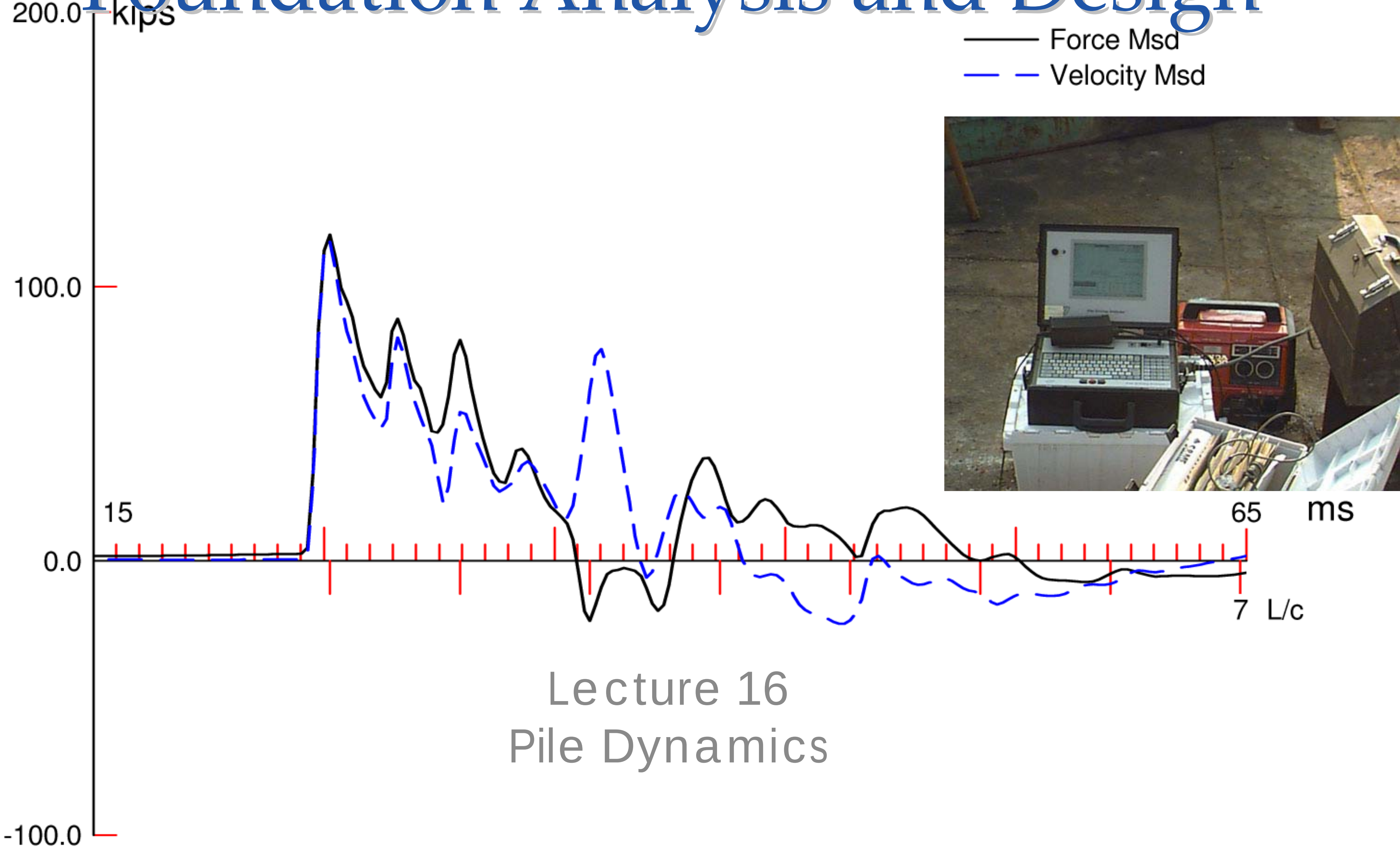
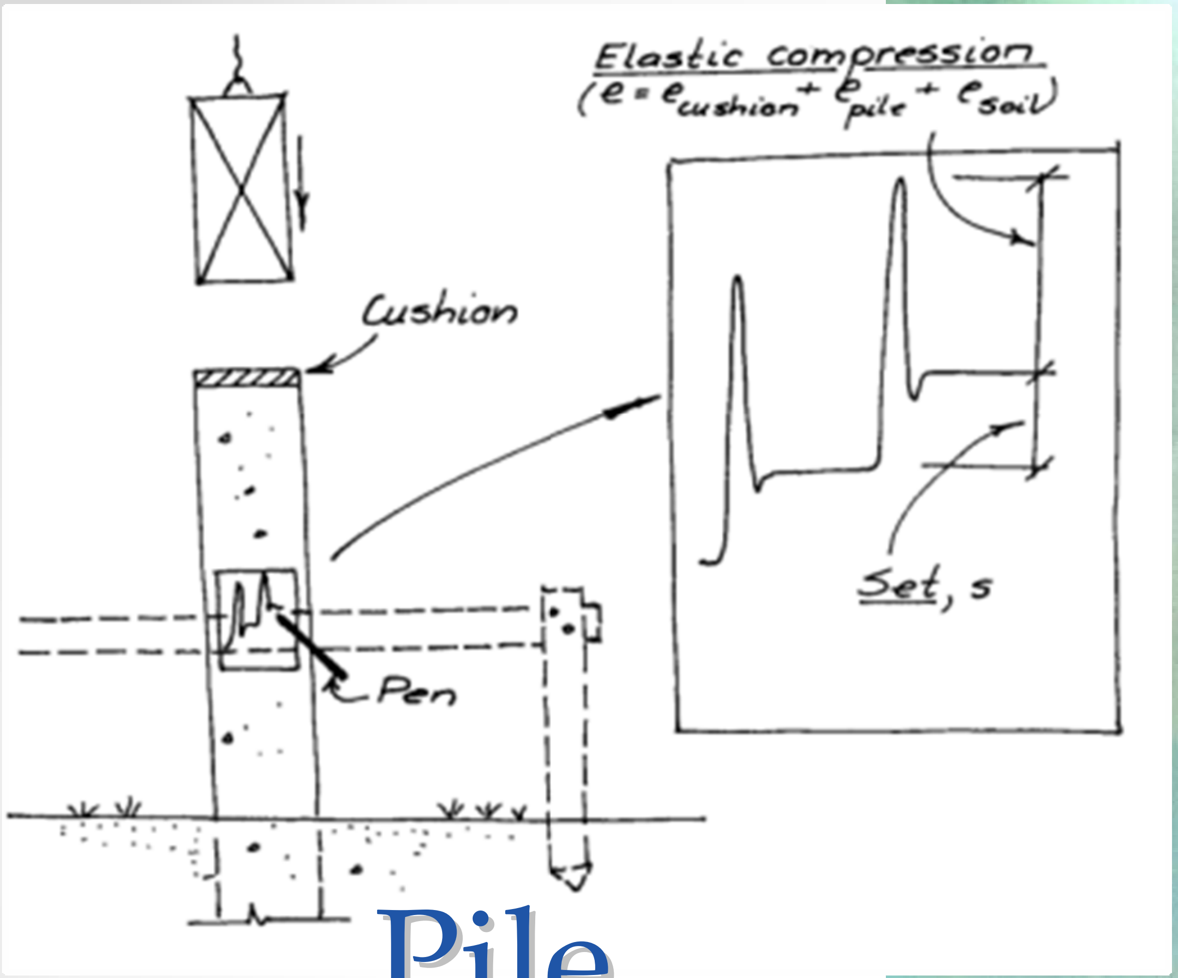
ENCE 4610
Foundation Analysis and Design



