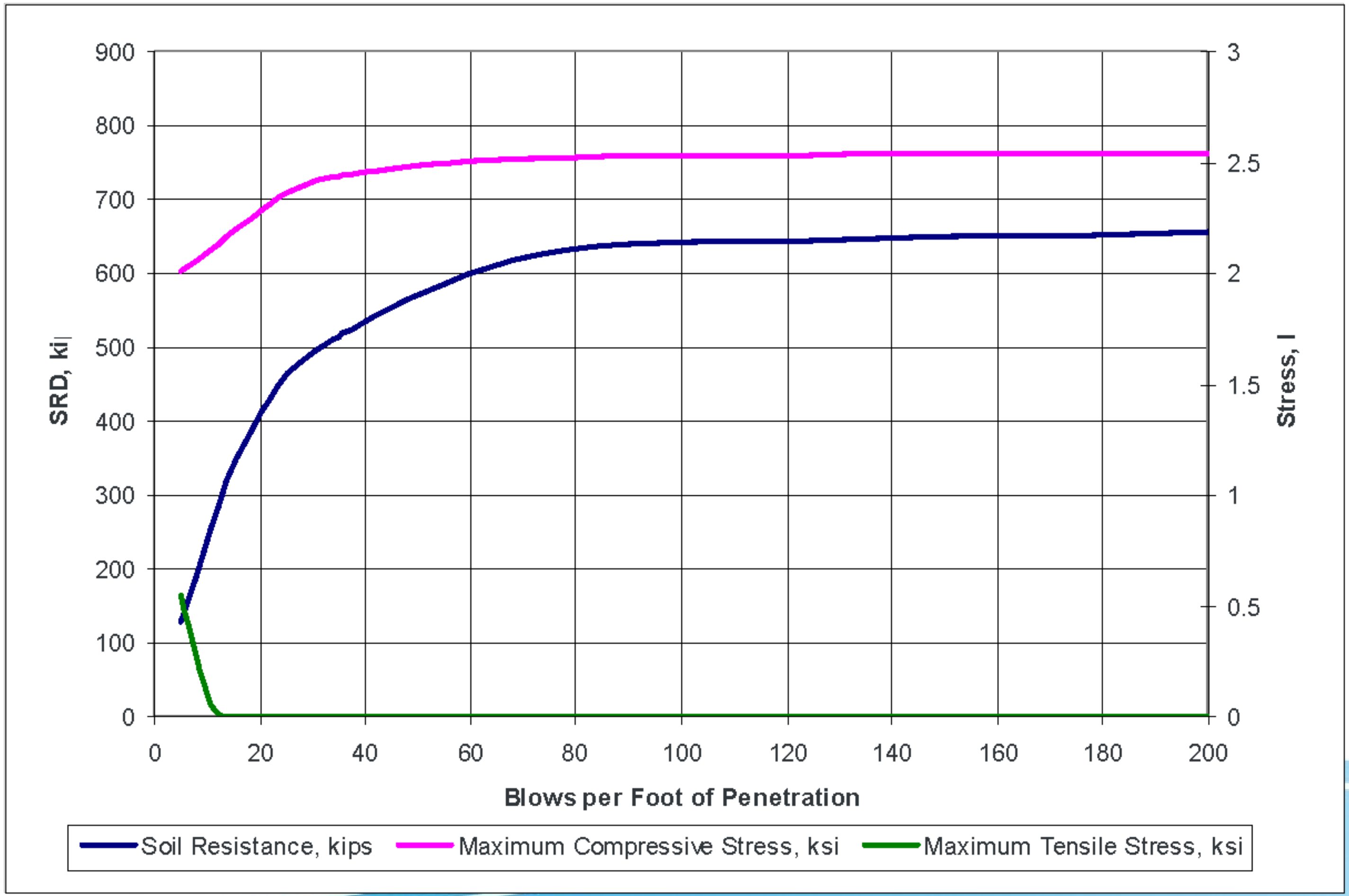
Time Step, msec	0.04024
Pile Weight, lbs.	16,000
Pile Stiffness, lb/ft	1,777,778
Pile Impedance, lb-sec/ft	103,000.1
L/c, msec	4.82813
Pile Toe Element Number	62
Length of Pile Segments, ft.	1
Hammer Manufacturer and Size	VULCAN O30
Hammer Rated Striking Energy, ft-lbs	90000
Hammer Efficiency, percent	67
Length of Hammer Cushion Stack, in.	25
Soil Resistance to Driving (SRD) *for detailed results only*, kips	637.8
Percent at Toe	56.70
Toe Quake, in.	0.320
Toe Damping, sec/ft	0.05



16" Concrete Pile Example

Soil Resistance, kips	Permanent Set of Pile Toe, inches	Blows per Foot of Penetration	Maximum Compressive Stress, ksi	Element of Maximum Compressive Stress	Maximum Tensile Stress, ksi	Element of Maximum Tensile Stress	Number of Iterations
127.6	2.335	5.1	2.01	3	0.55	40	1200
255.1	1.114	10.8	2.1	3	0.05	5	1200
382.7	0.67	17.9	2.24	33	0	62	1048
510.2	0.352	34	2.44	37	0	62	764
637.8	0.135	88.8	2.53	35	0	62	636
765.4	0.009	1381	2.58	32	0	62	609

●16” Concrete Pile Example



Basic Steps in Wave Equation Analysis

- Gather information
  - Hammer type, ram weight, cushion data, etc.
  - Suggested trial energy shown in chart below (included in program)
  - Pile data, including length, material, etc.
  - Soil data; layers, soil types, properties

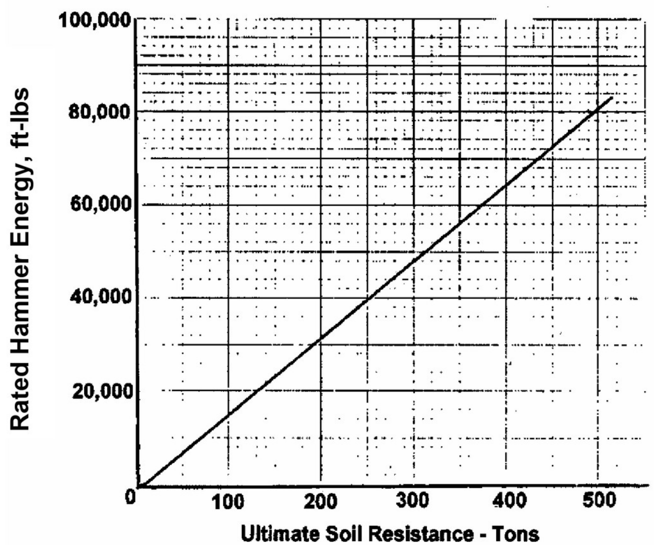


Figure 9-44. Suggested trial hammer energy for wave equation analysis.

- Construct Analysis
  - Run static capacity analysis on pile as pile driving resistance
  - Apply setup factor (if necessary) on static capacity
  - Input data for hammer, pile and soil resistance profile into wave equation analysis
- Run program
  - Run wave equation analysis for different soil resistances (factoring original static analysis) and (for some wave equation programs) different depths of driving
- Analyse Results
  - Blow counts, tension and compression stresses, driving time

Soil Resistance to Driving

A static analysis should also be used to calculate the **soil resistance to driving**, SRD, that must be overcome to reach the estimated pile penetration depth necessary to develop the ultimate capacity. This information is necessary for the designer to select a pile section with the driveability to overcome the anticipated soil resistance and for the contractor to properly size equipment. Driveability aspects of design are discussed in Section 9.9.

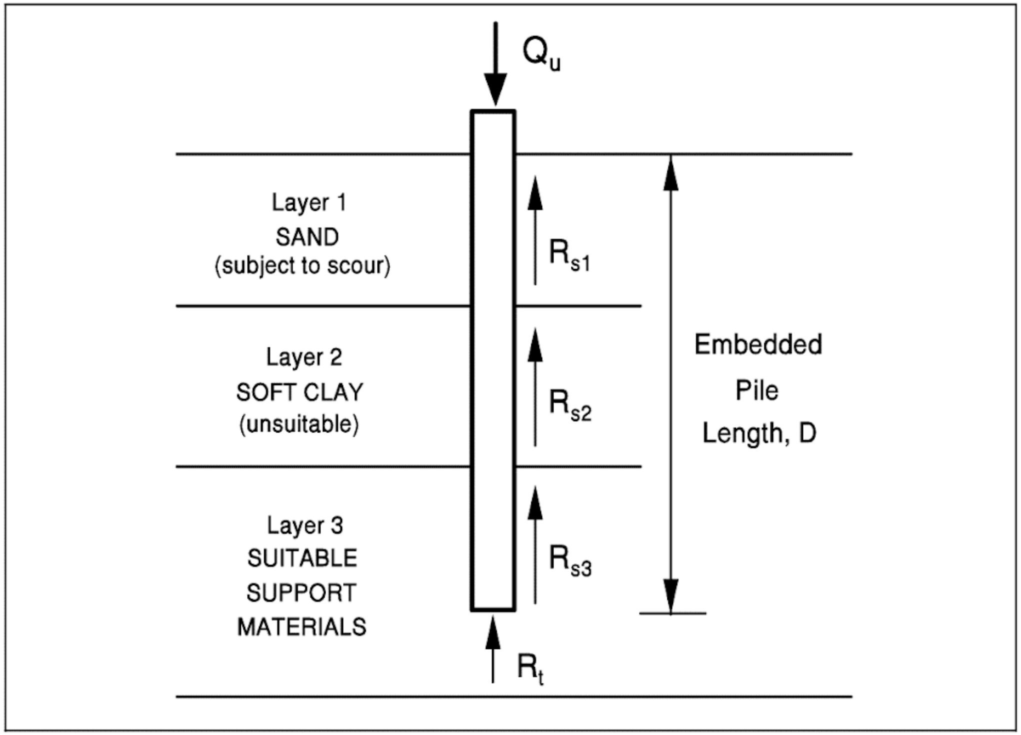
In the SRD calculation, a factor of safety is not used. The soil resistance to driving is the sum of the soil resistances from the scour susceptible and unsuitable layers plus the soil resistance in the suitable support materials to the estimated penetration depth.

$$SRD = R_{s1} + R_{s2} + R_{s3} + R_t$$

Soil resistances in this calculation should be the resistance at the time of driving. Hence time dependent changes in soil strengths due to **soil setup or relaxation** should be considered (see Table 5-8 in Chapter 5 for brief explanation of these terms and Section 9.5.5 for more discussion). For the example presented in Figure 9-5, the driving resistance from the unsuitable clay layer would be reduced by the sensitivity of the clay. Therefore, R_{s2} would be R_{s2} / 2 for a clay with a sensitivity of 2. The soil resistance to driving to depth D would then be as follows

$$SRD = R_{s1} + R_{s2}/2 + R_{s3} + R_t$$

This example problem considers only the driving resistance at the final pile penetration depth. In cases where piles are driven through hard or dense layers above the estimated pile penetration depth, the soil resistance to penetrate these layers should also be calculated. Additional information on the calculation of time dependent soil strength changes is provided in Section 9.9 of this chapter.



Pile Setup in Clays

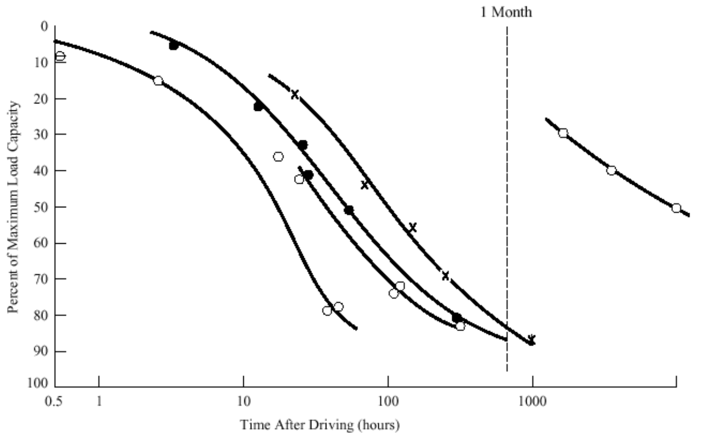


Table 9-8

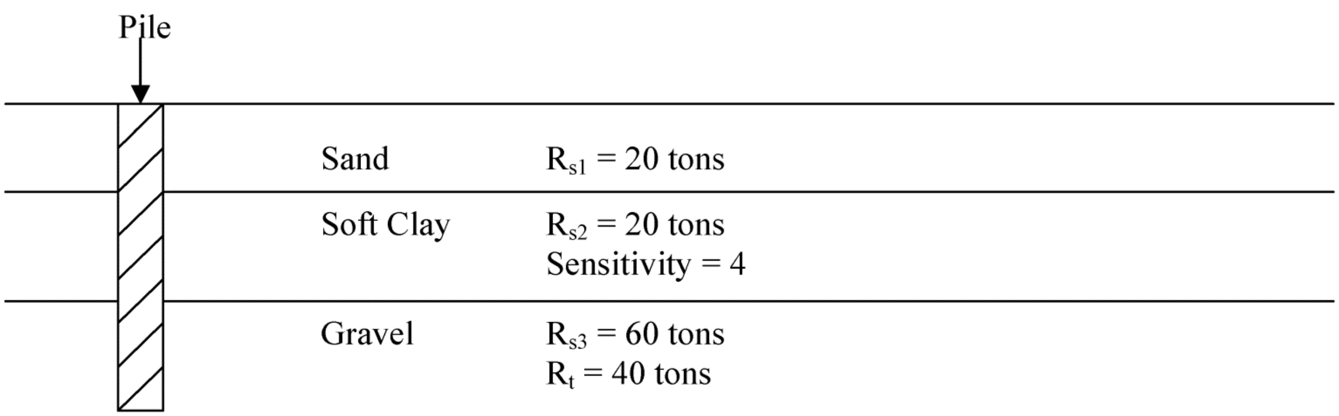
Soil setup factors (after FHWA, 1996)

Predominant Soil Type Along Pile Shaft	Range in Soil Set-up Factor	Recommended Soil Set-up Factors*	Number of Sites and (Percentage of Data Base)
Clay	1.2 - 5.5	2.0	7 (15%)
Silt - Clay	1.0 - 2.0	1.0	10 (22%)
Silt	1.5 - 5.0	1.5	2 (4%)
Sand - Clay	1.0 - 6.0	1.5	13 (28%)
Sand - Silt	1.2 - 2.0	1.2	8 (18%)
Fine Sand	1.2 - 2.0	1.2	2 (4%)
Sand	0.8 - 2.0	1.0	3 (7%)
Sand - Gravel	1.2 - 2.0	1.0	1 (2%)
* Confirmation with local experience recommended			

soil setup factor: the failure load from a static load test divided by the end-of-drive wave equation capacity

Pile Resistance Example

Example 9-1: Find the ultimate capacity and driving capacity for the pile from the data listed in the profile. The hydraulic specialist determined that the sand layer is susceptible to scour. The geotechnical specialist determined that the soft clay layer is unsuitable for providing resistance.