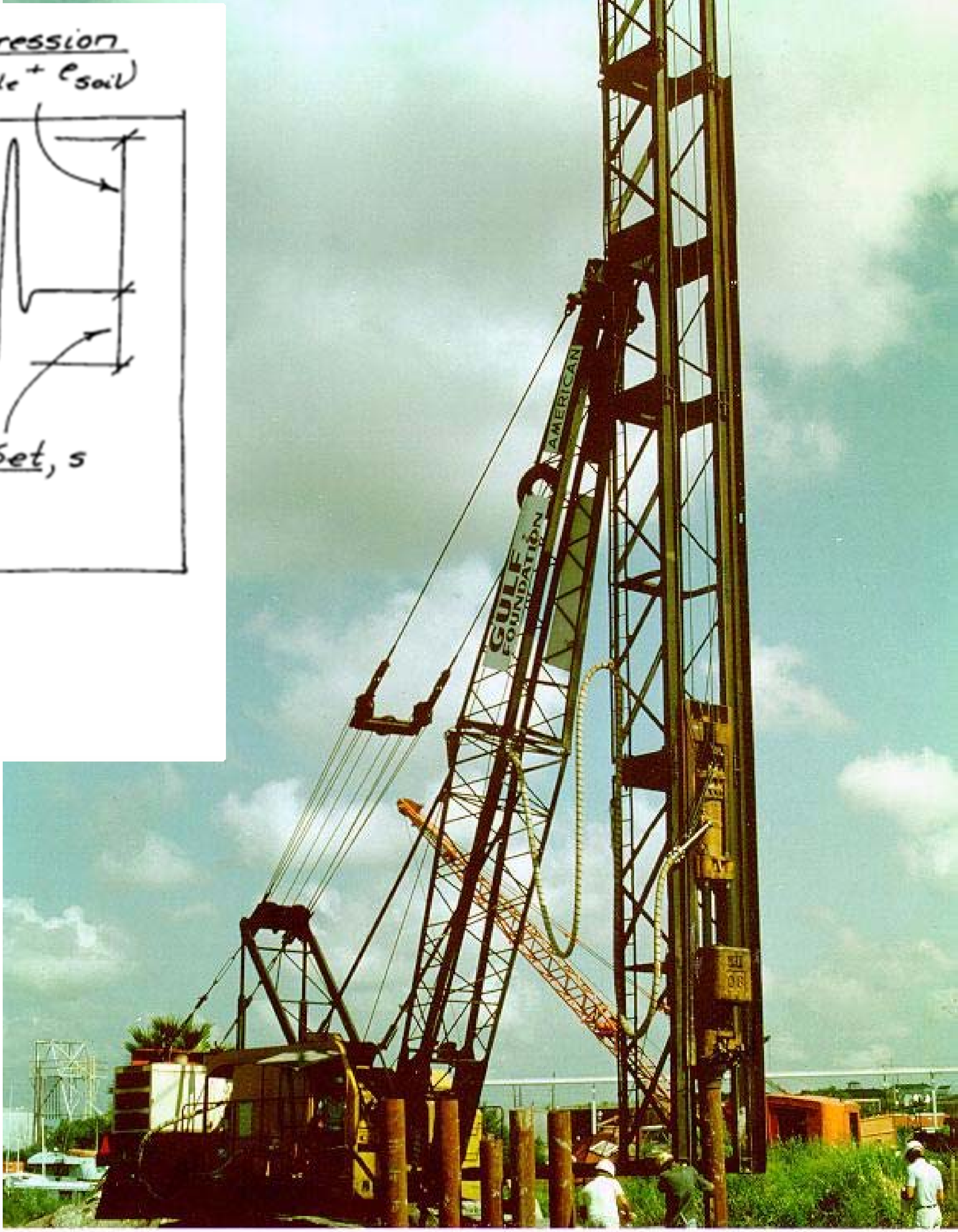
Lecture 16
Pile Dynamics

Overview of Pile Dynamics

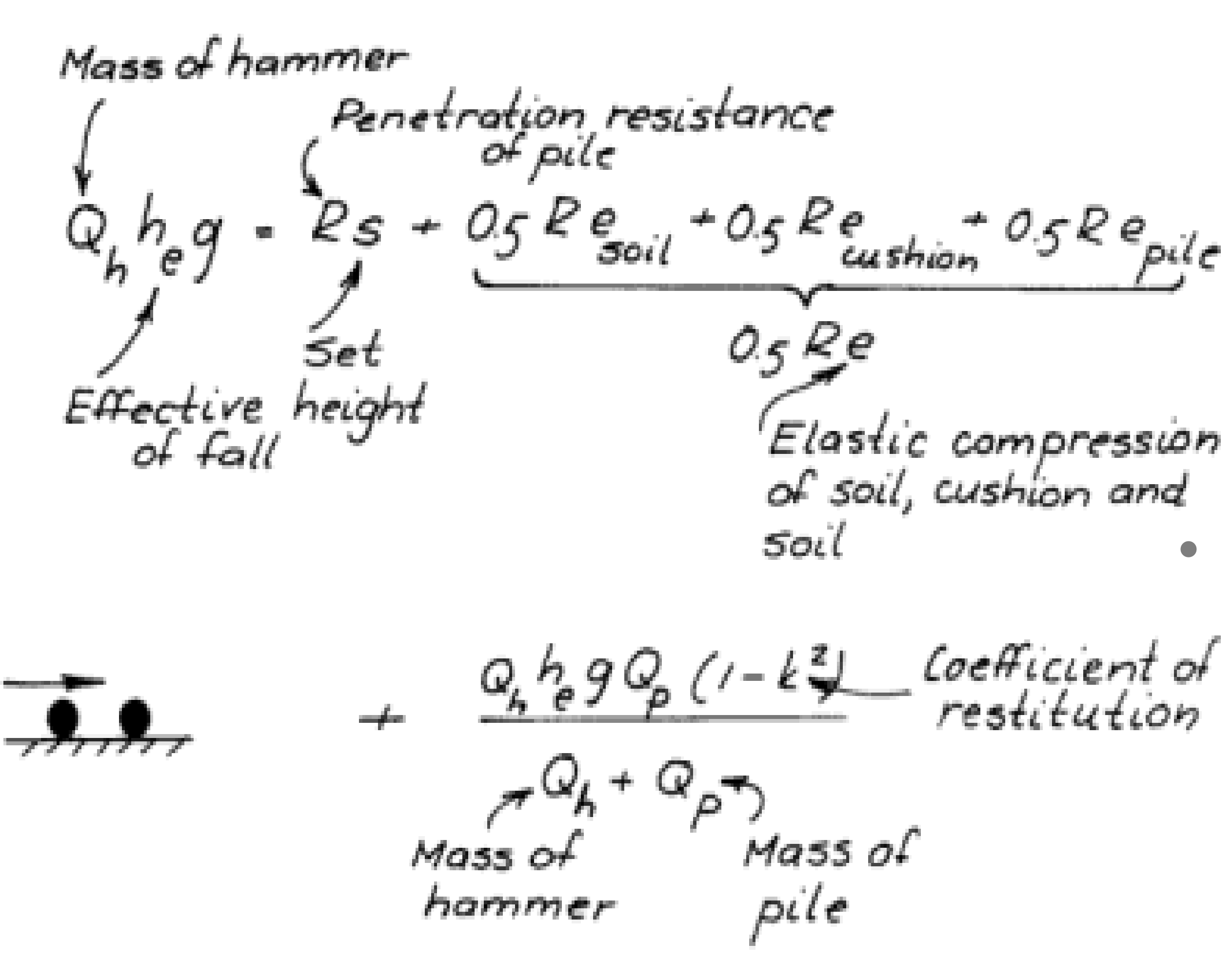
- Background and the Dynamic Formulae
- Development of the Wave Equation
- Application of the Wave Equation to Piles
- Use of the Wave Equation in field monitoring
- Statnamic Testing



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

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:

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

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):

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

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)

Weaknesses of Dynamic Formulae

- Does not take into consideration the elasticity of the pile, which is distributed with the mass
- No really accurate model of the cushion and cap system between the hammer and the pile
- Newtonian impact mechanics not applicable since the pile is in constant contact with the soil
- No ability to estimate or calculate tensile stresses in the pile

The Wave Equation

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

where

σ	=	stress	
E	=	Young's modulus	
c	=	wave propagation speed	$c = \sqrt{\frac{E}{\rho}}$
v	=	particle velocity	

- When applied to piling, must be expanded to include dampening 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:

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

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)

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

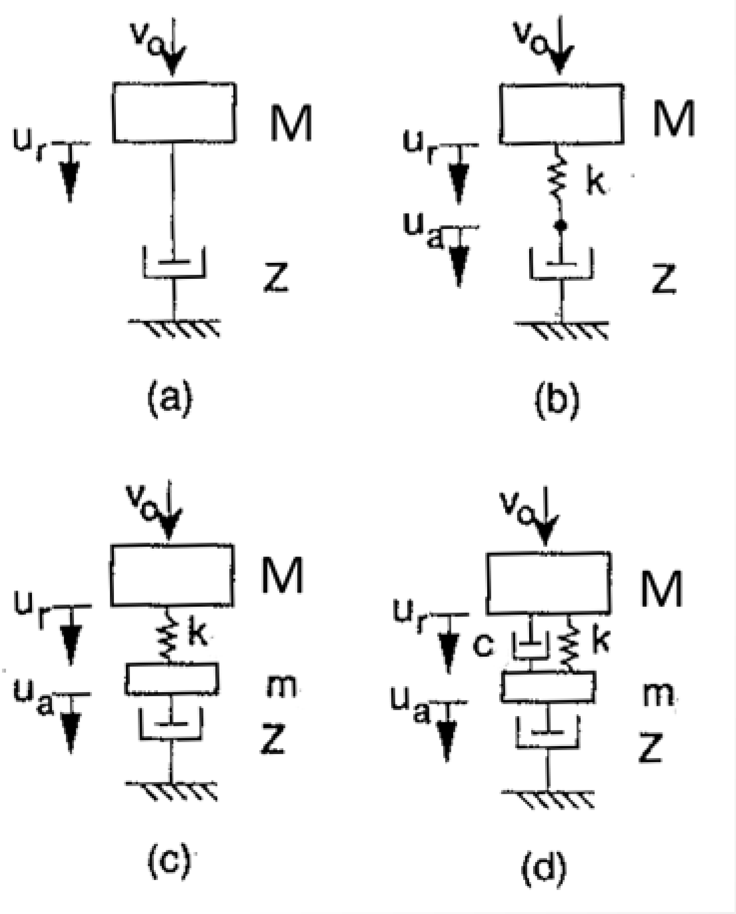
$$\sigma A = F = Z_P v_P \quad (9.5)$$

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

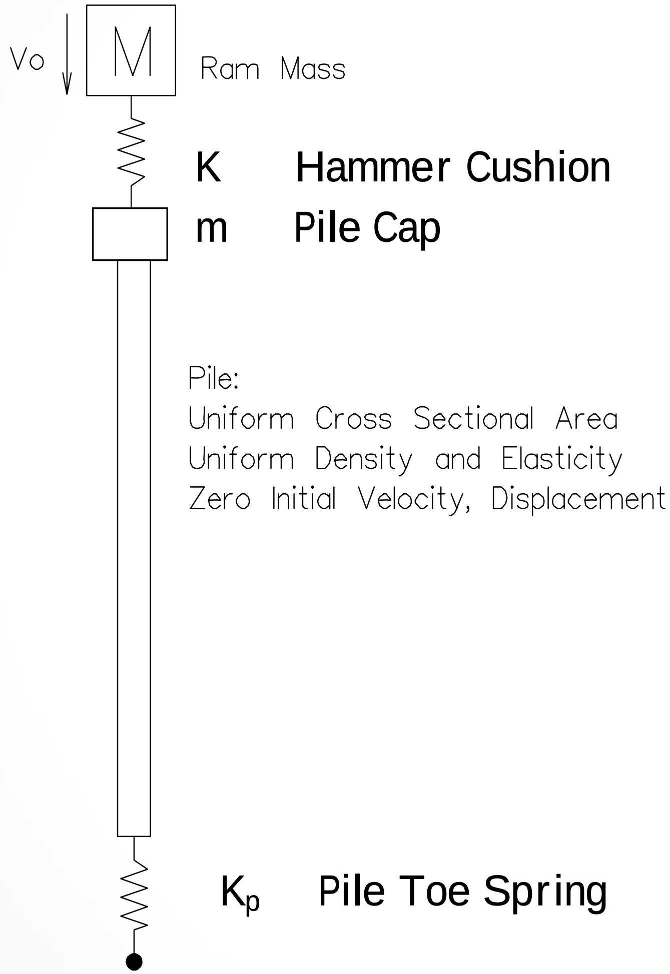
Modeling the Pile Hammer

- 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:

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



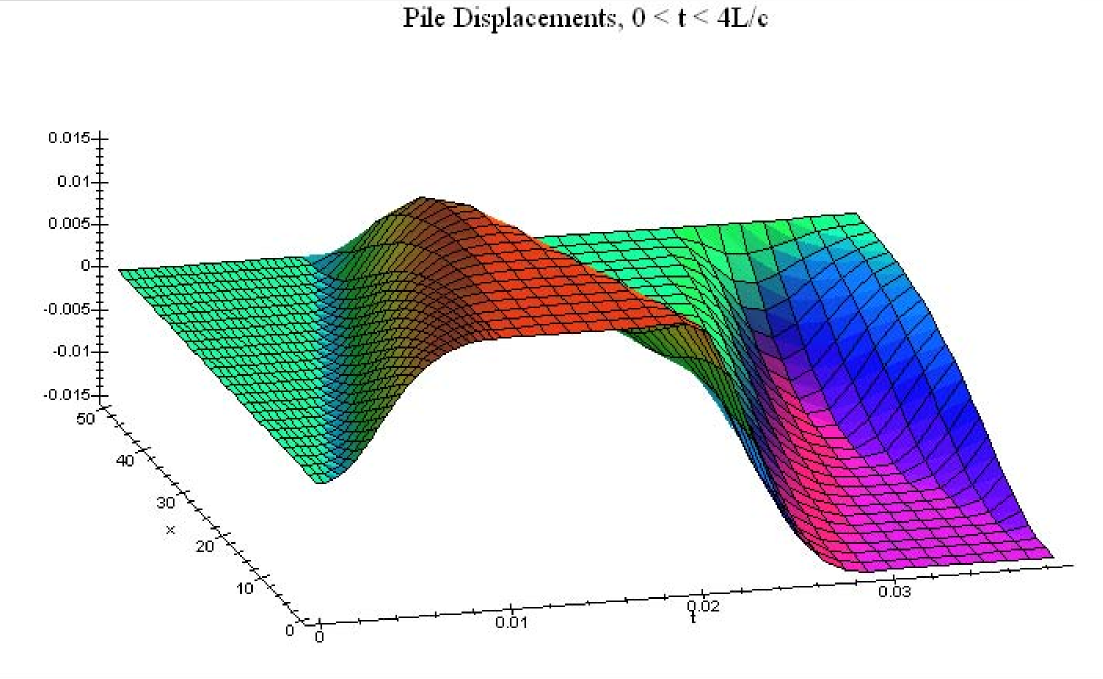
Closed Form Solution Finite Undamped Pile

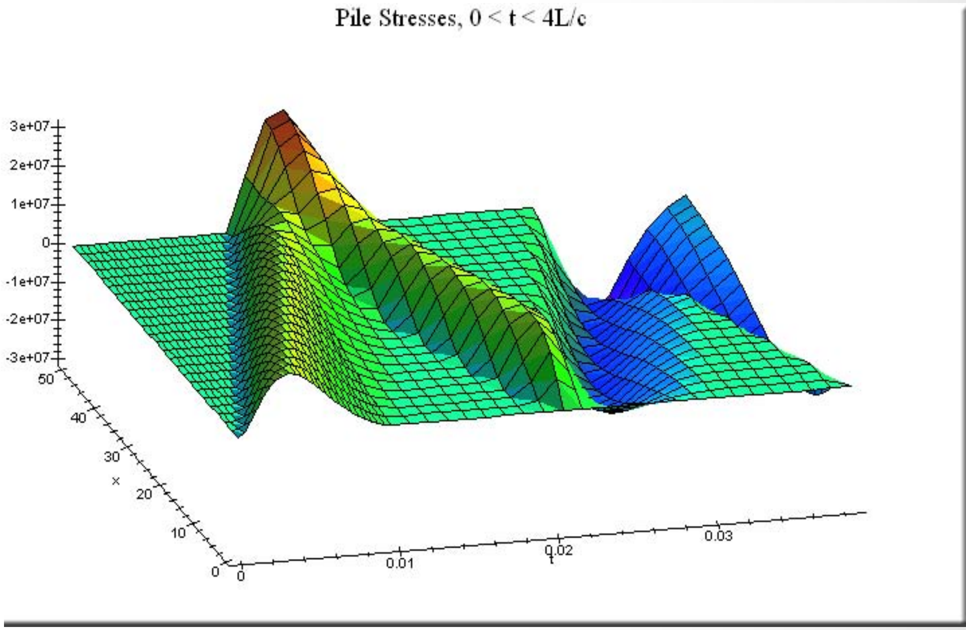


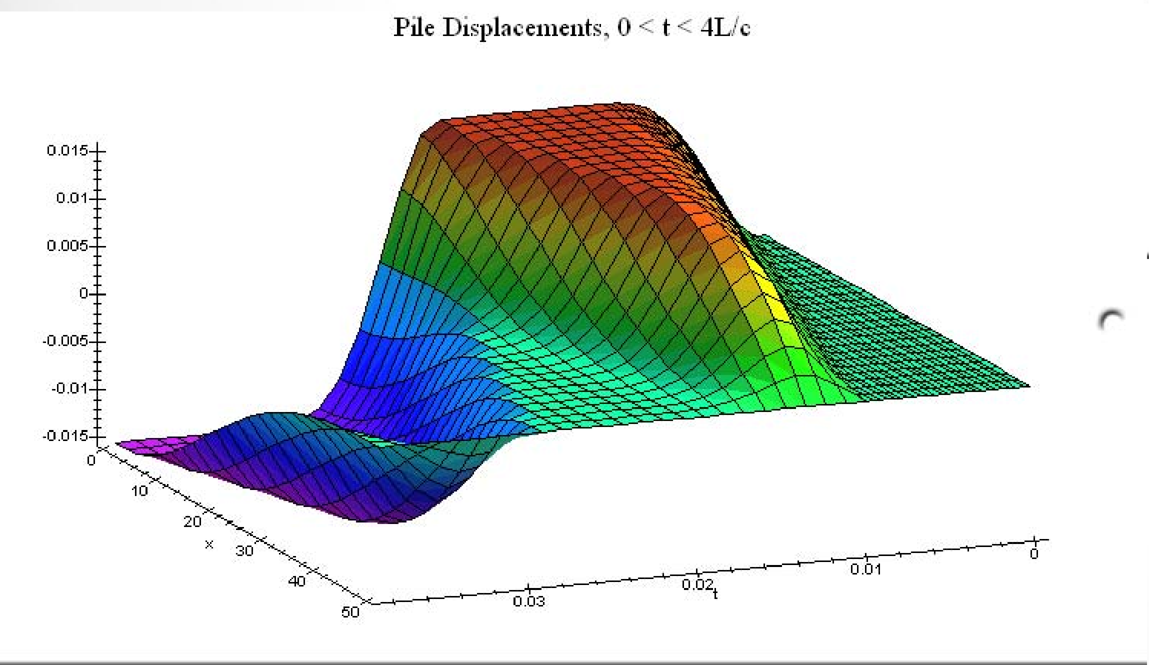
- Simple hammer-pile-soil system
- We will use this to analyze the effect of the variation of the pile toe
- Pile toe spring stiffness can vary from zero (free end) to infinite (fixed end) and an intermediate condition