Solution:

Ultimate capacity $= R_{s3} + R_t$

$= 60 \text{ tons} + 40 \text{ tons} = 100 \text{ tons}$

Driving capacity $= R_{s1} + (R_{s2}/\text{Sensitivity}) + R_{s3} + R_t$

$$= 20\,\text{tons} + \frac{20\,\text{tons}}{4} + 60\,\text{tons} + 40\,\text{tons} = 125\,\text{tons}$$

Dynamic Testing

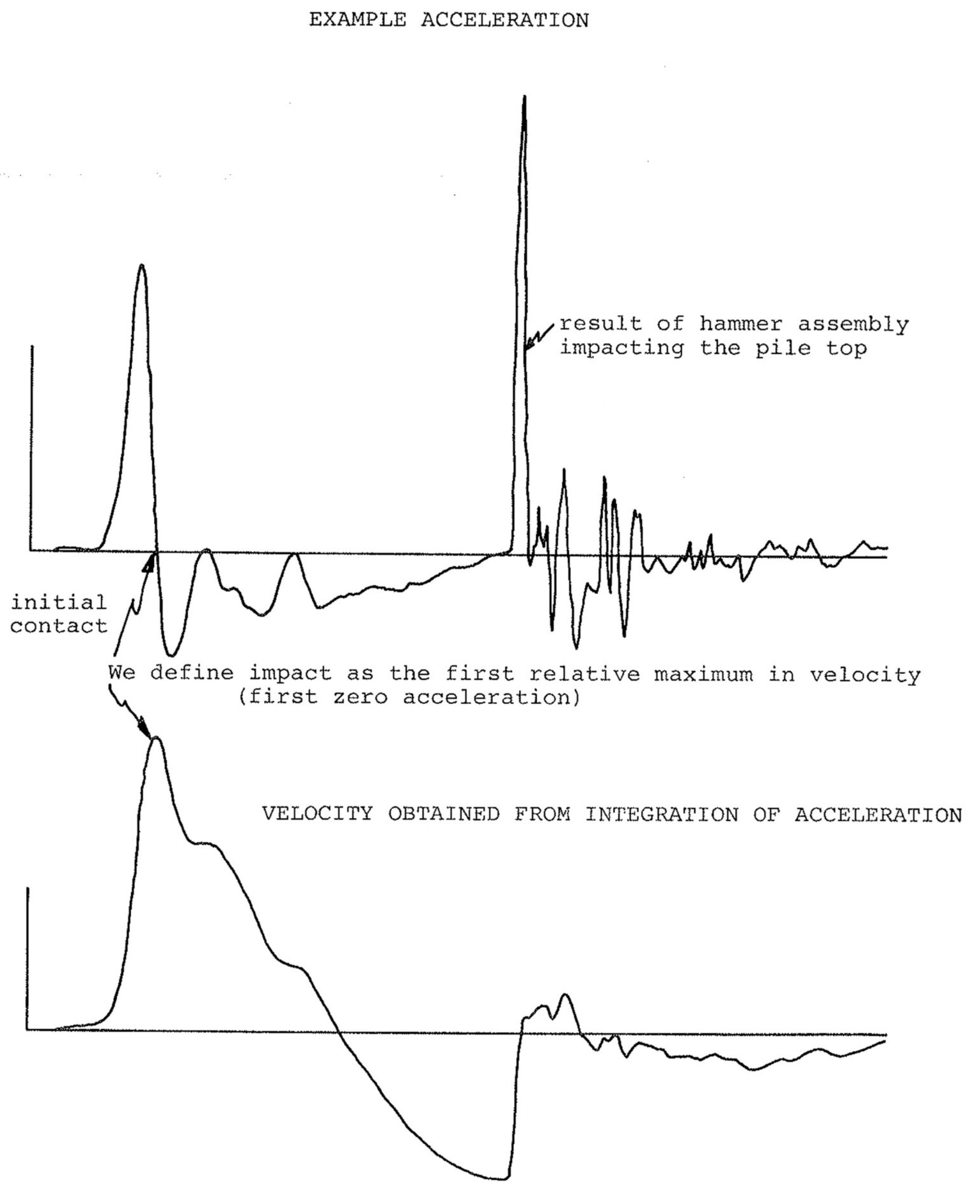
a) Driving of prestressed pile

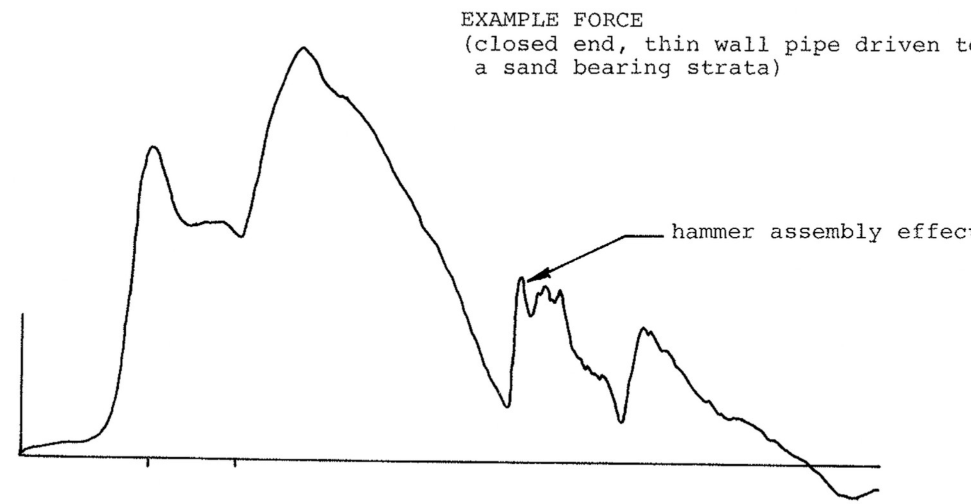
b) End of driving of prestressed pile showing driving cushion compressed

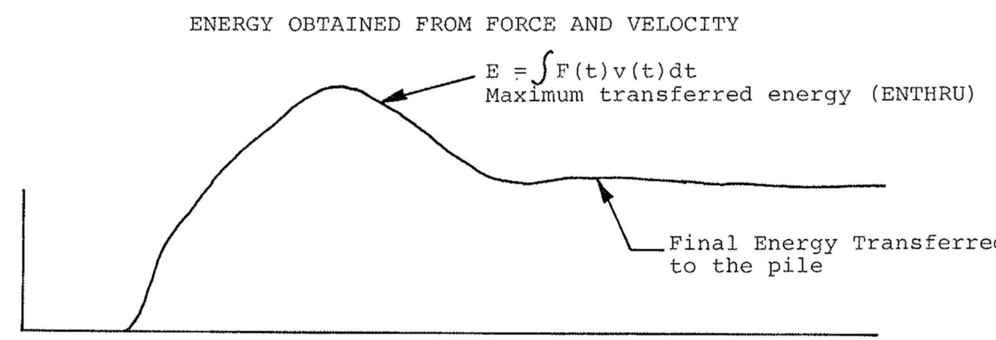
c) Restrike of prestressed pile

- Load is applied by impact force of ram
- Force is measured by strain gages
- Velocity is measured by accelerometers with the results integrated in time
- This force-time and velocity-time data is used to estimate the static capacity of the pile and monitor pile stresses during driving

Integration of Acceleration to Velocity
Energy Obtained from Force and Velocity





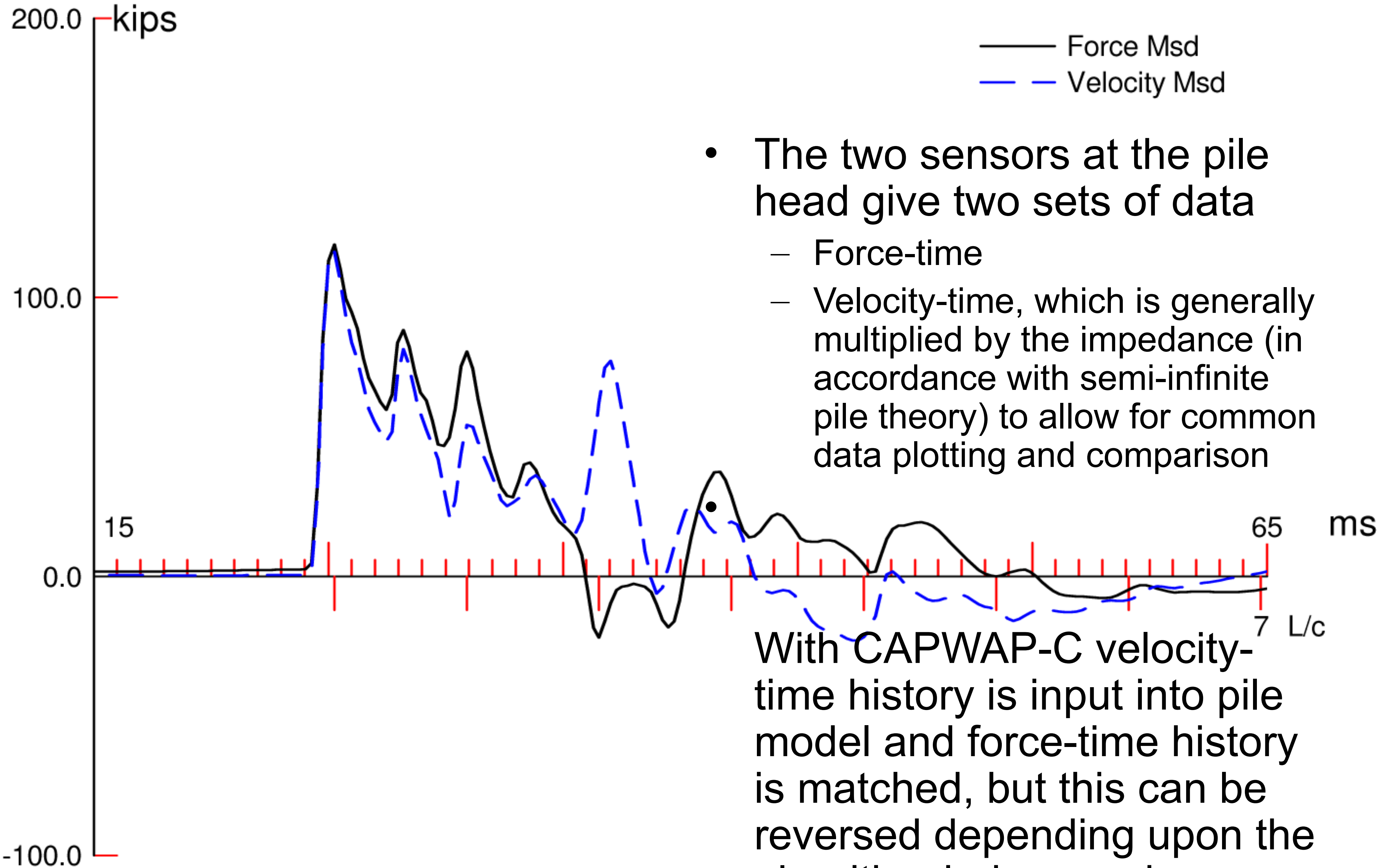


The Pile Driving Analyser

- For Dynamic Pile Monitoring:
  - Stresses
  - Hammer Performance
  - Pile Integrity

- For Dynamic Load Test:
  - Bearing Capacity at time of testing
  - Separating Dynamic from Total (Static + Dynamic) Soil Resistance
    - Case Method
    - CAPWAP-C