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

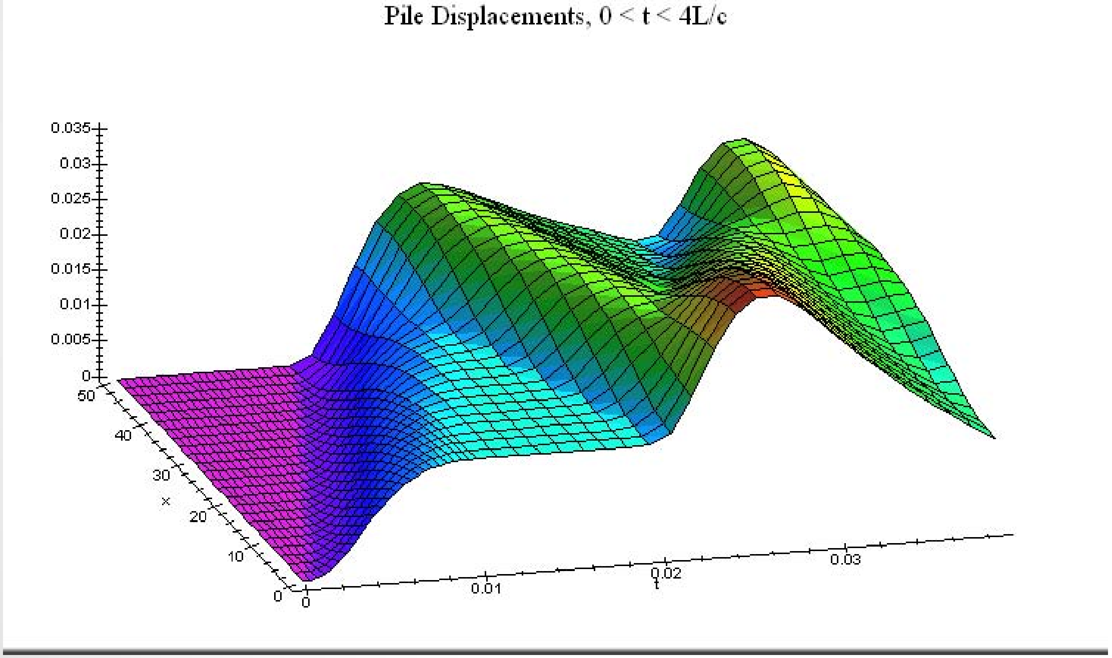
Fixed End Results

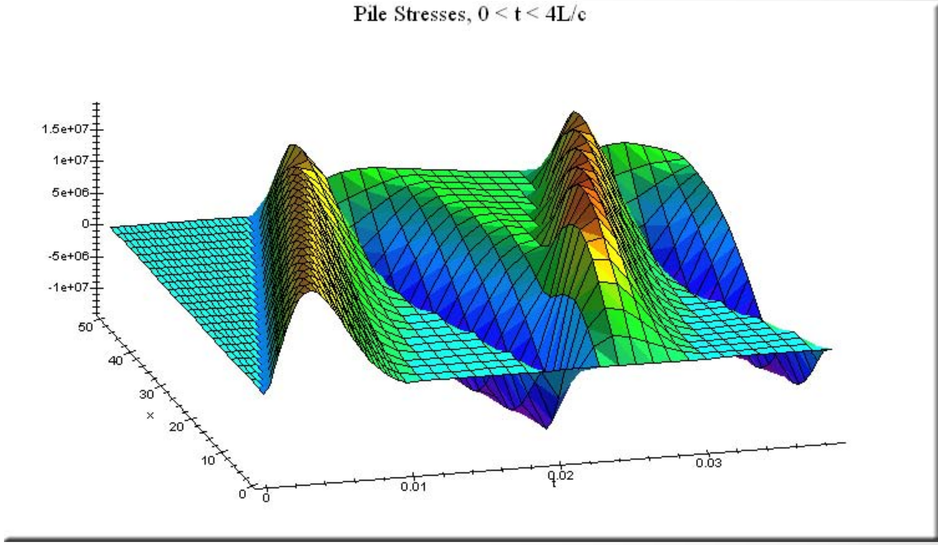




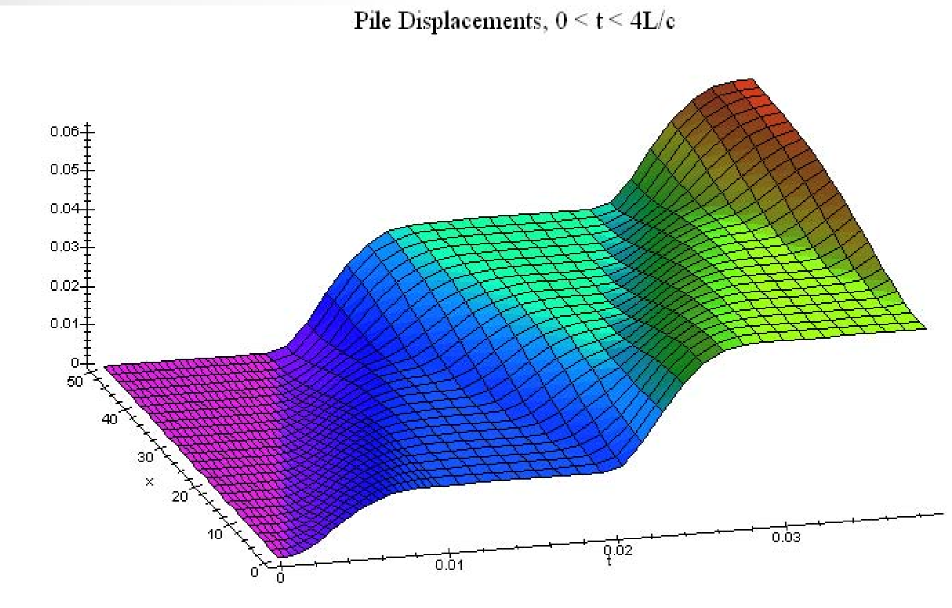


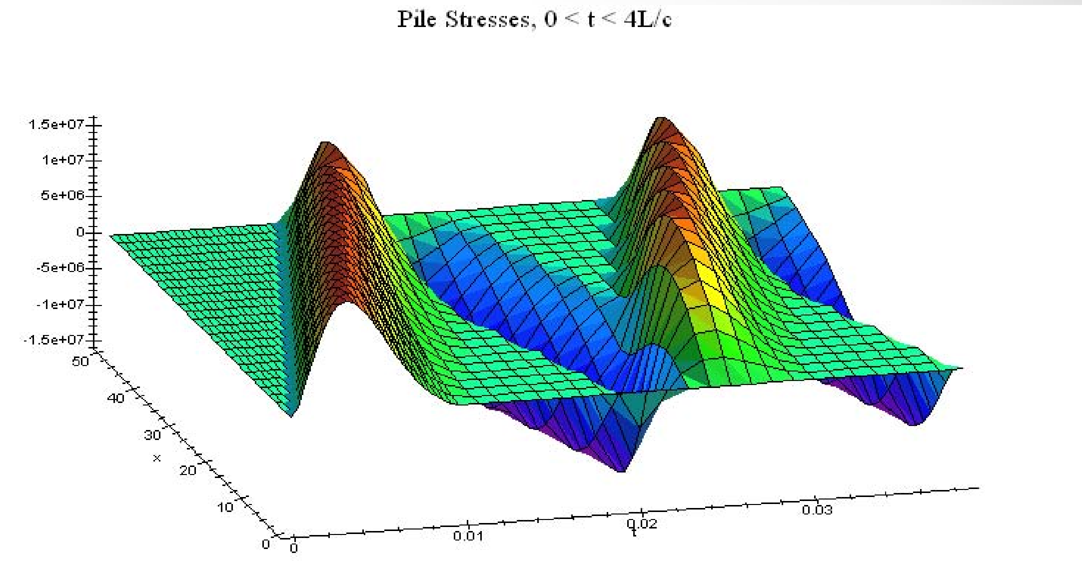
Intermediate Case Results

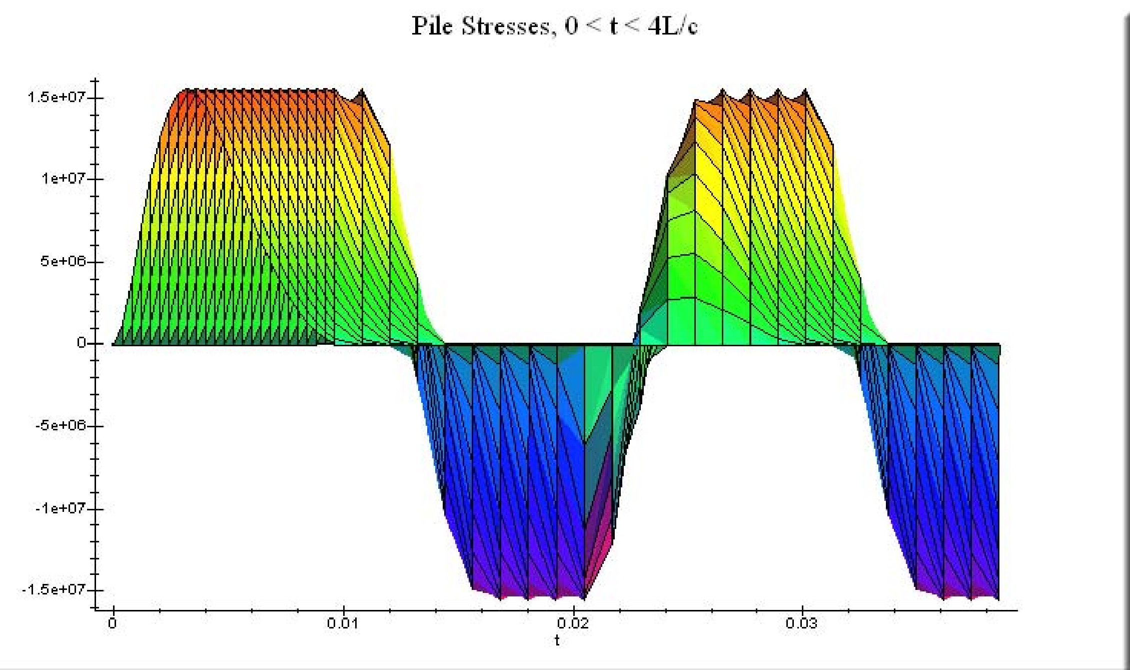




Free End Results







Numerical Solutions

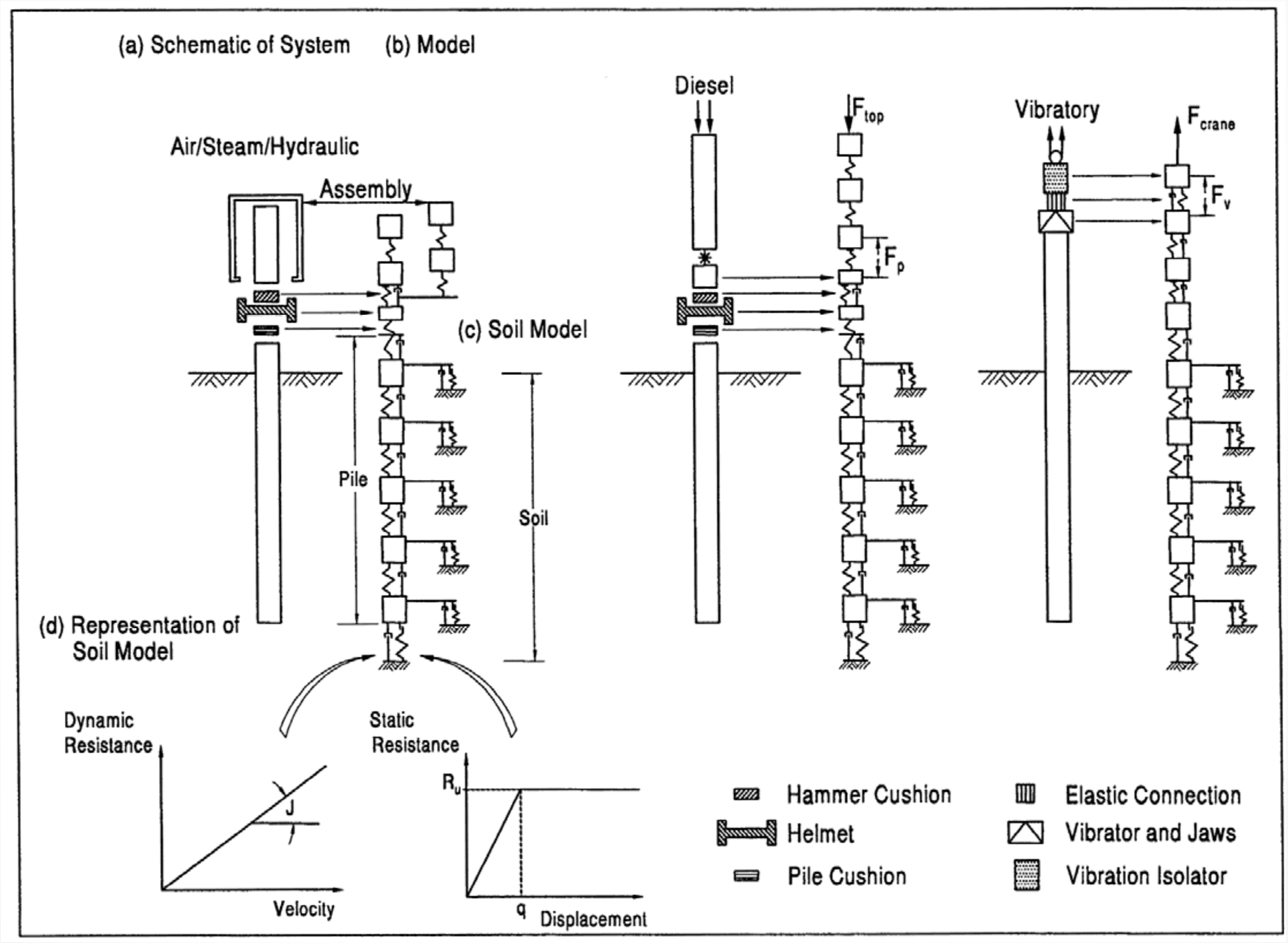
- Subsequent Solutions
  - TTI (Texas Transportation Institute) – late 1960's
    - Very similar to Smith's solution
  - GRL/Case – 1970's and 1980's
    - Added adequate modelling of diesel hammers
    - Added convenience features
  - TNO