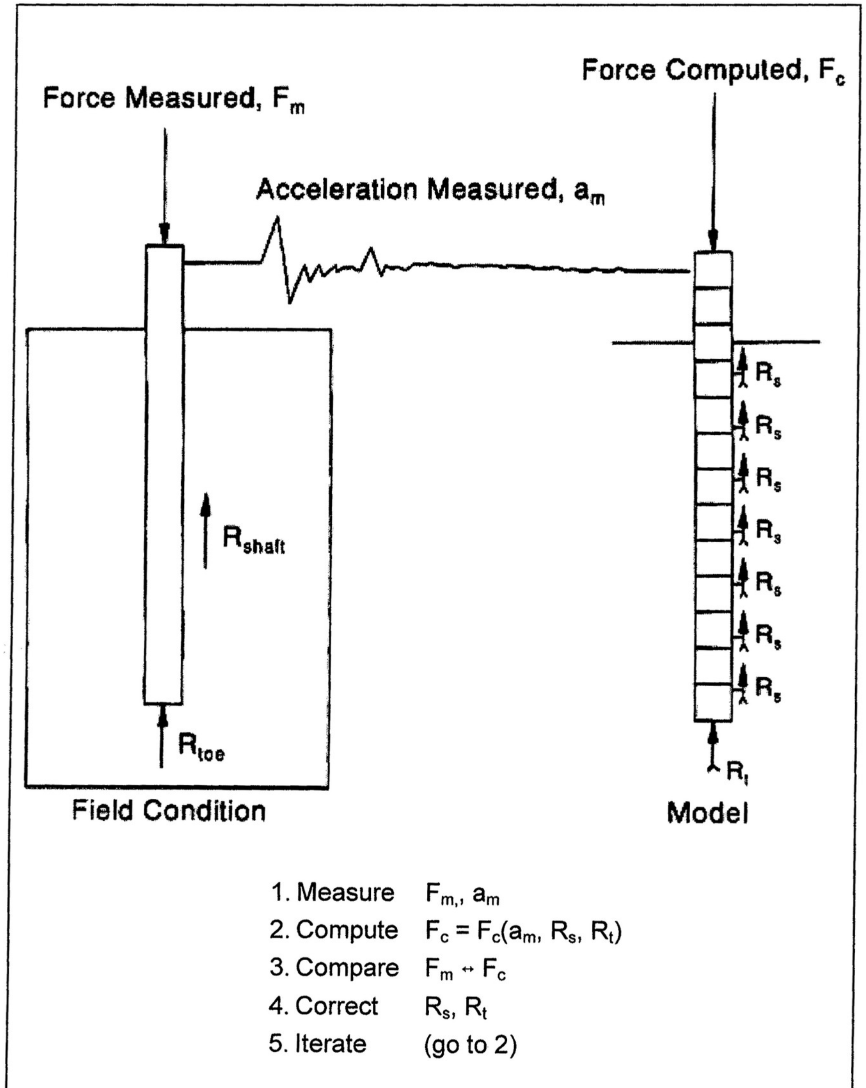
Data Output of Pile Driving Analyzer

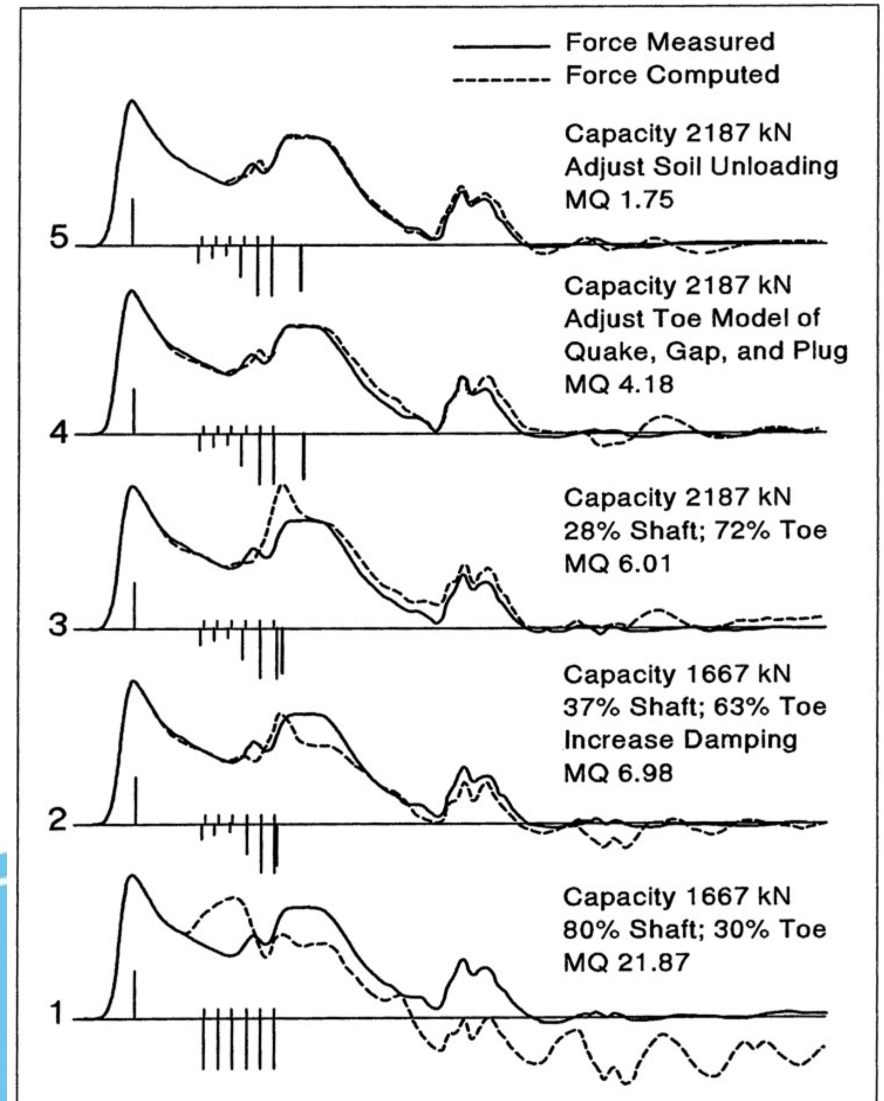


- The two sensors at the pile head give two sets of data
  - Force-time
  - Velocity-time, which is generally multiplied by the impedance (in accordance with semi-infinite pile theory) to allow for common data plotting and comparison

With CAPWAP-C velocity-time history is input into pile model and force-time history is matched, but this can be reversed depending upon the algorithm being used

CAPWAP Modeling of Pile Response and Capacity





Case Method for Pile Analysis

- Simple "Back of the Envelope" Method for Estimating Pile Capacity from Dynamic Results
- Assumptions:
  - The pile resistance is concentrated at the pile toe, as was the case with the closed form solutions above.
  - The static toe resistance is completely plastic, as opposed to the purely elastic resistance modelled above. (Both the wave equation numerical analysis and CAPWAP assume an elasto-plastic model for the static component of the resistance.)
-

- Uses output from Pile Analyzer
- Definitions
  - t_1 = time of peak impact force (or later, depending upon interpretation method)
  - $t_2 = t_1 + 2L/c$
  - F_1 = pile head force at time t_1, kN or kips
  - F_2 = pile head force at time t_2, kN or kips
  - V_1 = pile head velocity at time t_1, m/sec or ft/sec
  - V_2 = pile head velocity at time t_2, m/sec or ft/sec

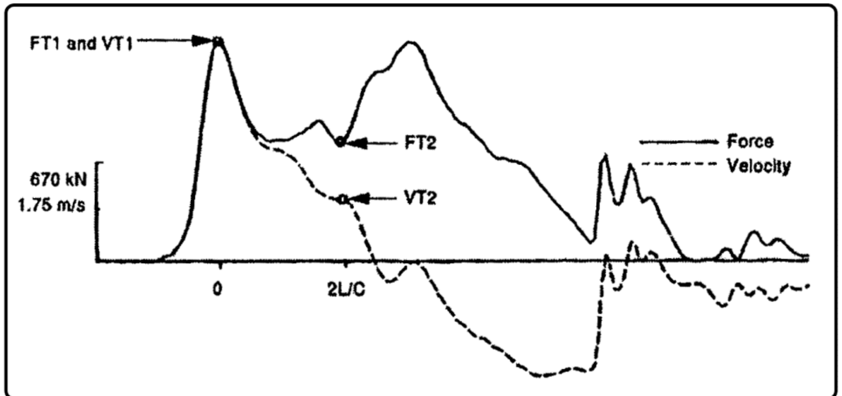
Case Method for Pile Analysis

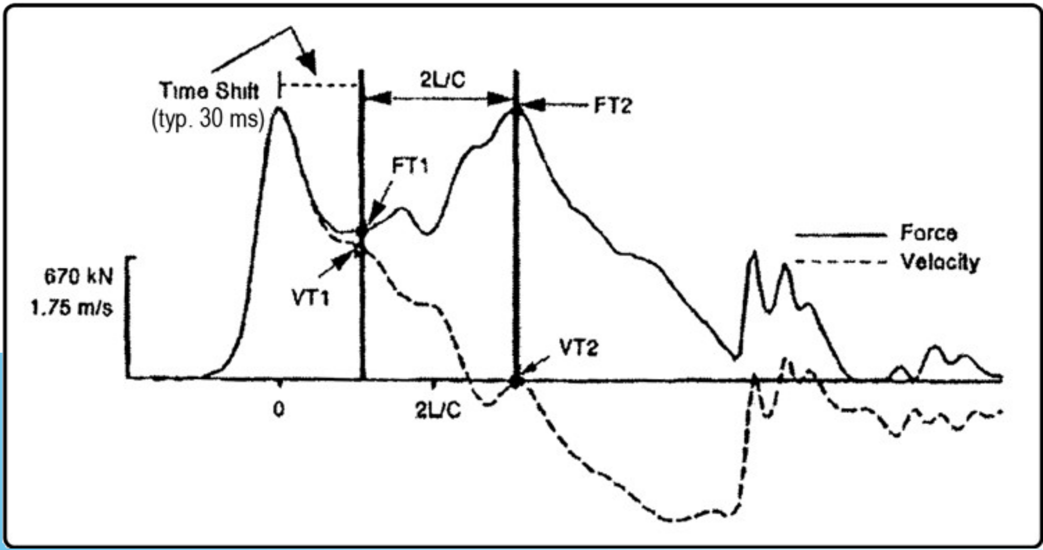
- Definitions
  - Z = pile impedance, kN-sec/m or kip-sec/ft
  - ZV1 = product of impedance and velocity at time t1, kN or kips
  - ZV2 = product of impedance and velocity at time t2, kN or kips
  - RTL = total resistance, kN or kips
  - RD = dynamic resistance, kN or kips
  - RS = static resistance, kN or kips
  - J = Case damping constant, dimensionless

- Basic Formulas
  - RTL = RD + RS
  - RTL = (F1 + F2)/2 + (ZV1 – ZV2)/2
  - RD = J(F1 + ZV1 – RTL)
  - RS = RTL - RD

Case Method for Pile Analysis

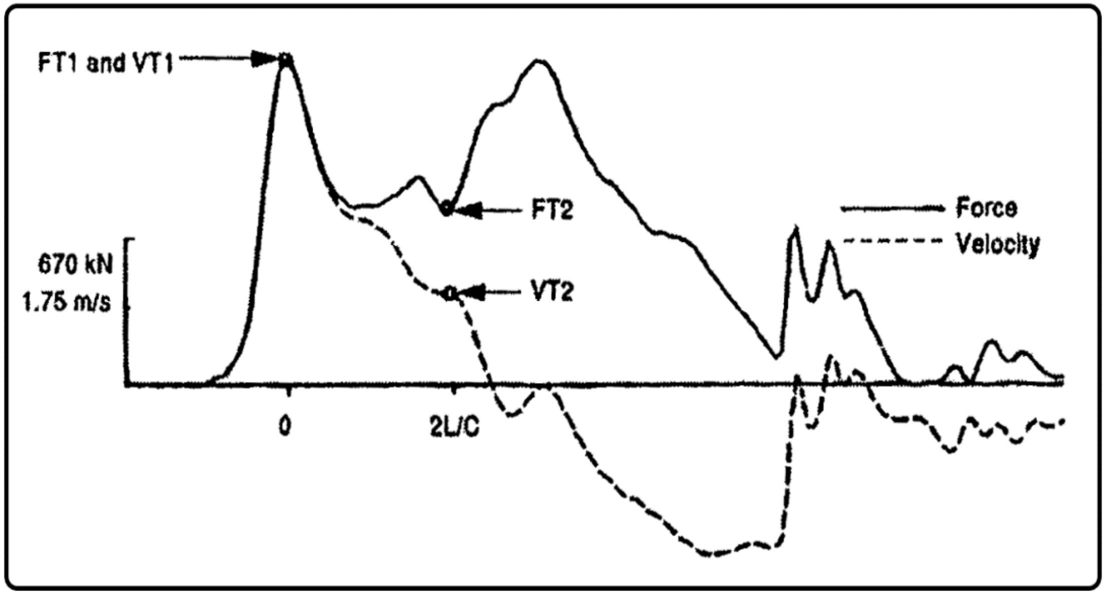
- Interpretation Methods
  - RSP Method
    - The t1 for the RSP method is the first peak point in the force-time curve (see below)
    - Best used for piles with low displacements and high shaft resistances.
  - RMX Method
    - The time t1 is for the RMX method is the peak initial force plus a time shift, generally 30 msec (Fellenius (2009), see below.) The time t2 is still t1 + 2L/c. This time shift is to account for the delay caused by the elasticity of the soil.
    - The RMX method is best for piles with large toe resistances and large displacement piles with the large toe quakes that accompany them. The quake of the soil is the distance from initial position of the soil-pile interface at which the deformation changes from elastic to plastic, see variable "Q". The toe quake is proportional to the size of the pile at the toe.