- 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

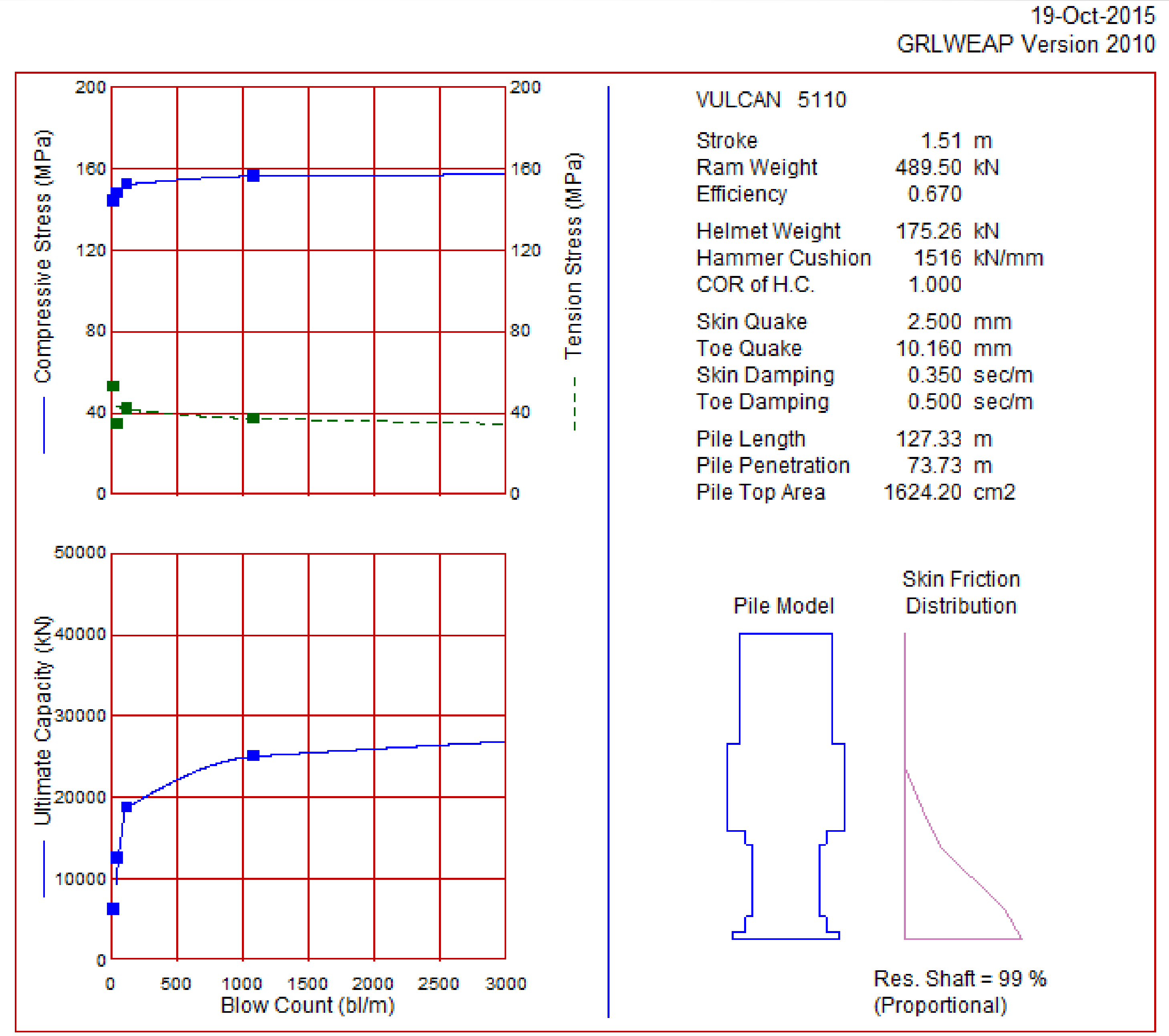
Necessity for Numerical Solution

- 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	**Design Stress** 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. **Driving Stress** 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	**Design Stress** 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. **Driving Stress** 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	**Design Stress** 0.25 f_y (on steel area) ***plus*** 0.40 f'_c (on concrete area) **Driving Stress** 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	**Design Stress** 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)
	Driving Stress 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) **US Units*** $< 0.25\ (f'_c)^{1/2} + f_{pe}$ (on gross concrete area) **SI Units** * 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	**Design Stress** 0.33 f'_c (on gross concrete area) ; f'_c minimum of 5.0 ksi (34.5 MPa)
	Driving Stress Compression Limit $< 0.85\ f'_c$; Tension Limit $< 0.70\ f_y$ (of steel reinforcement)
Timber Pile	**Design Stress** 0.8 to 1.2 ksi (5.5 to 8.3 MPa) for pile toe area depending upon species
	Driving Stress 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
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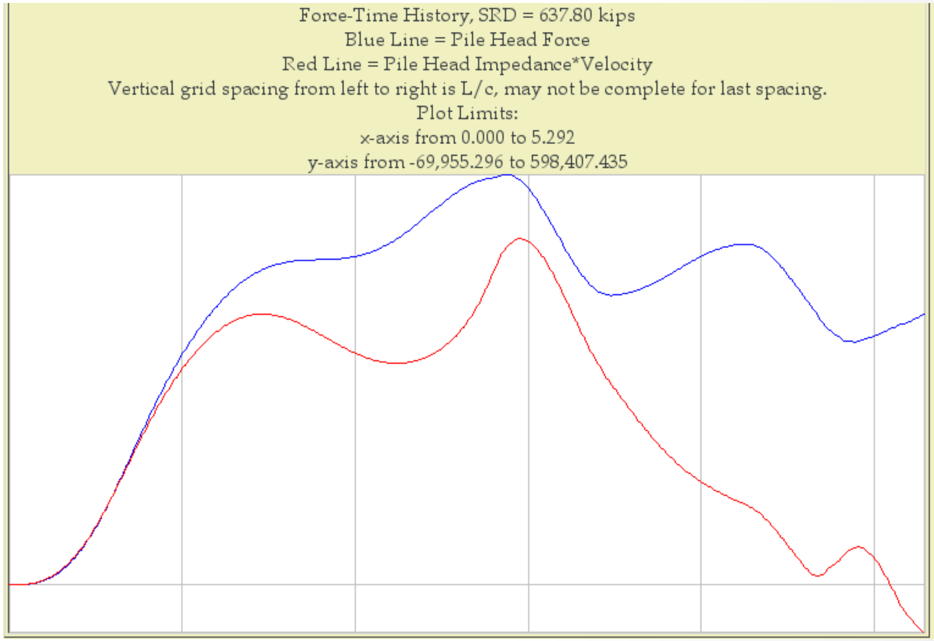
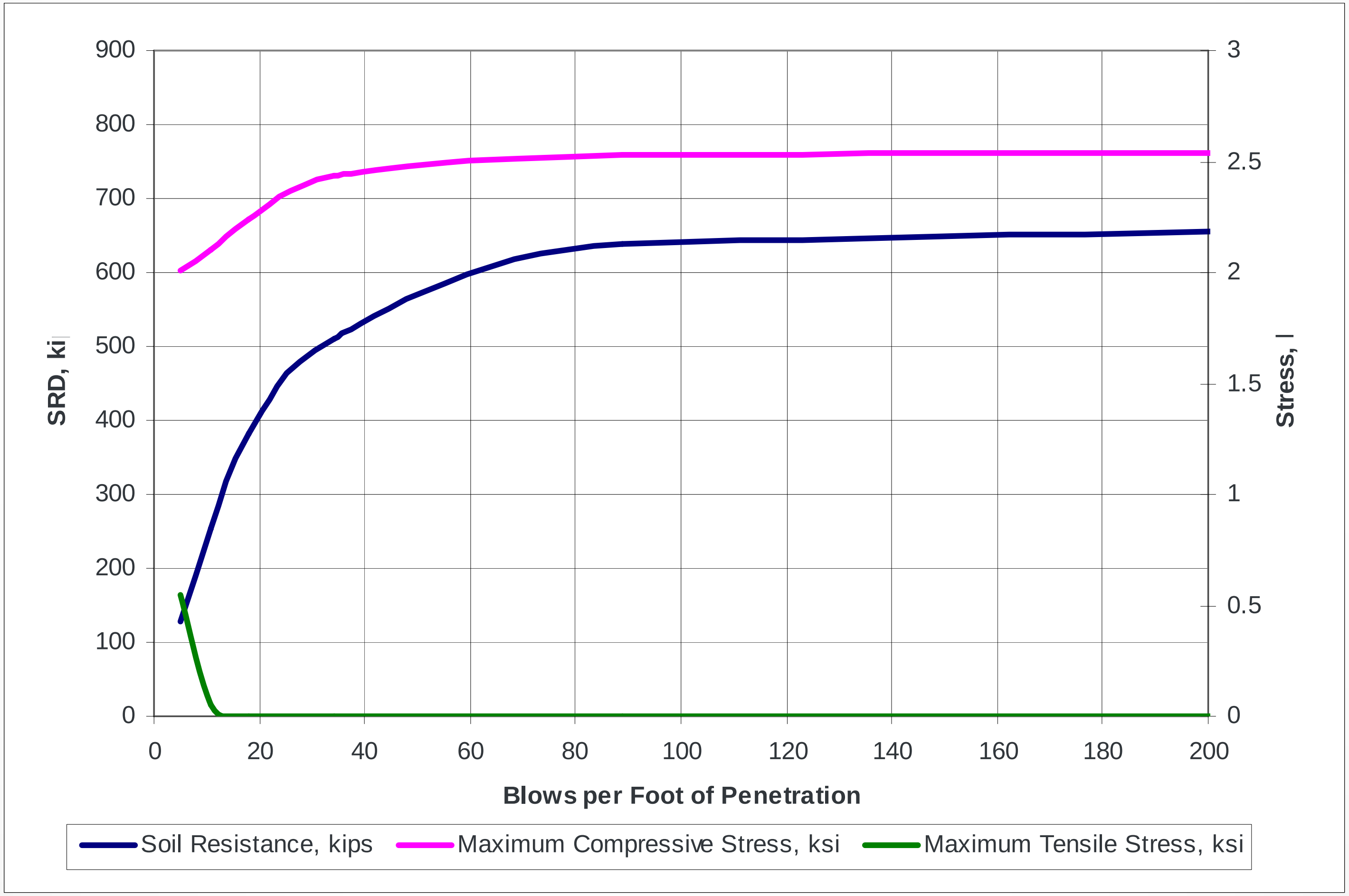
Time Step, msec	0.04024
Pile Weight, lbs.	16,000
Pile Stiffness, lb/ft	1,777,778
Pile Impedance, lb-sec/ft	103,000.1
L/c, msec	4.82813
Pile Toe Element Number	62
Length of Pile Segments, ft.	1
Hammer Manufacturer and Size	VULCAN O30
Hammer Rated Striking Energy, ft-lbs	90000
Hammer Efficiency, percent	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

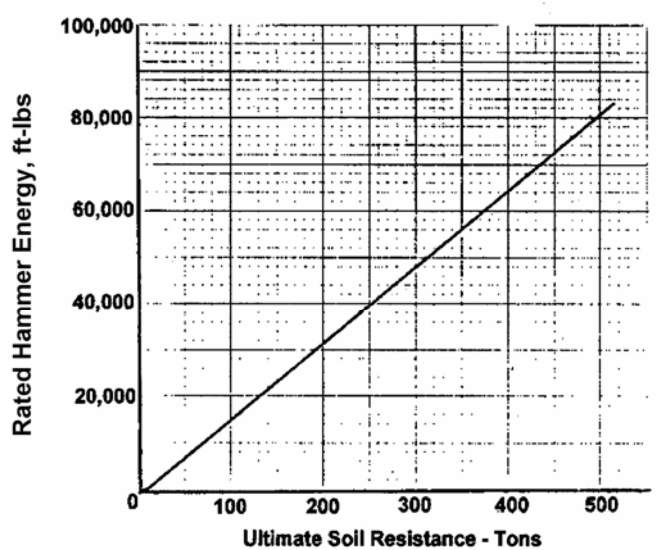


Figure 9-44. Suggested trial hammer energy for wave equation analysis.

- 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.

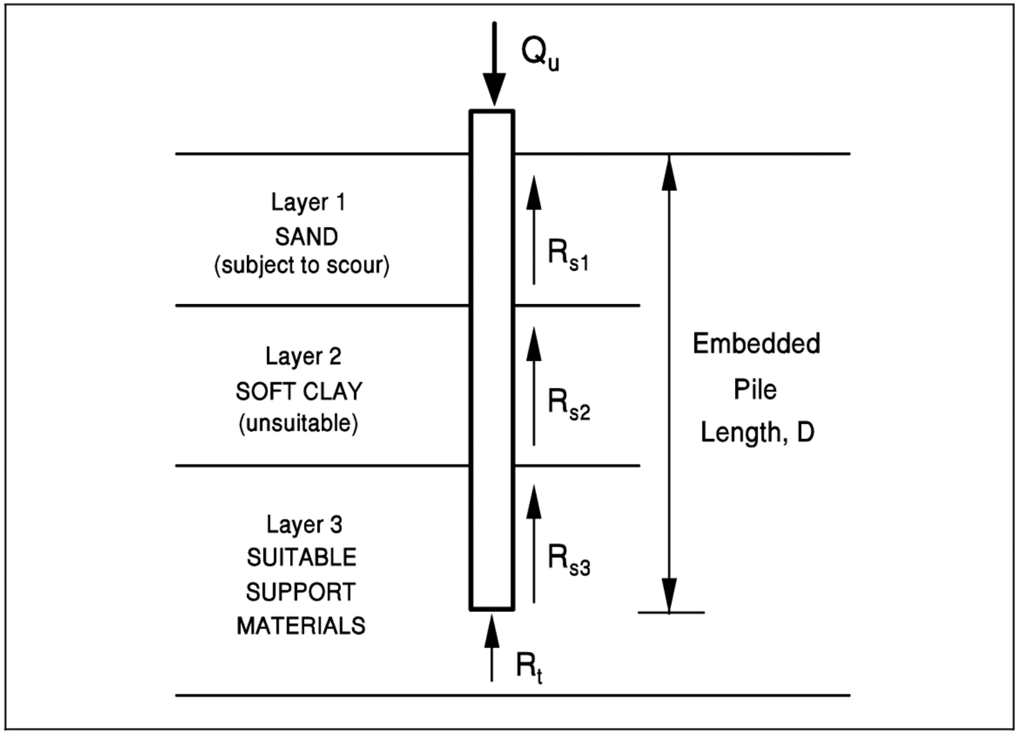
In the SRD calculation, a factor of safety is not used. The soil resistance to driving is the sum of the soil resistances from the scour susceptible and unsuitable layers plus the soil resistance in the suitable support materials to the estimated penetration depth.

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

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

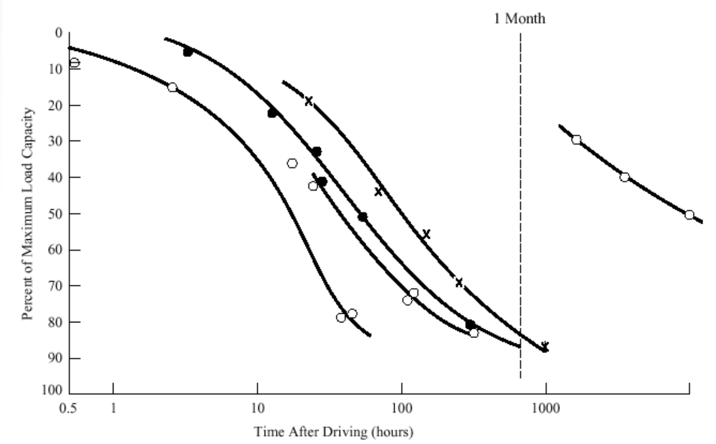


Table 9-8

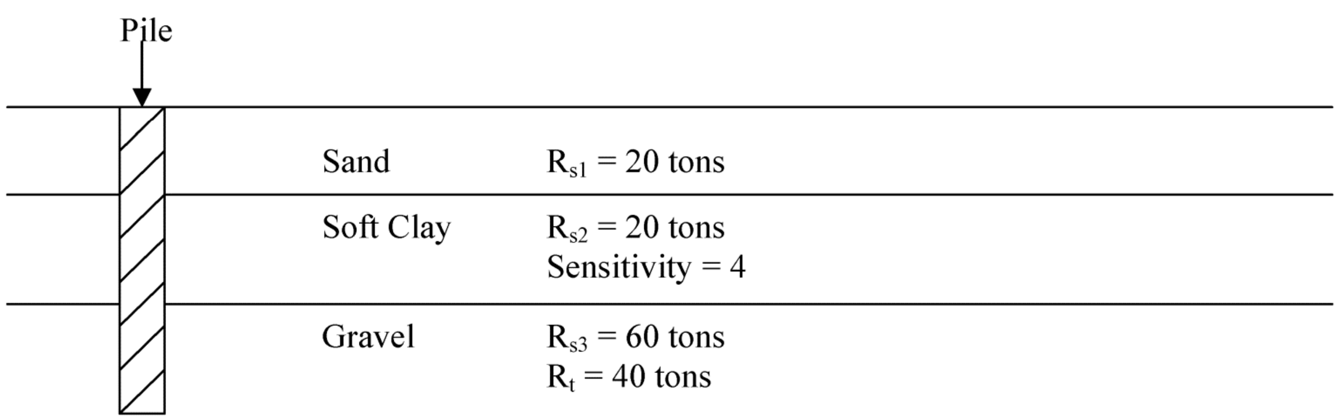
Soil setup factors (after FHWA, 1996)