Case Method Example

- Find
  - Case Method ultimate capacity for the RSP and RMX methods

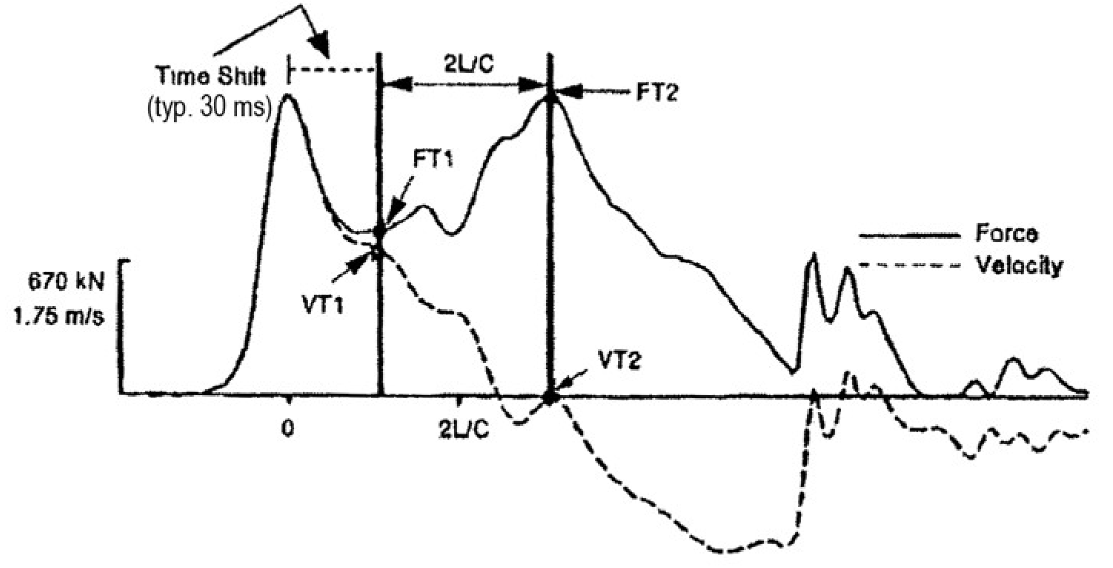


- Given
  - Pile with impedance of 381 kN-sec/m
  - Force-time history as shown at the left
    - F1 = 1486 kN (shown as FT1)
    - F2 = 819 kN (shown as FT2)
    - V1 = 3.93 m/sec (shown as VT1)
    - Z = 670 kN / 1.75 m/sec = 381 kN-sec/m
    - ZV1 = (381)(3.93) = 1497.33 kN
    - V2 = 1.07 m/sec (shown as VT2)
    - ZV2 = (381)(1.07) = 407.67 kN

Case Method Example

- RSP Method
  - Data points given
  - Assume J = 0.4
  - RTL = (1486 + 819)/2 + (1497.33 – 407.67)/2 = 1697 kN
  - RD = 0.4(1486 + 1497.33 – 1697) = 514 kN
  - RS = 1697 – 514 = 1183 kN

- RMX Method
  - Data points different, see graph below, because of the different values of t1 and t2
  - F1 = 819 kN
  - F2 = 1486 kN
  - V1 = 1.92 m/sec
  - ZV1 = (381)(1.92) = 731.52 kN
  - VT2 = 0 m/sec
  - ZV2 = (381)(0) = 0 kN-sec/m



Case Method Example

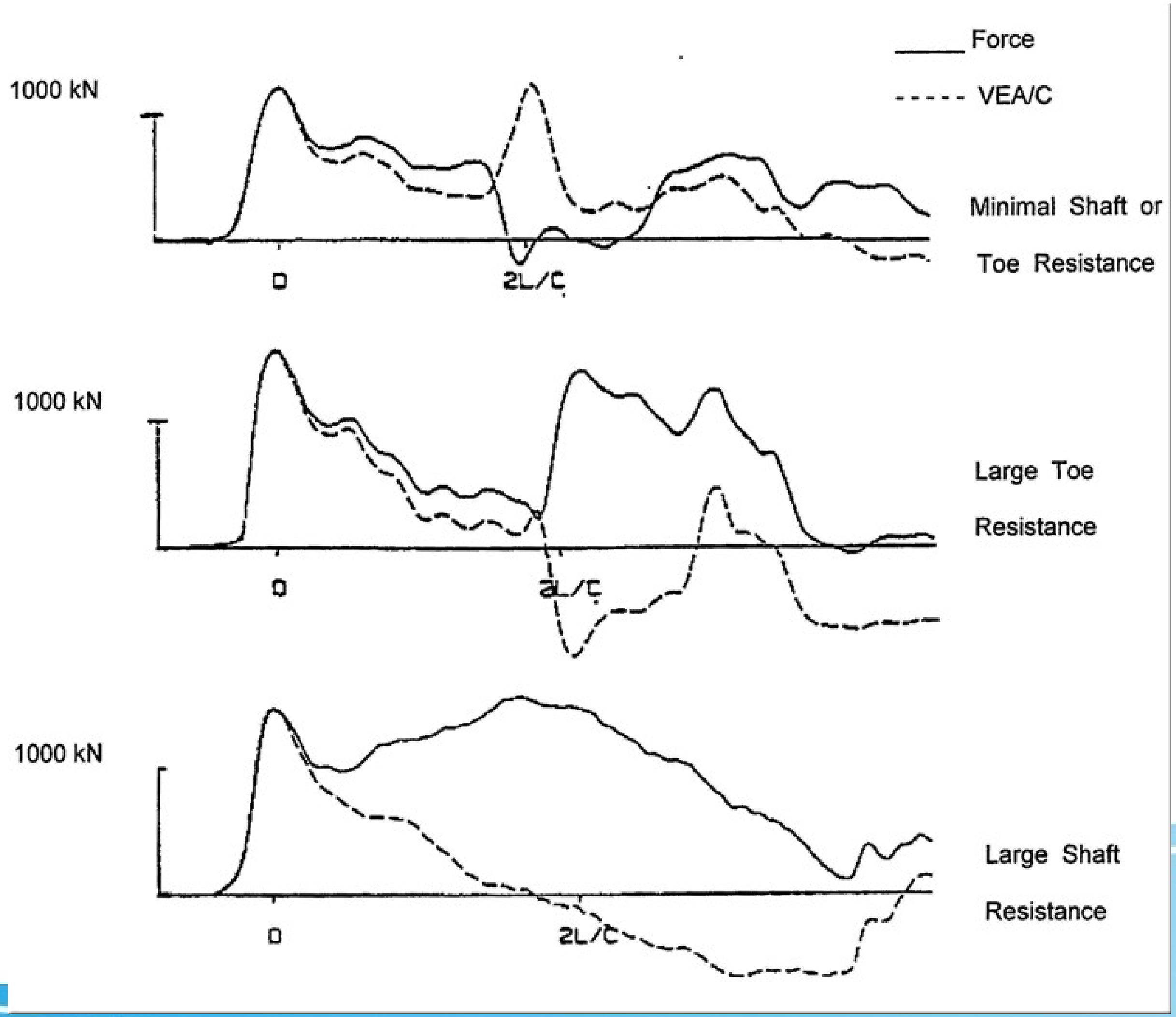
- RMX Method
  - Value for Case Damping Constant J
    - The Case damping constant for the RMX method is generally greater than the one used for RSP, typically by +0.2, and should be at least 0.4.
    - In this case we will assume J = 0.7.
- Substituting and Solving
  - RTL = (819 + 1486)/2 + (731.52 – 0)/2 = 1518 kN
  - RD = 0.7(819 + 731.52 – 1518) = 16 kN
  - RS = 1518 – 16 = 1498 kN

Determining Case Damping Constant

Soil Type at Pile Toe	Original Case Damping Correlation Range Goble *et al.* (1975)	Updated Case Damping Ranges Pile Dynamics (1996)
Clean Sand	0.05 to 0.20	0.10 to 0.15
Silty Sand, Sand Silt	0.15 to 0.30	0.15 to 0.25
Silt	0.20 to 0.45	0.25 to 0.40
Silty Clay, Clayey Silt	0.40 to 0.70	0.40 to 0.70
Clay	0.60 to 1.10	0.70 or higher

The reality is that the Case damping constant is a "job-specific" quantity which can and will change with changes of soil, pile and even pile hammer. These require calibration, either with CAPWAP or theoretically with the wave equation program. The Case Method requires a great deal of experience and judgment in its application to actual pile driving situations.

Interpreting Force-Time Curves



Dynamic Pile Testing

Critique

- *Quick and inexpensive*
- *Can test all types of preformed piles (concrete, steel and timber) and drilled shafts with well defined geometry*
- *No special preparation required*

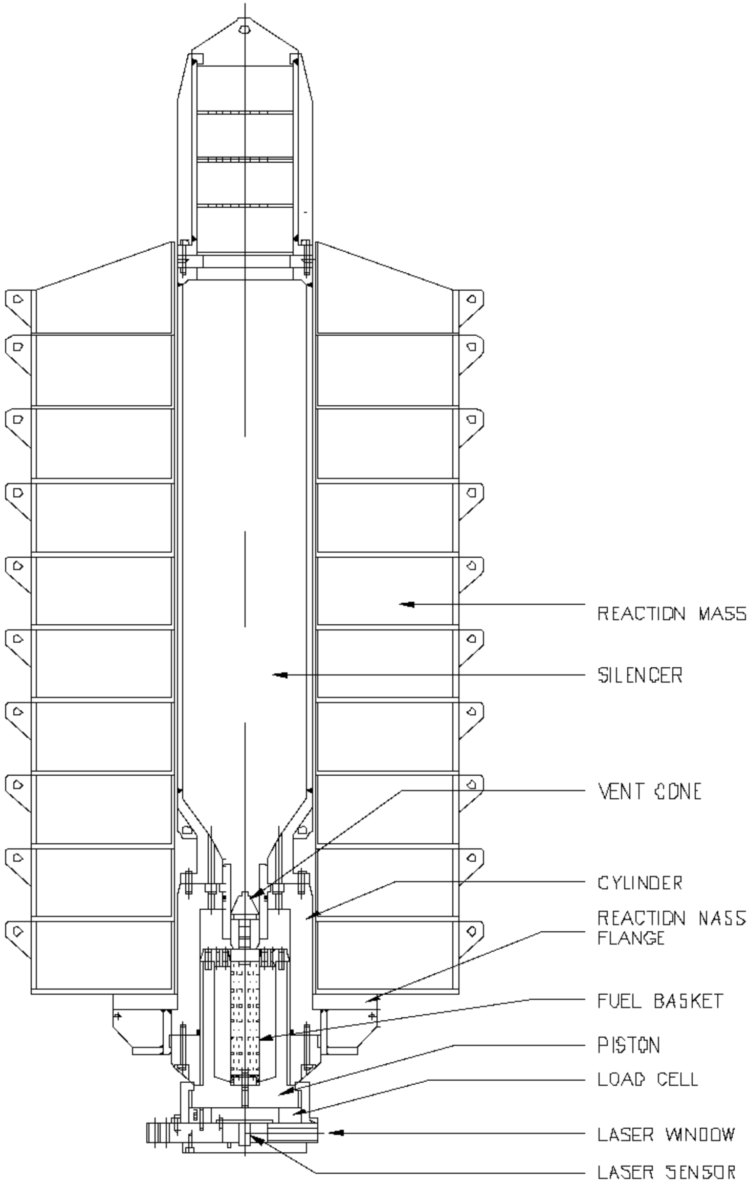
Static capacity is interpreted rather than measured directly

Requires experience for correct interpretation

Statnamic Tests

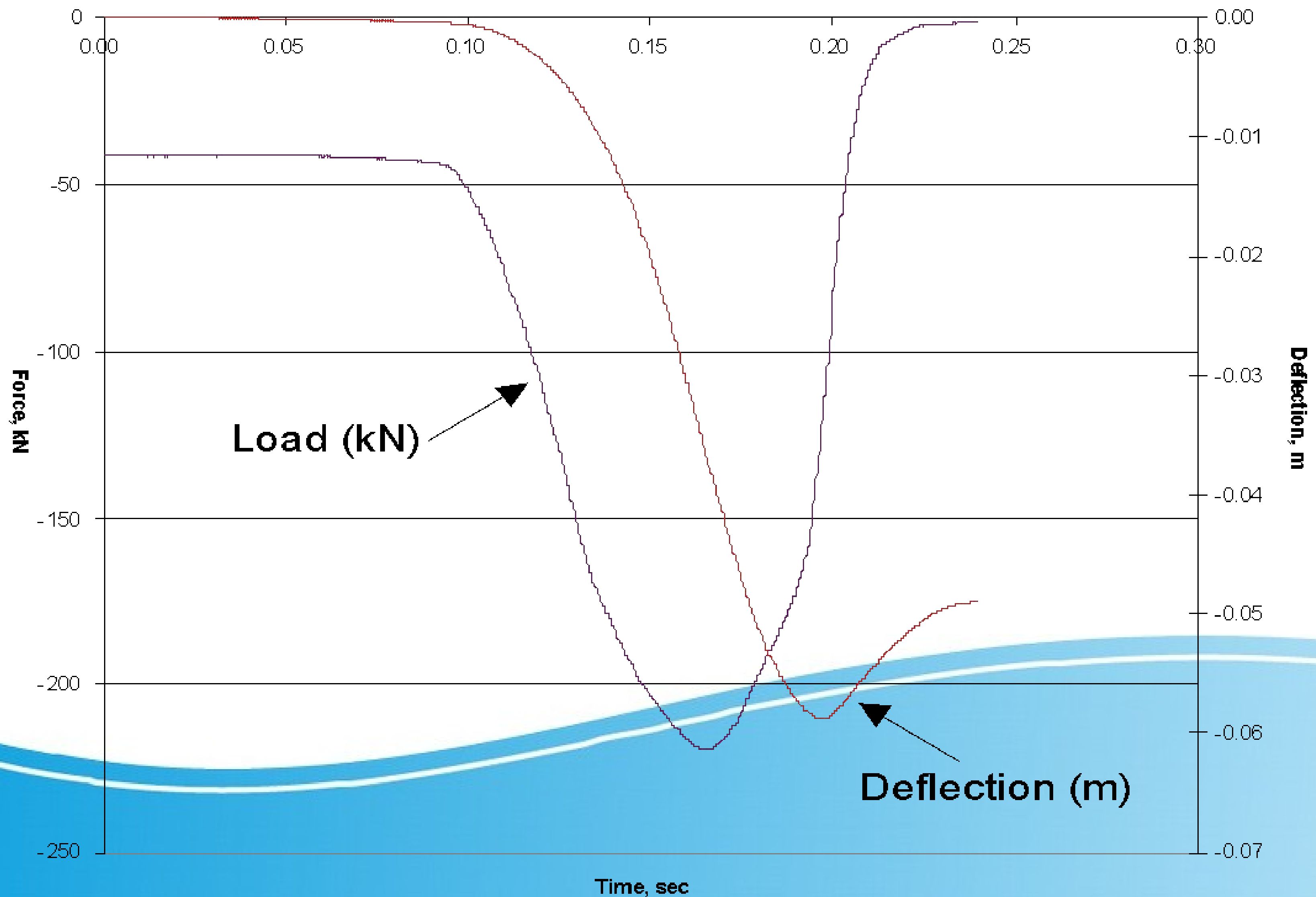
http://www.youtube.com/watch?v=IHnbd-QGdaw

Statnamic Device and Principles

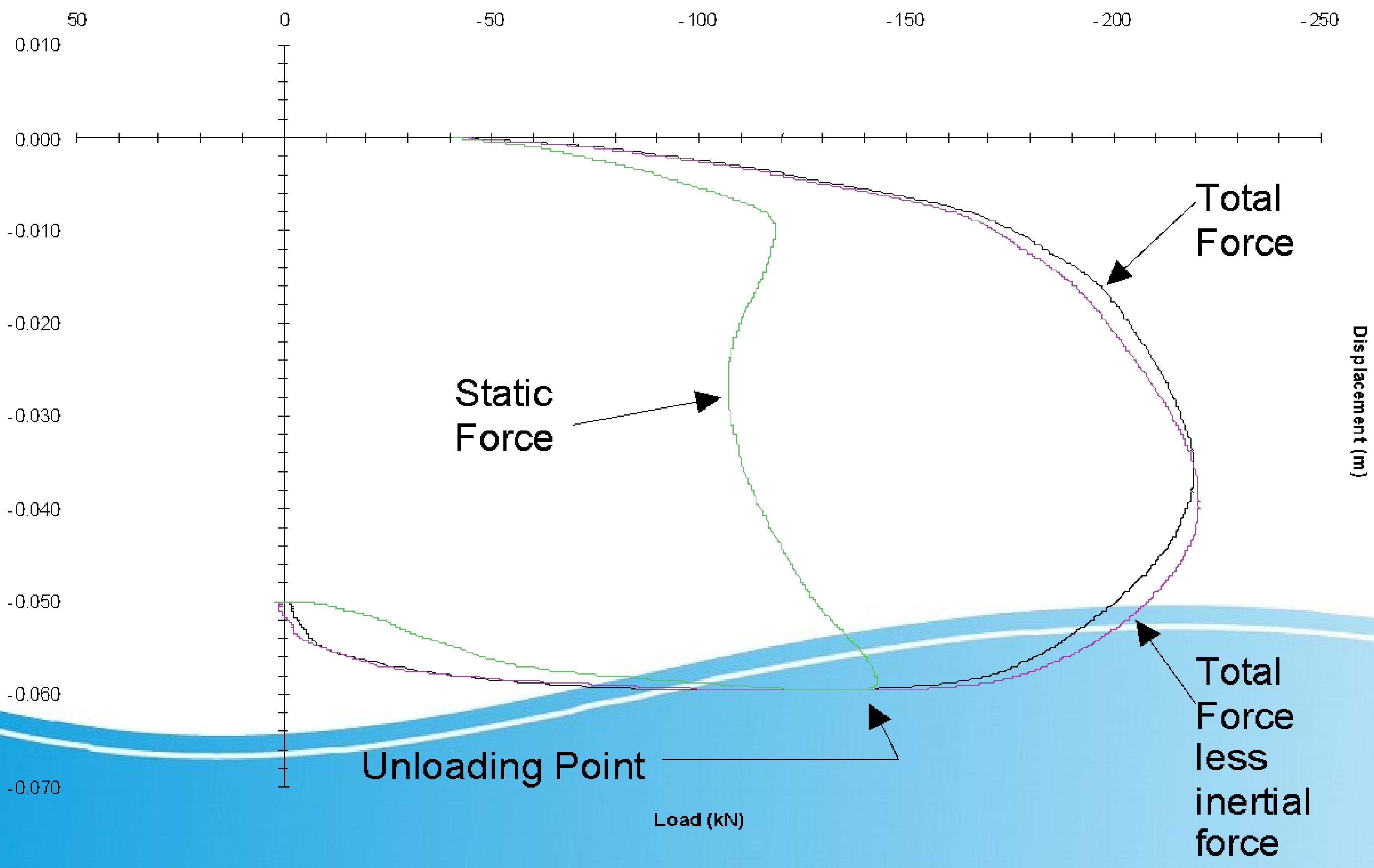


- Controlled explosion detonated; loads the pile over longer period of time than impact dynamic testing
- Upward thrust transferred to reaction weights
- Laser sensor records deflections; load cell records loads

Typical Statnamic Force-Time Curves



Typical Statnamic Load-Deflection Curves



Statnamic Advantages and Disadvantages

- Advantages
  - Much faster and simpler than static load testing
  - Does not require a pile hammer as high-strain dynamic testing does
  - Especially applicable to drilled shafts and other bored piles
- Disadvantages
  - Does not give a clear picture of the distribution of capacity between the shaft and the toe
  - Technique not entirely developed for clay (high dampening) soils

TABLE 12
Identification and Characteristics of Special Materials

Material	Geographic/Geomorphic Features	Engineering Conditions
"Quick Clay"	• Marine or brackish water clay composed of glacial rock flour that is elevated above sea level. • Generally confined to far north areas; Eastern Canada, Alaska, Scandinavia.	• Severe loss of strength when disturbed by construction activities or seismic ground shaking. • Replacement of formation water containing dissolved salt with fresh water results in strength loss. • Produces landslide prone areas (Anchorage, Alaska).
Hydraulic Fills	• Coastal facilities, levees, dikes, tailings dams	• High void ratio • Uniform gradation but variable grain sizes within same fill • High liquefaction potential • Lateral spreading • Easily eroded
Collapsing Soil	• Desert arid and semi-arid environment • Alluvial valleys, playas, loess	• Loss of strength when wetted • Differential settlement • Low density • Moisture sensitive • Gypsum/Anhydrite often present

TABLE 12 (continued)
Identification and Characteristics of Special Materials

Material	Geographic/Geomorphic Features	Engineering Conditions
Submarine Soils	• Continental shelf deposits at water depths up to several hundred feet. • Submarine canyons, turbidity flows, deltaic deposits, abyssal plain	• Distribution and physical properties of sand, silt, and clay may change with time and local geologic conditions. • Shelf deposits have few unique characteristics requiring modification of soil mechanics principals. • Local areas, such as the Gulf of Mexico have weak, underconsolidated deposits. • Deep sea calcareous deposits have water contents up to 100% and shear strengths up to about 220 psf. • Deep sea silty clays have average water contents of 100-200% and shear strengths of 35-75 psf. • Deep sea deposits are normally consolidated but near shelf deposits may be underconsolidated.
Lateritic Soils	• Tropical rainforest and savanna • Deep residual soil profile • Shield and sedimentary cover outside shield in South and Central America, Central and West Africa, southeast Asia, and other parts of the world.	• Loss of soil strength with time • High void ratio/permeability • Aggregate deterioration • Variable moisture content • Shrinkage cracks

TABLE 12 (continued)
Identification and Characteristics of Special Materials

Material	Geographic/Geomorphic Features	Engineering Conditions
Lateritic Soils (cont'd)		• Easily compacts • Shear characteristics somewhere between sand and silt • Landslide prone • Depth of wetting affects slope stability • Varied foundation conditions
Limestone and Coral	• Humid tropics and subtropics, island environment. • Karst topography accelerated in humid climates. • Limestone that are cavernous or prone to cavity formations are widely distributed throughout the world in countries of arid and humid climates. In the U. S., cavernous limestone is found in Kentucky, Pennsylvania, California, Indiana, Michigan, New Mexico, Texas, and Virginia.	• Solution cavities • Extreme variations in porosity • Void ratios in coral up to 2 • Chimney-like sinkholes and collapse structures • Slump failures, ravelling • Rock settlement and consolidation • Piles or bridging often required

Overview of Collapsible Soils

5.7.2 Collapse Potential of Soils

There are several types of soils that can experience collapse under moisture ingress. Examples of such soils are wind-blown deposits such as loess, or alluvial soils deposited in arid or semi-arid environments where evaporation of soil moisture takes place at such a rapid rate that the deposits do not have time to consolidate under their self weight or where the deposits are cemented by precipitated salts. Such soils are predominantly composed of silts and some clay. Typically, the structure of such soils is flocculated and the soil particles are held together by "clay bridges" or some other cementing agent such as calcium carbonate. In both cases disturbed samples obtained from these deposits are generally classified as silt. When dry or at low moisture content the in-situ material gives the appearance of a stable deposit. At elevated moisture contents these soils generally undergo sudden changes in volume and collapse. **Full saturation is not required to realize collapse of such soils; often collapse of the soil structure occurs at moisture contents corresponding to pre-collapse degrees of saturation between 50 to 70%.** Such soils, unlike other non-cohesive soils, will stand on almost a vertical slope until inundated. Collapse-susceptible soils typically have a low relative density, a low unit weight and a high void ratio. Figure 5-28 is a useful tool for assessing whether a soil is collapsible or not based on LL and dry unit weight.

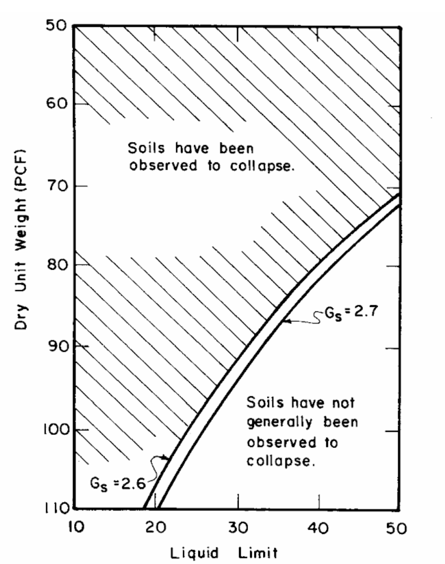


Figure 5-28. Chart for evaluation of collapsible soils (after Holtz and Hilf, 1961).

Structures founded on such soils may be seriously damaged if the soils are inundated and collapse. Therefore, if a soil is suspected to be collapse susceptible, then it is of primary importance to estimate the magnitude of potential collapse that may occur if the soil becomes wetted. To do this, a one-dimensional collapse potential test can be performed in an oedometer on undisturbed or recompacted samples according to ASTM D 5333. For this test, a sample is placed in an oedometer at it natural or compacted moisture content and the vertical pressure on the sample is increased in increments to the anticipated final loading in the field. Readings of vertical deformation are taken during the loading sequence. At the anticipated final load level, water is introduced to the sample and the resulting deformation due to collapse is recorded. The percent collapse (%C) is defined as:

$$\%C = \frac{100\,\Delta H_c}{H_o} \qquad \text{5-15}$$

where ΔH_c is the change in height upon wetting and H_o is the initial height of the specimen. Conceptually, C is a strain. Therefore, for a soil layer with a given thickness, H, the settlement due to collapse, $s_{collapse}$, if the entire thickness is inundated may be calculated as:

$$s_{collapse} = H\left(\frac{(\%C)}{100}\right) \qquad \text{5-16}$$

The collapse potential (CP) is calculated as the percent collapse (%C) of a soil specimen subjected to a total load of 4 ksf (200 kPa) as measured by using procedures specified in ASTM D 5333. The CP is an index value used to compare the susceptibility of various soils to collapse. Table 5-12 provides a relative indication of the degree of severity for various values of CP.

Table 5-12
Qualitative assessment of collapse potential (after ASTM D 5333)

Collapse Potential (CP)	Severity of Problem
0	None
0.1 to 2%	Slight
2.1 to 6%	Moderate
6.1 to 10%	Moderately Severe
>10%	Severe

Dealing With Collapsible Soils

- Solutions for Pavements
  - Ponding water over the region of collapsible soils.
  - Infiltration wells.
  - Compaction - conventional with heavy vibratory roller for shallow depths (within 0.3 or 0.6 m (1 or 2 ft))
  - Compaction - dynamic or vibratory for deeper deposits of more than half a meter (a few feet) (could be combined with inundation)
  - Excavated and replaced.
- Other Foundation Solutions
  - Shallow Foundations—furnish a system of grade beams to distribute the load and mitigate the effects of uneven collapse of the underlying soils
  - Deep foundations—avoid the effects of collapsible soils altogether by transferring the structure load to a more stable stratum

Expansive Soils

2. EXPANSIVE SOILS.

a. Characteristics. Expansive soils are distinguished by their potential for great volume increase upon access to moisture. Soils exhibiting such behavior are mostly montmorillonite clays and clay shales.

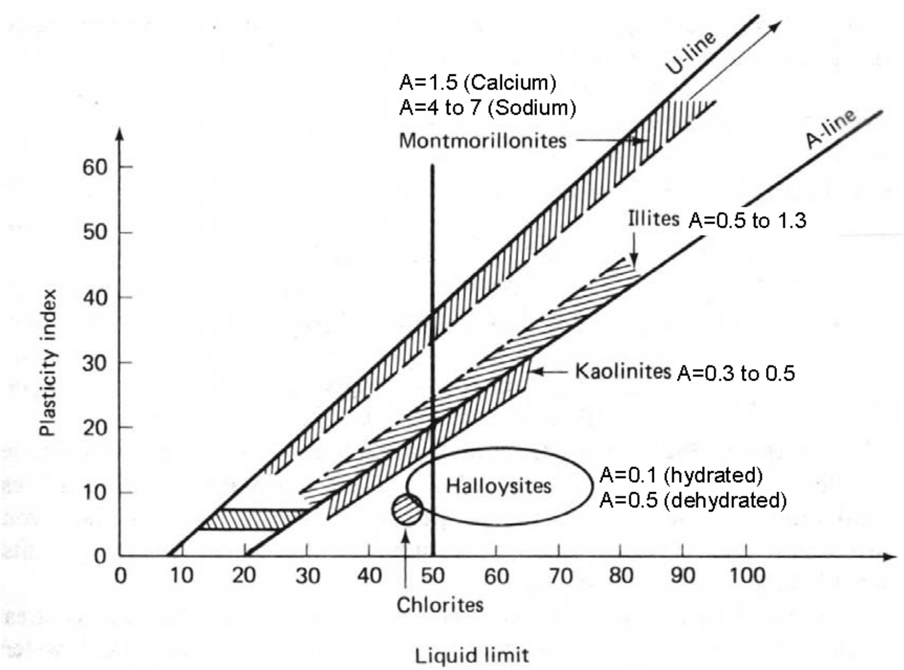
b. Identification and Classification. Figure 4 (Reference 17, Shallow Foundations, by the Canadian Geotechnical Society) shows a method based on Atterberg limits and grain size for classifying expansive soils. Activity of clay is defined as the ratio of plasticity index and the percent by weight finer than two microns (2μ). The swell test in a one dimensional consolidation test (see Chapter 3) or the Double Consolidometer Test (Reference 18, The Additional Settlement of Foundations Due to Collapse of Structures of

●Activity Index

Another useful index proposed by Skempton (1953) based on the proportion of clay and PI is known as the "Activity Index." The activity index of a clay soil is denoted by A and is generally defined as follows:

$$A = \frac{PI}{CF} \qquad 5\text{-}3$$

where CF is the clay fraction is usually taken as the percentage by weight of the soil with a particle size less than 0.002 mm. Clays with $0.75 < A < 1.25$ are classified as "normal" clays while those with $A < 0.75$ are "inactive" and $A > 1.25$ are "active." Values of activity index, A, can be correlated to the type of clay mineral that, in turn, provides important information relative to the expected behavior of a clay soil. A clay soil that consists predominantly of the clay mineral montmorillonite behaves very differently from a clay soil composed predominantly of kaolinite. Figure 5-5 also shows the activities of various clay minerals and their location on the Casagrande's plasticity chart. The symbol for the activity index (A) in Figure 5-5 should not be confused with the "A-line" also shown in the figure.