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 $= R_{s3} + R_t$

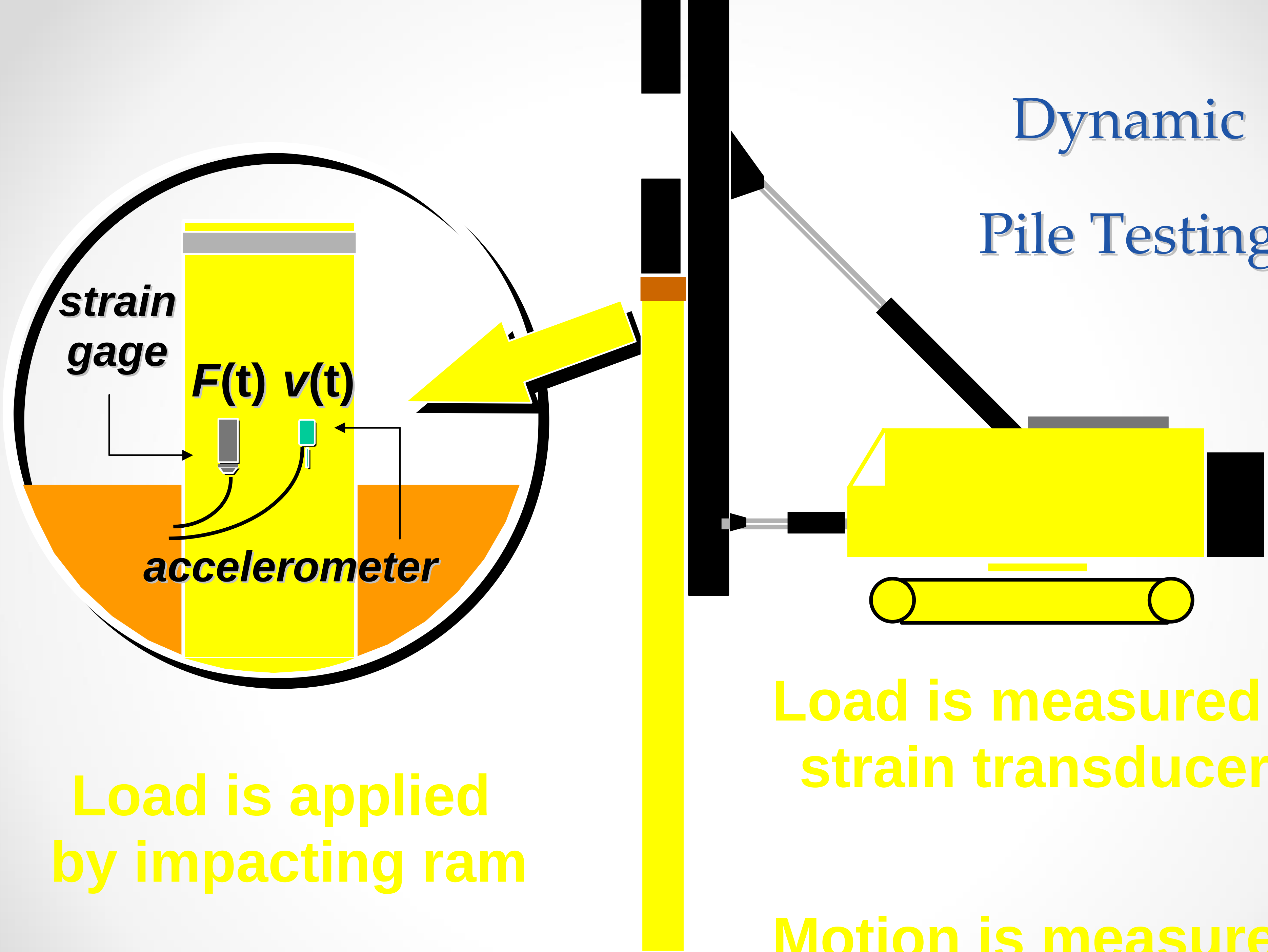
$= 60 \text{ tons} + 40 \text{ tons} = 100 \text{ tons}$

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

Pile Testing



Load is applied by impacting ram

Load is measured by strain transducers

Motion is measured by accelerometers

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

Dynamic Pile Testing

Isolation of the static pile resistance from the total pile response is the key challenge in the interpretation of dynamic pile testing methods.

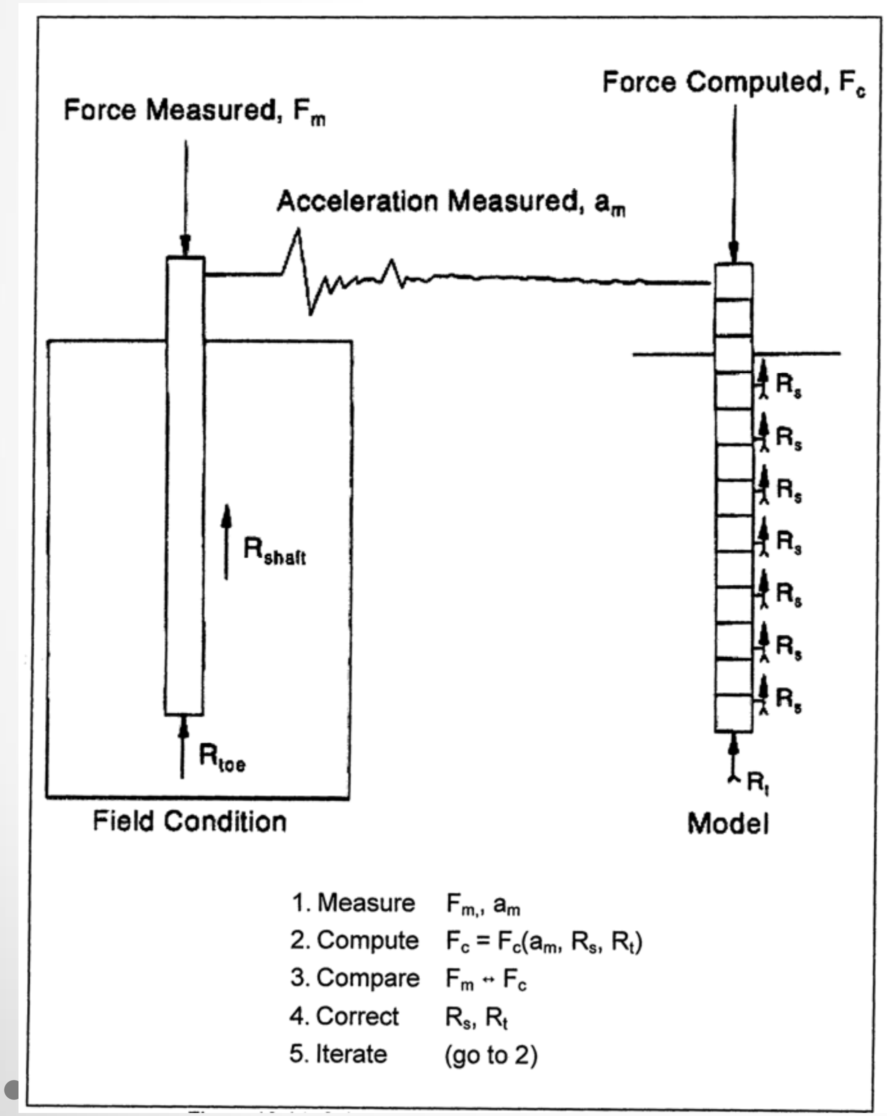
1. CASE METHOD

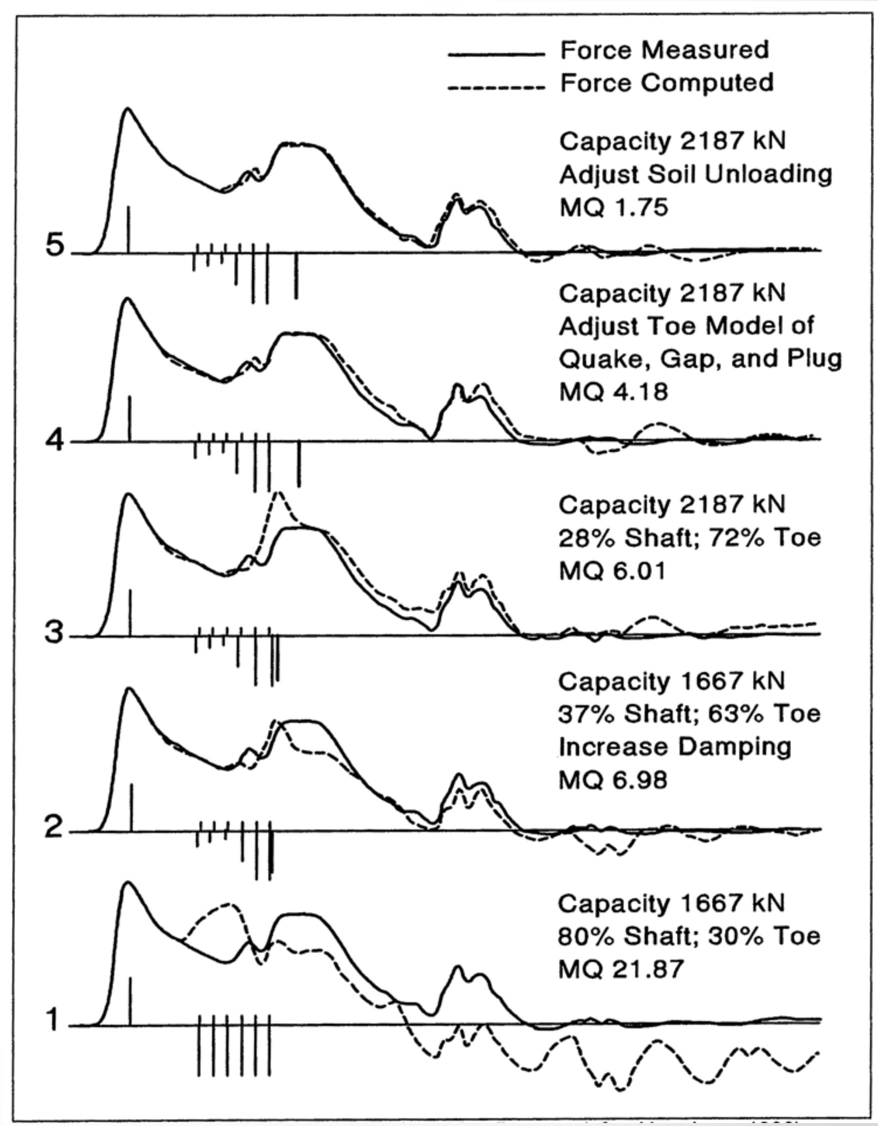
Simple closed-form solution which can be computed in real time on site, but needs a damping factor to be estimated.

2. WAVE EQUATION ANALYSIS

The mechanics of the pile and soil behavior is modelled. The model is adjusted to match the measured and computed responses.

CAPWAP Modeling of Pile Response and Capacity





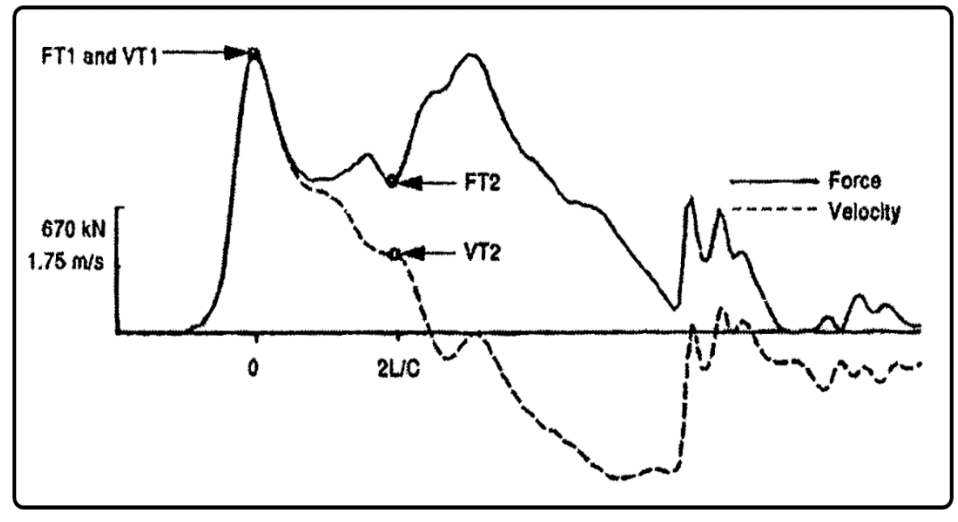
Case Method for Pile Analysis

- Governing Equation:
  - $RTL=\frac{F1+F2}{2}+Z\frac{V1-V2}{2}$
  - F1 = pile head force at the peak force of impact (or other time,) N
  - F2 = pile head force at a time 2L/c later than F1, N
  - V1 = pile head velocity at the peak force of impact (or other time,) N
  - V2 = pile head velocity at a time 2L/c later than F1, N
- Dynamic Resistance
  - $RD=J(F1+ZV1-RTL)$
  - RD = dynamic resistance of the pile, N
  - J = Case Damping Constant, dimensionless
- Static Resistance
  - $RS=\frac{F1+F2+Z(V1-V2)}{2}-J\frac{F1-F2+Z(V1+V2)}{2}$
  - RS = static resistance of the pile, N

- Simple 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.