Activity Index, A, of other minerals			
Attapulgite	0.5-1.2	Mica (muscovite)	0.2
Allophane	0.5-1.2	Quartz	0
Calcite	0.2		

Figure 5-5. Location of clay minerals on the Casagrande Plasticity Chart and Activity Index values (after Skempton, 1953, Mitchell, 1976, Holtz and Kovacs, 1981).

<u>Modified Activity Index, A_m</u>: Based on their studies regarding the swell potential of compacted natural and artificial clay soils, Seed *et al.* (1962) proposed that for natural clay soils compacted as per the requirements of ASTM D 698 and Atterberg limits determined by ASTM D 4318 (AASHTO T 89, T 90), a Modified Activity Index, A_m, defined as follows is more appropriate:

$$A_m = \frac{PI}{CF - 5} \qquad 5\text{-}4$$

The above definition is used to define the swell potential of soils (see Section 5.7).

Regions of Expansive Soils

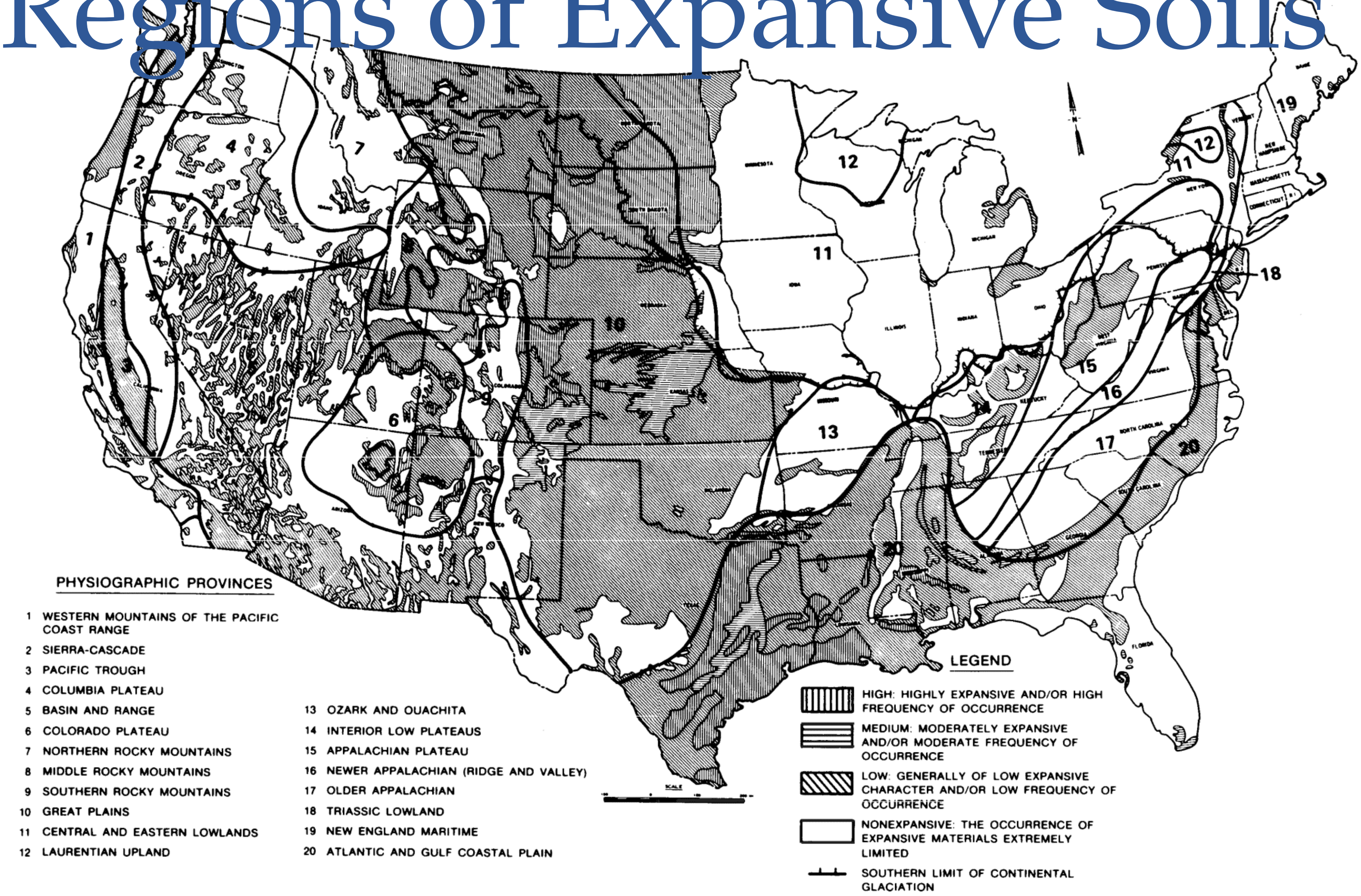


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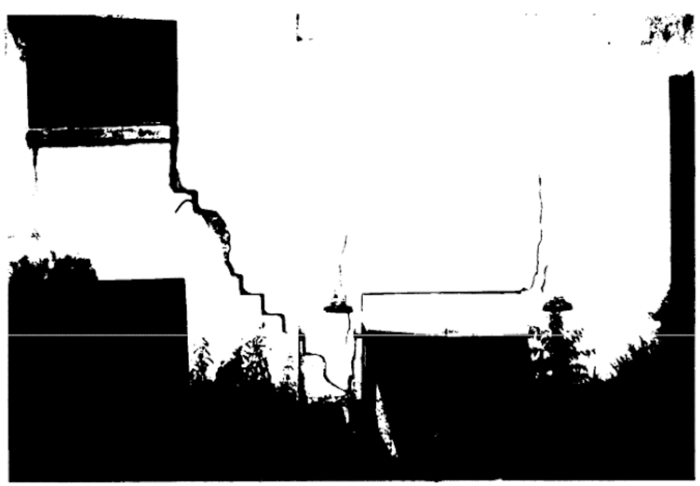
Figure 2-1. Occurrence and distribution of potentially expansive materials in the United States, 1977, with boundaries of physiographic provinces.

Damage Due to Expansive Soils

a. Vertical cracks



b. Diagonal and vertical cracks

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Figure 1-1. Examples of cracks in an exterior wall.

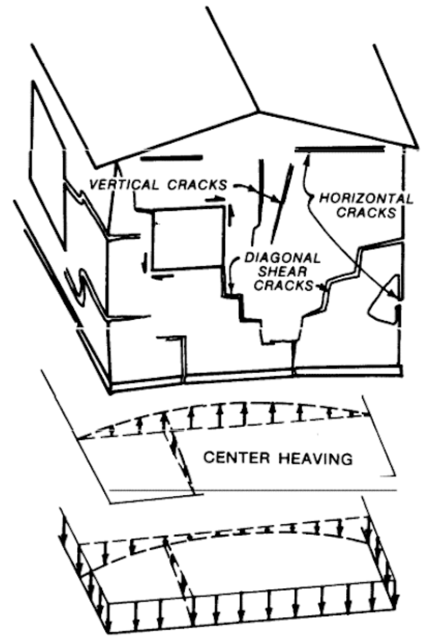

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Figure 1-2. Examples of wall fractures from doming heave of swelling and shrinking foundation soils.

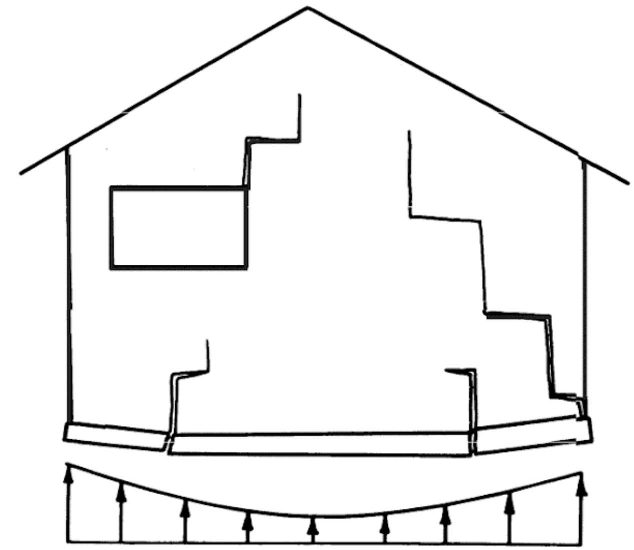

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Figure 1-3. Examples of fractures from dish-shaped lift on swelling foundation soils.

Swell Potential

5.7 VOLUME CHANGE PHENOMENA DUE TO LOADING AND MOISTURE

Depending on mineralogy and depositional patterns, natural soils can exhibit either swell (expansion) or collapse under various degrees of loading and moisture ingress. Moisture may be in liquid or frozen form. For foundation design, it is very important to recognize and evaluate the potential for soils to swell or collapse. It is important to realize that these phenomena happen in both natural and compacted soils. Every year millions of dollars are spent dealing with the consequences of swelling (expanding) and collapsing soils. This section briefly discusses these two mechanisms and the tests that can be performed to evaluate the swell (expansion) and collapse potentials.

5.7.1 Swell Potential of Clays

Swelling is a characteristic reaction of some clays to water ingress. The potential for swell depends on the mineralogical composition of the soil fines. While montmorillonite (smectite) exhibits a high degree of swell potential, illite has no to moderate swell potential, and kaolinite exhibits almost none. The percentage of volumetric swell of a soil depends on the amount and type of clay, its relative density, the compaction moisture content and dry density, permeability, location of the groundwater table, the presence of vegetation and trees, overburden pressure, etc. Expansive soils are found throughout the U.S., however, damage caused by expansive clays is most prevalent in certain parts of California, Wyoming, Colorado, and Texas, where the climate is considered to be semi-arid and periods of intense rainfall are followed by long periods of drought. This pattern of wet and dry cycles results in periods of extensive near-surface drying and desiccation crack formation. During intense precipitation, water enters the deep cracks causing the soil to swell; upon drying, the soil will shrink. This weather pattern results in cycles of swelling and shrinking that can be detrimental to the performance of pavements, slabs on-grade, and retaining walls built on or in such soils.

Deep-seated volume changes in expansive soils are rare. More common are volume changes within the upper 3-10 feet (1- 3 m) of a soil deposit. These upper few feet are more likely to be affected by seasonal moisture content changes due to climatic changes. The zone over which volume changes are most likely to occur is defined as the active zone. The active zone can be evaluated by plotting the moisture content with depth for samples taken during the wet season and for samples taken during the dry season at the same location. The depth at which the moisture content becomes nearly constant is the limit of the active zone depth, which is also referred to as the depth of seasonal moisture change. The active zone is an important consideration in foundation design. In the design of piles or drilled shafts, it is important to recognize that full side friction resistance may not be realized in this zone. As the soil undergoes cycles of shrinking and swelling, it may lose contact with the pile or shaft. Alternatively, as the soil swells, it may impose significant uplift pressures on the foundation element.

In the field, the presence of surface desiccation cracks and/or fissures in a clay deposit is an indication of expansion potential. Experience has indicated that the most problematic expansive near-surface soils are typically highly plastic, stiff, fissured, overconsolidated clays. Several classification methods are used to identify expansive soils in the laboratory. Currently, there is not a standard classification procedure; different methods are used in various locations across the U.S. Typically, methods include the use of Atterberg limits and/or clay size percentage to describe a soil qualitatively as having low, medium, high, or very high expansion potential. Generally, soils with a plasticity index less than 15 percent will not exhibit expansive behavior. For soils with a plasticity index greater than 15 percent, the clay content of the soil should be evaluated in addition to the Atterberg limits. Figure 5-27 shows the swelling potential of natural soils and soils compacted to standard Proctor procedures (ASTM D 698) as a function of modified activity index, A_m, (Equation 5-4) and clay fraction.

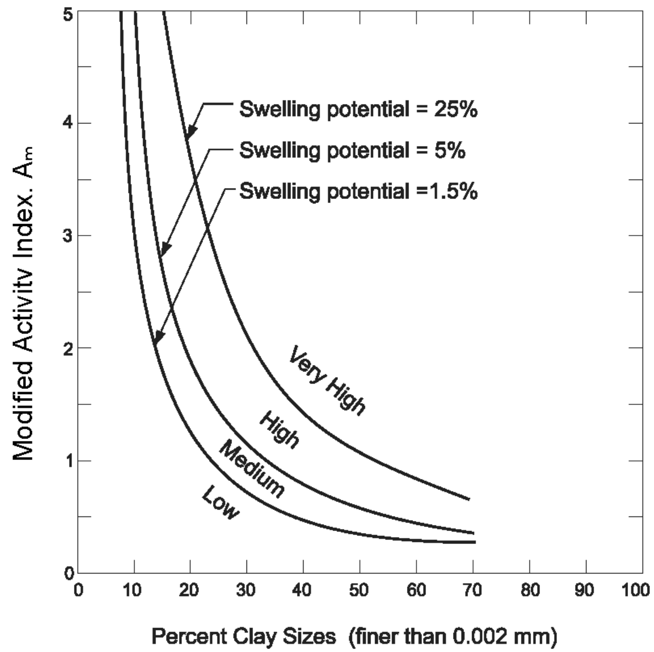

Figure 5-27. Classification of swell potential for soils (after Seed *et al.*, 1962).