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
    - FT1 = 1486 kN
    - FT2 = 819 kN
    - VT1 = 3.93 m/sec (Z·VT1 = (381)(3.93) = 1497.33 kN)
    - VT2 = 1.07 m/sec (Z·VT2 = (381)(1.07) = 407.67 kN)

Notes About Case Method Example

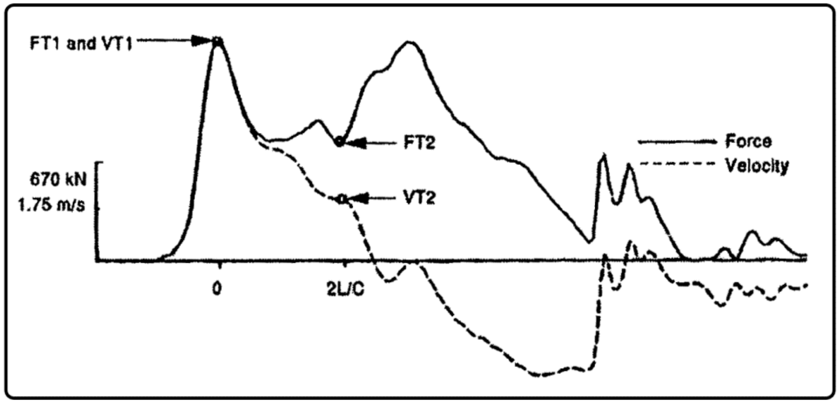
RSP Solution

- There are two curves, both at the pile top. The first “F” curve (solid line) is the force-time history of the impact blow. The “V” curve (dashed line) is the velocity-time history. Generally speaking, the velocity history is multiplied by the pile impedance, as is the case here. This is not only to make the two quantities scale properly on one graph; as noted earlier, if the pile were semi-infinite, the two curves would be identical. This is in fact the case in the early portion of the impact; neither pile movement relative to the soil nor reflections from the shaft are a factor until later.

- Case Method results can be interpreted in several ways. The method shown in the graph is the RSP method, best used for piles with low displacements and high shaft resistances. The t1 for the RSP method is the first peak point in the force-time curve; the time t2 is time 2L/c after that. The time t1 is not the same as the time t = 0 in the closed form solution, or the very beginning of impact.
- A Case damping constant J = 0.4 is assumed.

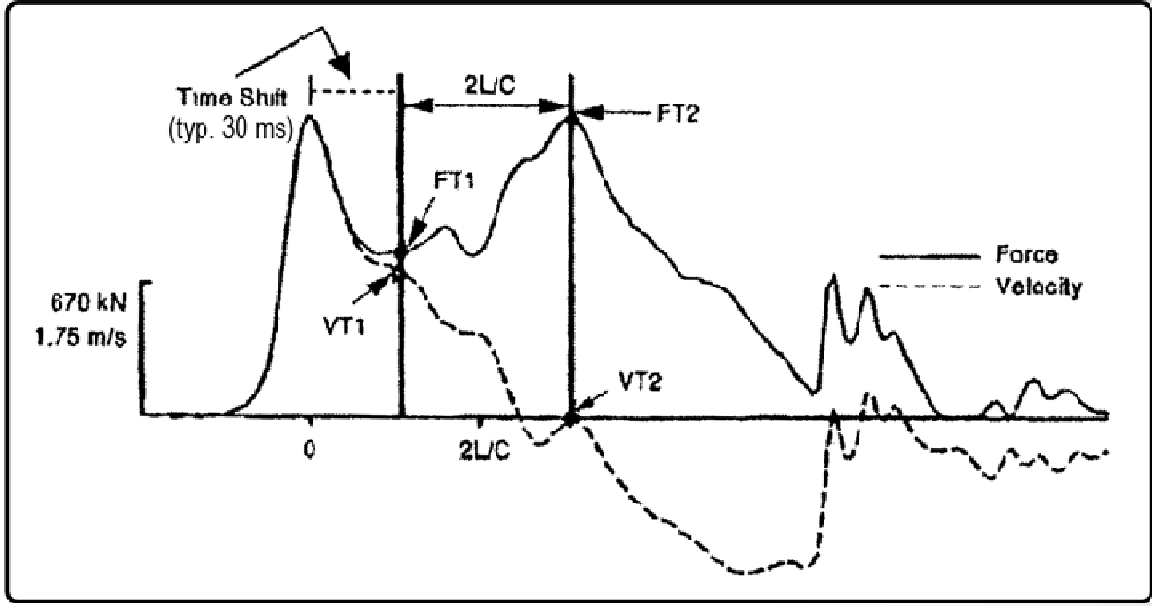
$$RSP=\frac{1486+819+381(3.93-1.07)}{2}-0.4\frac{1486-819+381(3.93+1.07)}{2}$$

$$=1697-514=1183\text{ kN}$$



Case Method Example

RMX Solution



- The time t1 is now the peak initial force plus a time shift, generally 30 msec with the RMX method (Fellenius (2009).) The time t2 is still t1 + 2L/c. This time shift is to account for the delay caused by the elasticity of the soil. (It is worth repeating that one of the implicit assumptions of the Case Method is that the soil resistance is perfectly plastic.)
- 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.
- 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.

- FT1 = 819 kN
- FT2 = 1486 kN
- VT1 = 1.92 m/sec (Z·VT1 = (381)(1.92) = 731.52 kN)
- VT2 = 0 m/sec (Z·VT2 = (381)(0) = 0 kN)

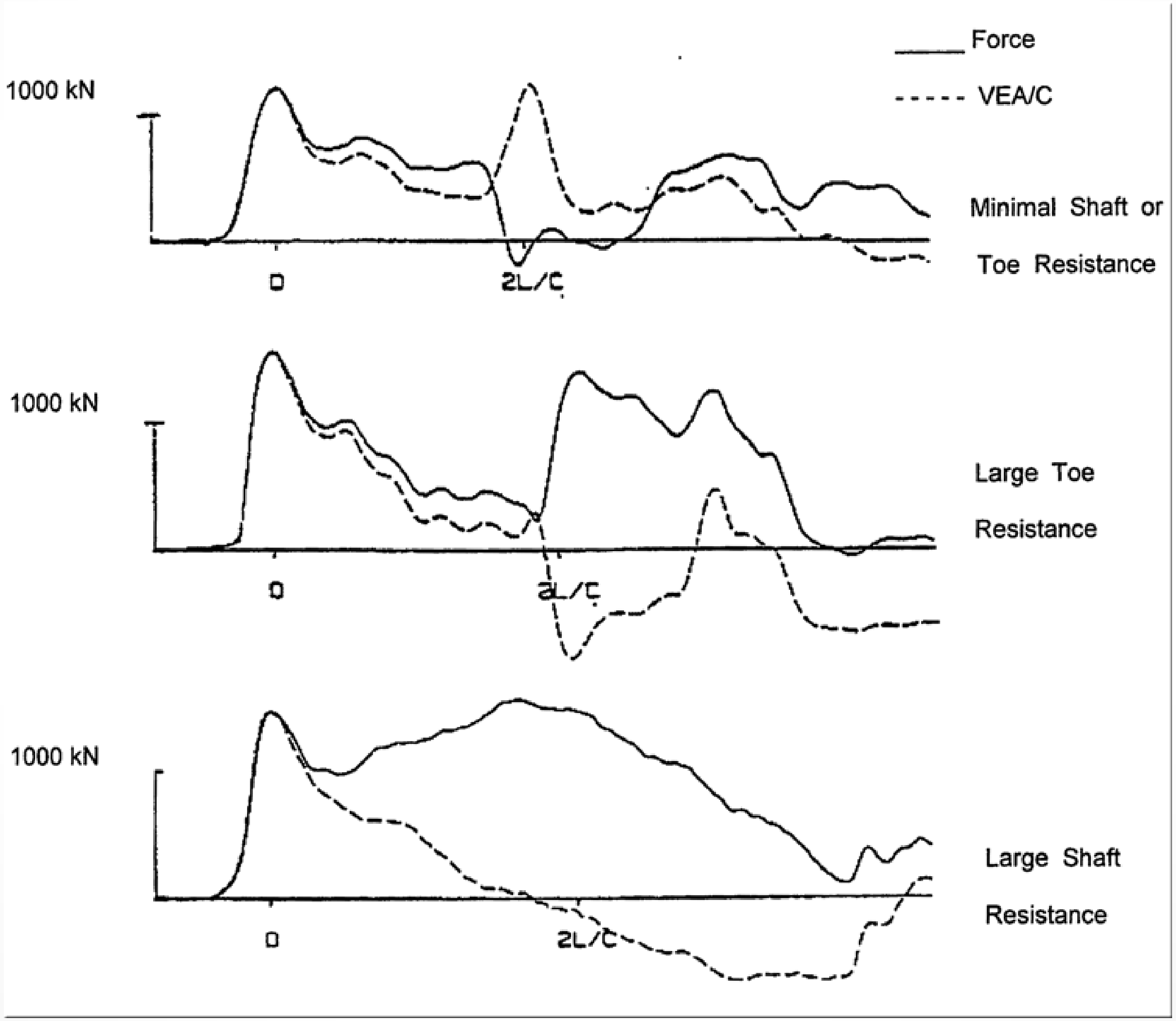
$$RMX = \frac{819+1486+381(1.92-0)}{2} - 0.7\frac{819-1486+381(1.92+0)}{2} = 1518-22 = 1496\text{ kN}$$

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.