●Swell Potential

5.7.1.1 Evaluation of Expansion (Swell) Potential

For situations where it is necessary to construct a facility in and around expansive soils, it will be necessary to estimate the magnitude of swell, i.e., surface heave, and the corresponding swelling pressures that may occur if the soil becomes wetted. The swelling pressure represents the magnitude of pressure that would be necessary to resist the tendency of the soil to swell. A one-dimensional swell potential test can be performed in an oedometer on undisturbed or recompacted samples according to AASHTO T256 or ASTM D 4546. In this test, the swell potential is evaluated by observing and measuring the swell of a laterally confined specimen when it is lightly surcharged and flooded with water. Alternatively, if the swelling pressure is to be measured, the height of the specimen is kept constant by adding load after the specimen is inundated. The swelling pressure is then defined as the vertical pressure necessary to maintain zero volume change. Swelling pressures in some expansive soils may be so large that the loads imposed by lightweight structures or pavements do little to counteract the swelling.

The use of the one-dimensional swell potential test to evaluate in-situ swell potential of natural and compacted clay soils has limitations including:

- Lateral swell and lateral confining pressure are not simulated in the laboratory. The calculated magnitude of swell in the vertical direction may not be a reliable estimate of soil expansion for structures that are not confined laterally (e.g., bridge abutments);

- The rate of swell calculated in the laboratory will not likely be indicative of the rate of swell experienced in the field. Laboratory tests cannot simulate the actual availability of water in the field.

It should be noted that there is a lack of a standard definition of swell potential in the technical literature based in part on variations in the test procedures, e.g., the condition of the test specimen (remolded or undisturbed), the magnitude of the surcharge, etc. Therefore the geotechnical specialist must be sure that the conditions used in the laboratory swell test simulate those expected in the field. In general, soils classified as CL or CH according to the USCS and A-6 or A-7 according to the AASHTO classification system should be considered potentially expansive.

Determination of Actual Soil Expansion

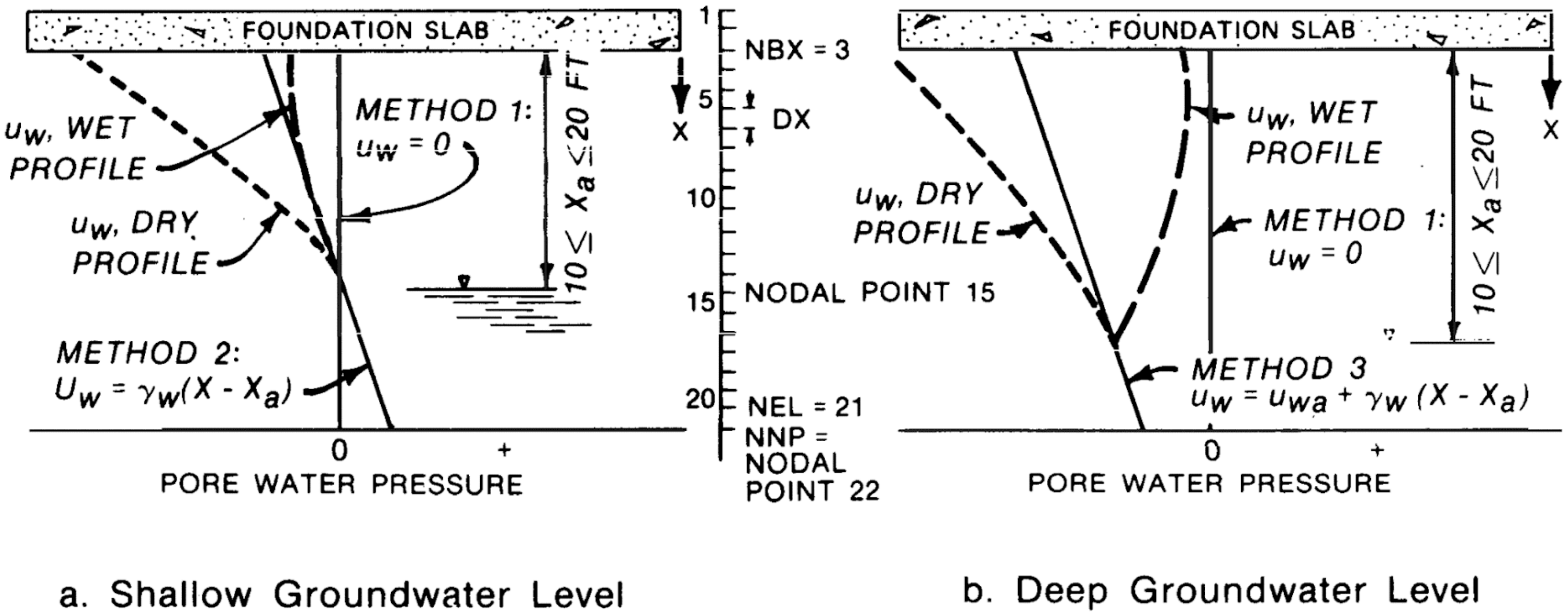

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Figure 5-1. Assumed equilibrium pore water pressure profiles beneath foundation slabs.

Basis for Calculations

a. Vertical movement. Methodology for prediction of the potential total vertical heave requires an assumption of the amount of volume change that occurs in the vertical direction. The fraction of volumetric swell N that occurs as heave in the vertical direction depends on the soil fabric and anisotropy. Vertical heave of intact soil with few fissures may account for all of the volumetric swell such that N = 1, while vertical heave of heavily fissured and isotropic soil may be as low as N = 1/3 of the volumetric swell.

b. Lateral movement. Lateral movement is very important in the design of basements and retaining walls. The problem of lateral expansion against basement walls is best managed by minimizing soil volume change using procedures described in chapter 7. Otherwise, the basement wall should be designed to resist lateral earth pressures that approach those given by

$$\delta_h = K_o \delta_v \leqslant K_p \delta_v \qquad (5\text{-}1)$$

where

- δ_h = horizontal earth pressure, tons per square root
- K_o = lateral coefficient of earth pressure at rest
- δ_v = soil vertical or overburden pressure, tons per square foot
- K_p = coefficient of passive earth pressure

The K_o that should be used to calculate δ_h is on the order of 1 to 2 in expansive soils and often no greater than 1.3 to 1.6.

a. Basis of calculation. The potential total vertical — heave at the bottom of the foundation, as shown in figure 5-1, is determined by

$$\Delta H = N \cdot DX \sum_{i=NBX}^{i=NEL} DELTA(i)$$

$$= N \cdot DX \sum_{i=NBX}^{i=NEL} \frac{e_f(i) - e_o(i)}{1 + e_o(i)} \qquad (5\text{-}2)$$

where

- ΔH = potential vertical heave at the bottom of the foundation, feet
- N = fraction of volumetric swell that occurs as heave in the vertical direction
- DX = increment of depth, feet
- NEL = total number of elements
- NBX = number of nodal point at bottom of the foundation
- DELTA(i) = potential volumetric swell of soil element i, fraction
- $e_f(i)$ = final void ratio of element i
- $e_o(i)$ = initial void ratio of element i

The ΔH is the potential vertical heave beneath a flexible, unrestrained foundation. The bottom nodal point NNP = NEL + 1, and it is often set at the active depth of heave X_a.

Van der Merwe Method

Van der Merwe's Method:

$$\Delta H = \sum_{i=1}^{n} \frac{(\Delta D)_i (F)_i}{\left(\bar{PE}\right)_i}$$

where

- ΔH =total change in height, feet or meters
- i =layer index
- n =total number of layers
- $\left(\bar{PE}\right)_i$ =inverse swell potential of layer i (see figure at right)
  - $= \infty$ (Low)
  - $= 48$ (Medium)
  - $= 24$ (High)
  - $= 12$ (Very High)
- $(\Delta D)_i$ =thickness of layer i, feet or meters
- $(F)_i$ =reduction factor for layer i
  - $= 10^{-\frac{D_i}{20}}$, D_i in feet
  - $= 10^{-\frac{D_i}{6}}$, D_i in meters
  - D_i =depth of mid-point of layer from surface

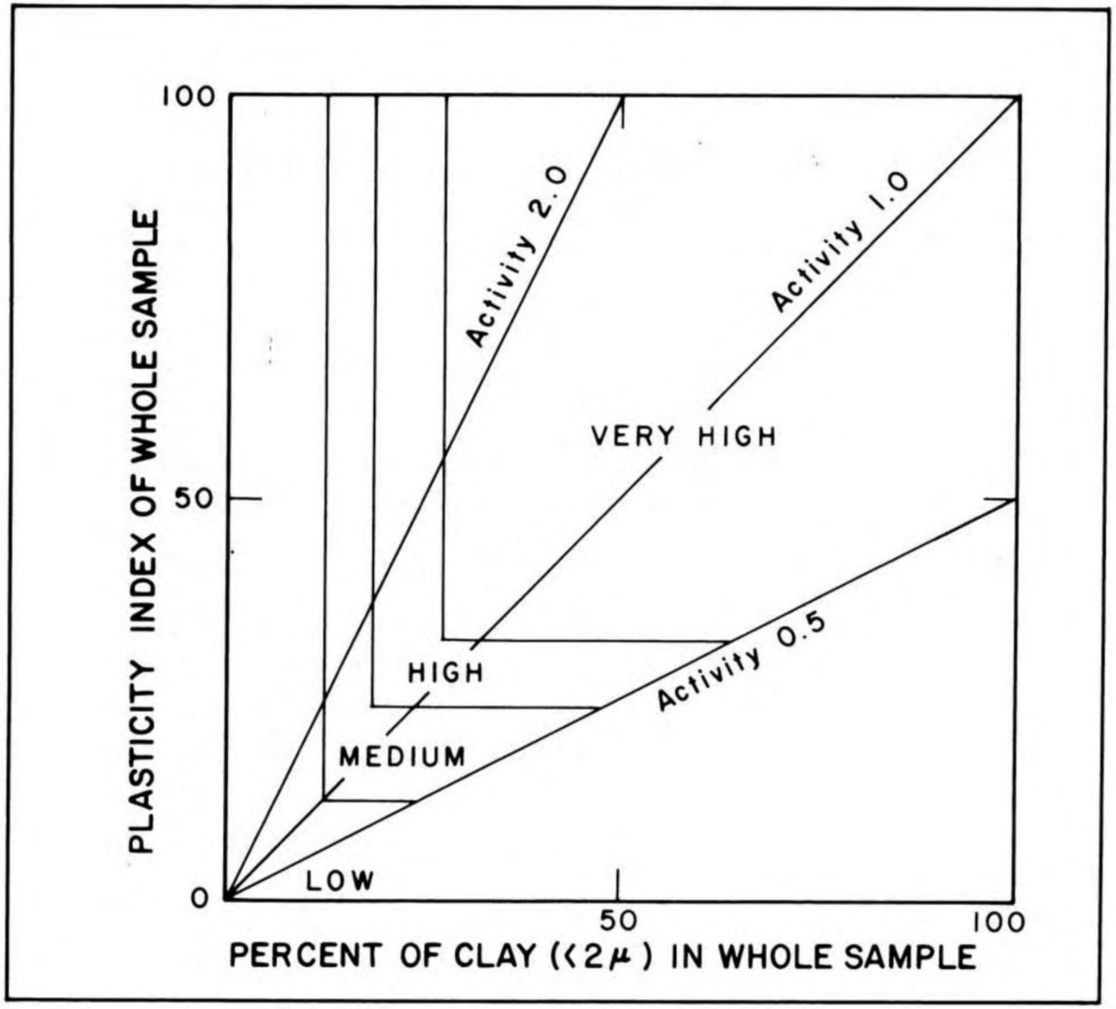


FIGURE 4
Volume Change Potential Classification for Clay Soils

●Example of Swell Calculation

- Given
  - Clay soil, PI = 40, CF = 55
- Find
  - Swell potential
  - Amount of vertical movement assuming only 1 m of soil is significant

- Solution
  - Compute Activity Indices
    - A = PI/CF = 40/55 = 0.72
    - Am = PI/(CF-5) = 40/50 = 0.8
  - Obtain swell potential
    - From Figure 5-27, High, approx. 10%
    - From van der Merwe's chart, Very High
  - Compute actual swell using van der Merwe's Method w/1 layer

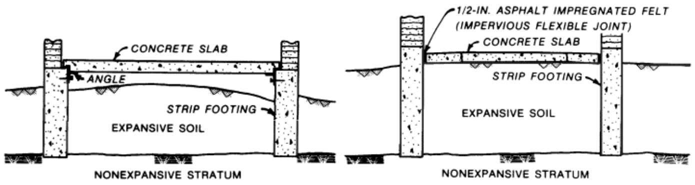
$$\Delta H = \frac{\Delta DF}{PE} = \frac{(1)\left(10^{-\frac{0.5}{6}}\right)}{12} = 0.069\,m = 69\,mm$$

Foundations for Expansive Soils

Table 6-1. Foundation Systems

Predicted Differential Movement, inches	Effective Plasticity Index, PI	Foundation System	Remarks		
1/2	<15	Shallow individual Continuous wall Strip	Lightly loaded buildings and residences.		
		Reinforced and stiffened thin mat	Residences and lightly loaded structures; on-grade 4- to 5-in. reinforced concrete slab with stiffening beams; maximum free area between beams 400 ft^2; 1/2 percent reinforcing steel; 10- to 12-in.-thick beams; external beams thickened or deepened, and extra steel stirrups added to tolerate high edge forces as needed; dimensions adjusted to resist loading. Beams positioned beneath corners to reduce slab distortion.		
			Type of Mat	Beam Depth, in.	Beam Spacing, ft
1/2 to 1	15 to 25		Light	16 to 20	20 to 15
1 to 2	26 to 40		Medium	20 to 25	15 to 12
2 to 4	>41		Heavy	25 to 30	15 to 12
No limit		Thick, reinforced mat	Large, heavy structures; mats usually 2 ft or more in thickness.		
No limit		Deep foundations, pile or drilled shaft	Foundations for any light or heavy structure; grade beams span between piles or shafts 6 to 12 in. above ground level; suspended floors or on-grade slabs isolated from grade beams and walls. Concrete drilled shafts may be underreamed or straight, reinforced, and cast in place with 3000-psi concrete of 6-in. slump.		

Shallow Foundations for Expansive Soils



a. Suspended Structural Floor (See Figures C-8 and C-9 for Details)

b. Floating Floor Slab

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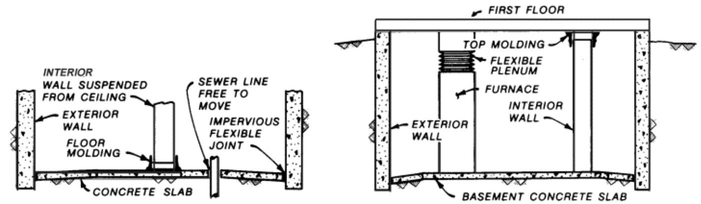
Figure 6-2. Footings on nonexpansive stratum.



a. Wall Suspended from Ceiling

b. Furnace and Interior Wall Supported on Floor

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Figure 6-3. Interior joint details for slab-on-grade.

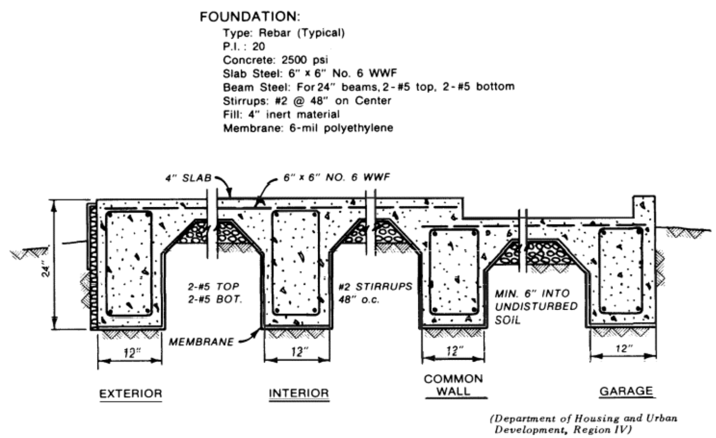


Figure 6-5. Typical conventional rebar slab in Little Rock, Arkansas, for single-family, single-story, minimally loaded frame residence with 11- to 12-foot wall spacing.

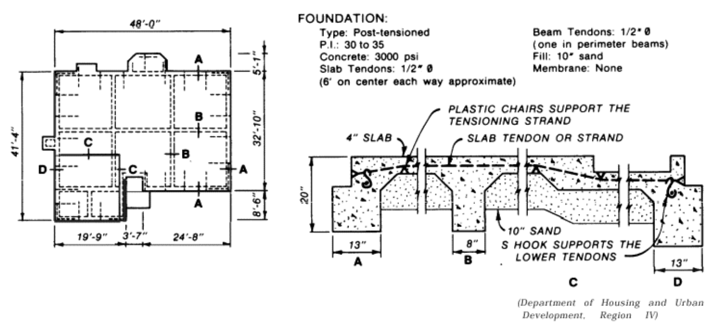
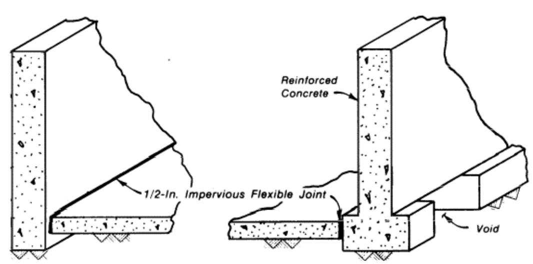


Figure 6-6. Post-tensioned slab in Lubbock, Texas, for single-family, single-story, minimally loaded frame residence.



a. Wall Without Footing

b. Wall with Footing and Void Space

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Figure 6-4. Basement walls with slab-on-grade.

Use of Drainage Techniques

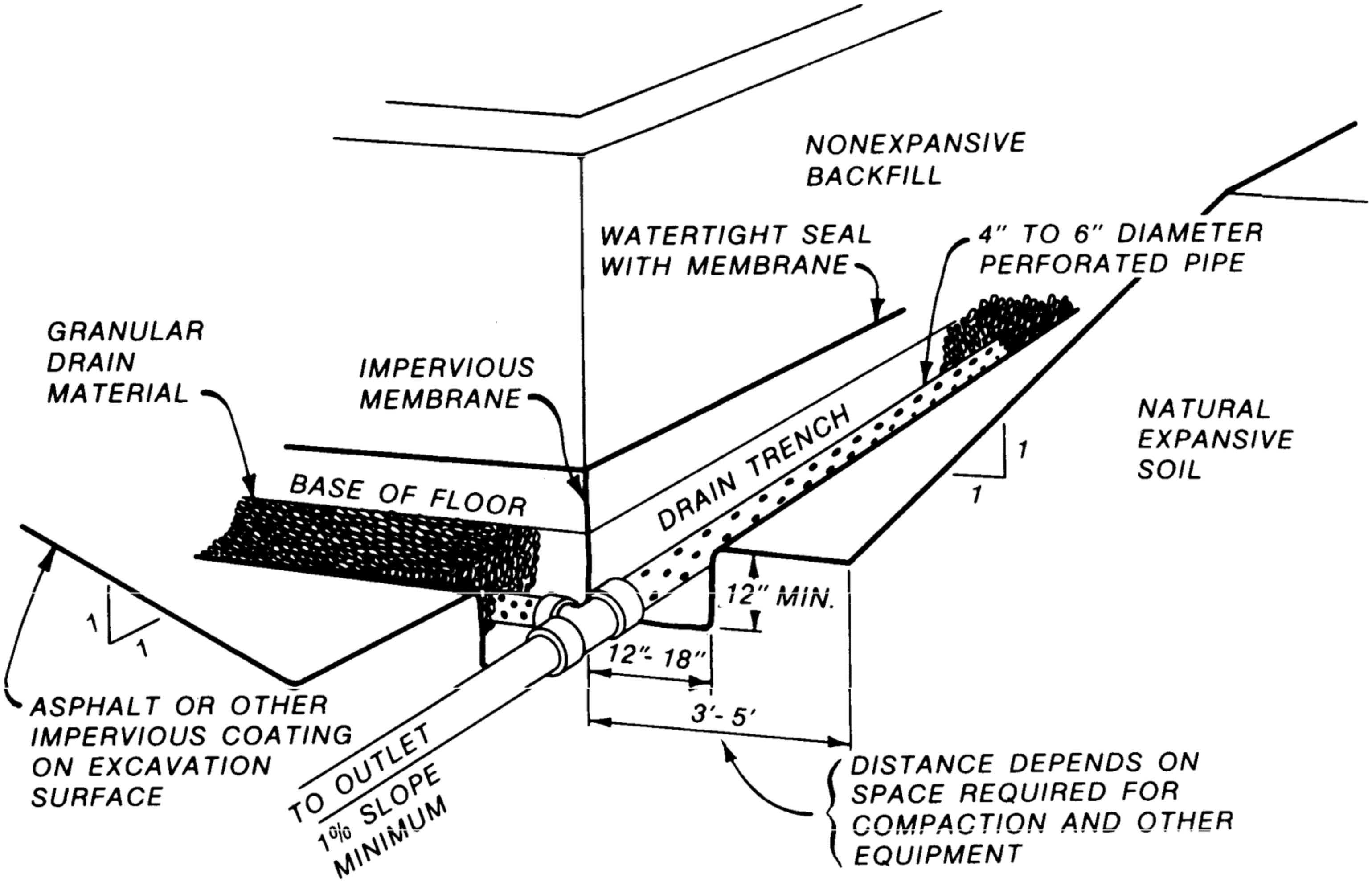

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Figure 7-1. Drainage trench around outside of structure.

Belled Bases for Drilled Shafts

- Use of an enlarged base, in conjunction with the shaft resistance of the straight portion, is a common way of dealing with expansive soils for major structures
- Enlarged bases offer additional uplift capacity
  - Only applicable to clays, since sands would usually collapse
  - Difficult to quantify

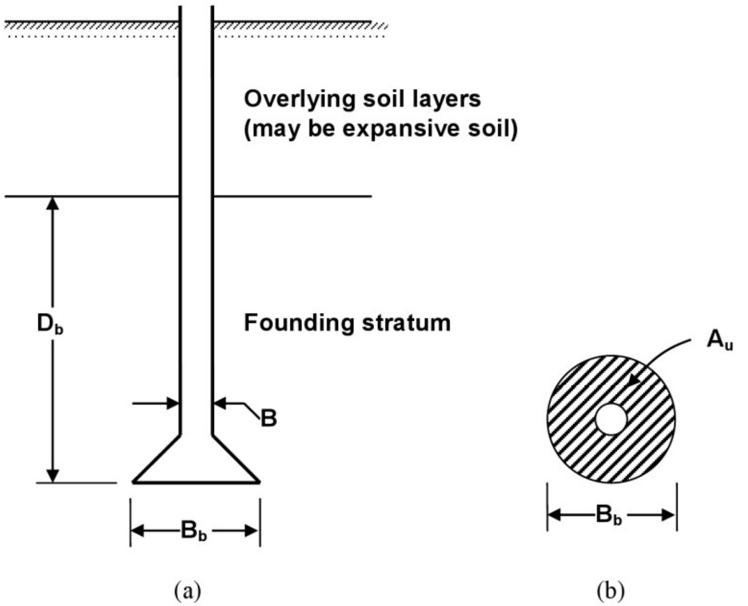


Figure C-10 (a) Geometry of Belled Shaft and (b) Projected Area of Bell in Uplift

Upward Load Capacity of Belled Shafts

$$\left(P_{upward}\right)_a = \frac{\pi\left(B_b^2 - B_s^2\right)}{4} s_u N_u$$

Unfissured Clay:

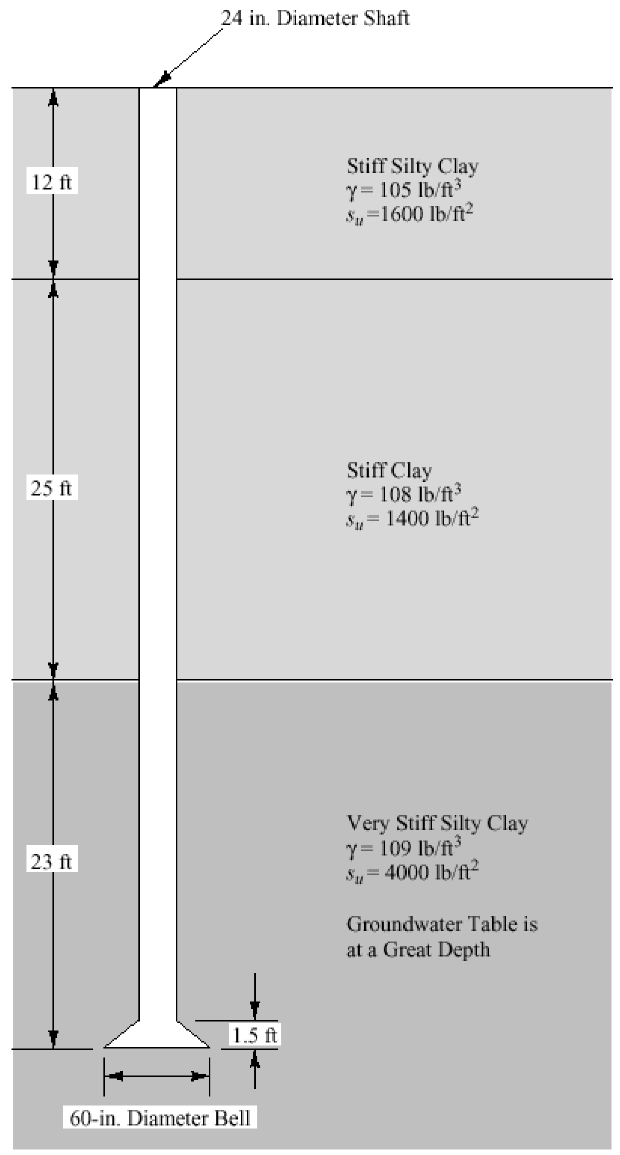
$$N_u = \frac{7D_b}{2B_b} \leqslant 9$$

Fissured Clay:

$$N_u = \frac{7D_b}{10B_b} \leqslant 9$$

- Variables for uplift capacity
  - $(P_{upward})_a$ = ultimate/unfactored upward load capacity
  - s_u = average undrained shear strength of the cohesive soil between the base of the bell and $2B_b$ above the base
  - σ_{zD} = total stress at the bottom of the base
  - B_b = diameter of the enlarged base
  - B_s = diameter of the shaft
  - D_b = depth of embedment of enlarged base into bearing stratum
- **Uplift of base only**

Example of Uplift Capacity



- Given
  - Drilled Shaft as shown
- Find
  - Uplift capacity of shaft
- Assume
  - Factor of safety = 5
  - Very stiff clay at toe is fissured
  - Include weight of foundation

Compute Shaft Friction and N_u

Layer	Layer Start	Layer End	Thickness, ft.	su, psf	alpha	fs, psf	As, sq. ft.	fsAs, kips
1	0	5	5	1600	0	0	31.42	0.00
2	5	12	7	1600	0.55	880	43.98	38.70
3	12	37	25	1400	0.55	770	157.08	120.95
4	37	52	15	4000	0.48	1920	94.25	180.96
5	52	62	10	4000	0	0	62.83	0.00
							Total Shaft Resistance	340.61

Shaft Diameter	2	feet
Perimeter	6.28	feet
Top Segment to be Ignored	5	feet
Toe Segment to be Ignored	10	feet

Different from compression
Equal to twice the diameter of the belled toe

Compute Uplift Capacity

- Compute Ultimate Uplift Capacity of Bell

$$\left(P_{upward}\right)_a = \frac{\pi\left(B_b^2 - B_s^2\right)}{4} N_c s_u$$

$$\left(P_{upward}\right)_a = \frac{\pi\left(5_b^2 - 2_s^2\right)}{4}(4)(3.29)$$

$$\left(P_{upward}\right)_a = 217 \text{ kips}$$

- Compute Weight of Foundation and Total Uplift Capacity

$$W_f = \left(\frac{\pi 2^2}{4} 58.5 + \frac{\pi\left(\frac{2+5}{2}\right)^2}{4} 1.5\right) 0.150$$

$$W_f = 34.5 \ kips$$

$$\left(P_{upward}\right)_a = \frac{34.5 + 340.6 \times 0.75 + 217}{5} = 101.4 \ kips$$

Note that we use FS=5 for uplift

Questions