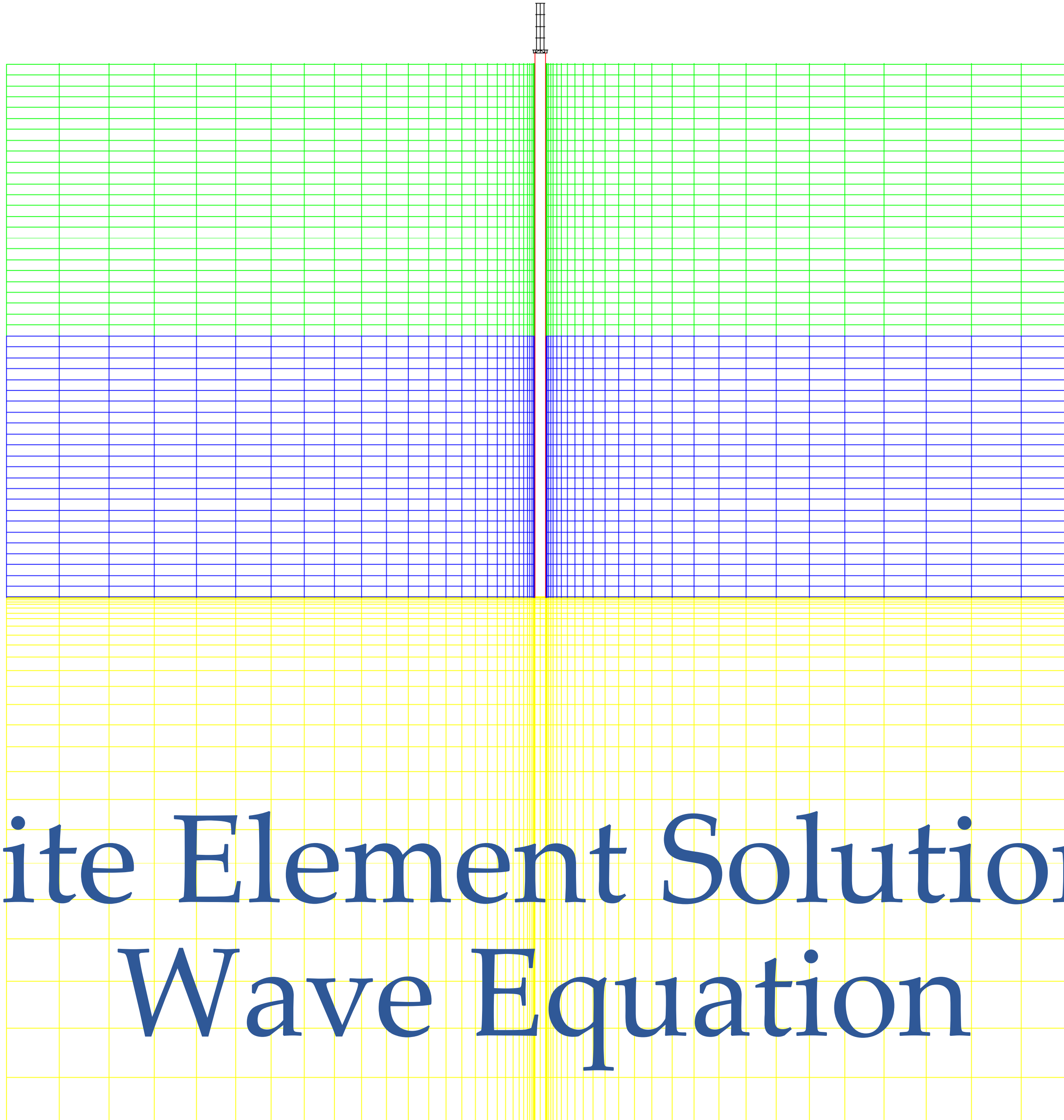
Interpreting Force-Time Curves



Dynamic Pile Testing

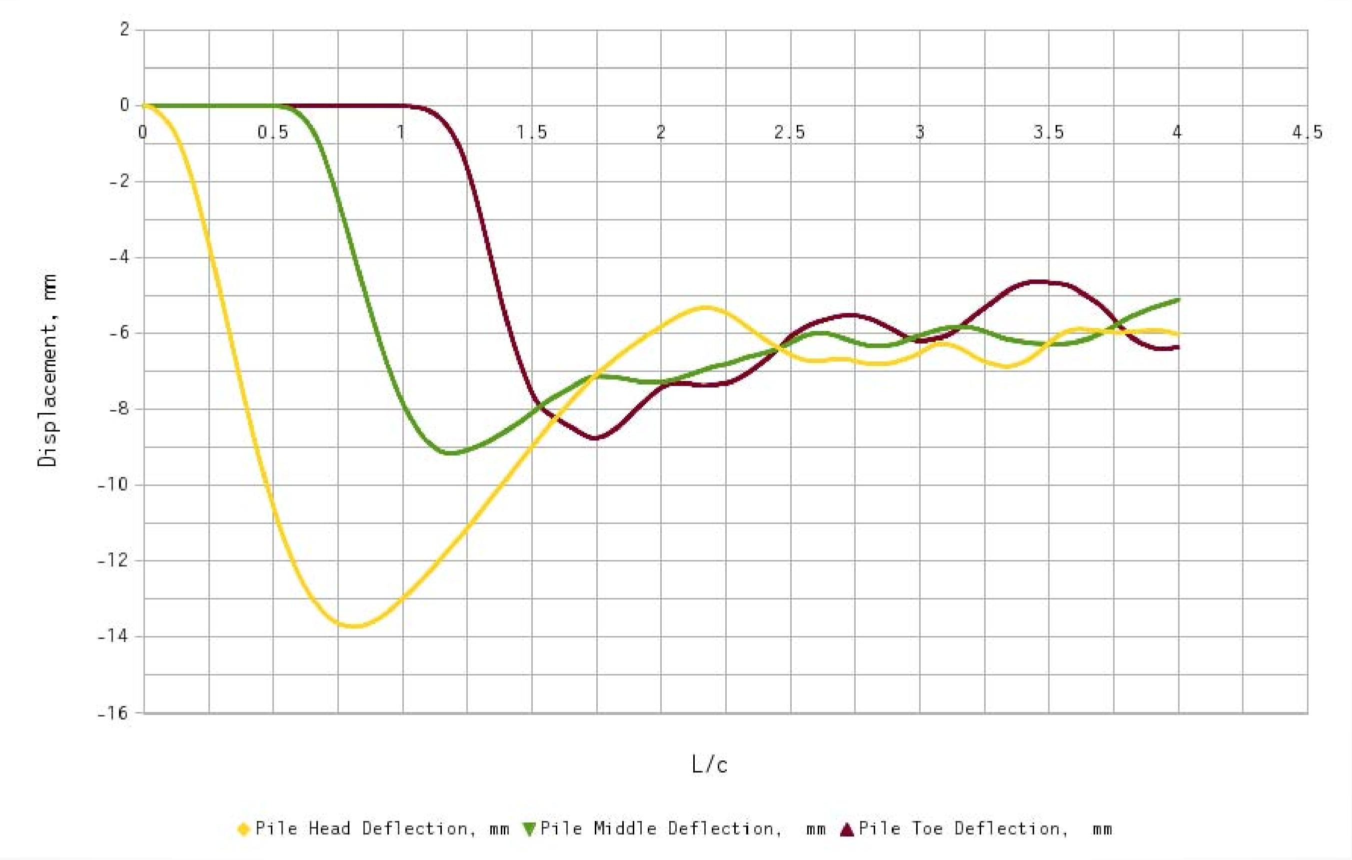
Critique

- *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***

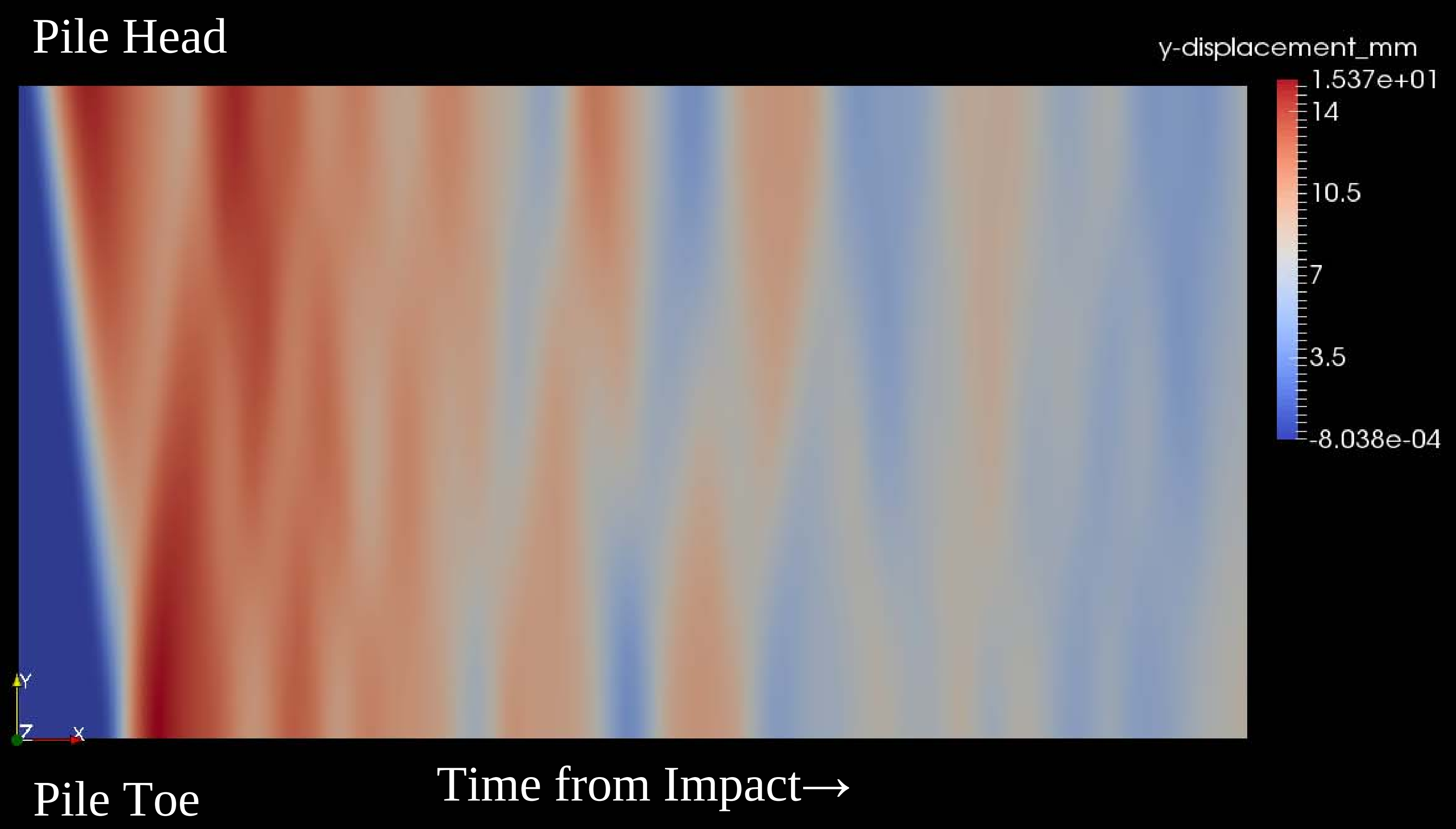


Finite Element Solution of Wave Equation

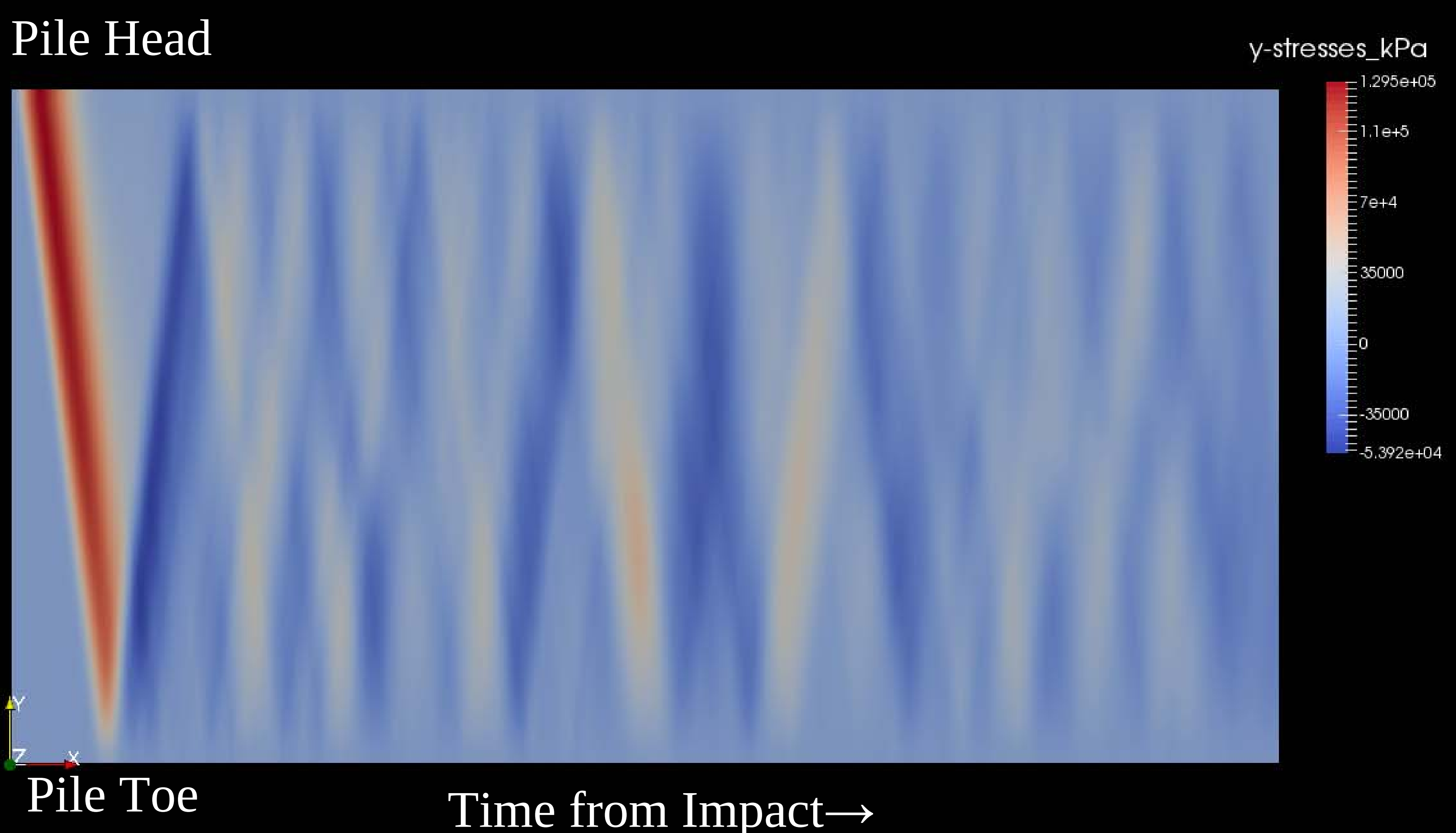
Deflection Curves



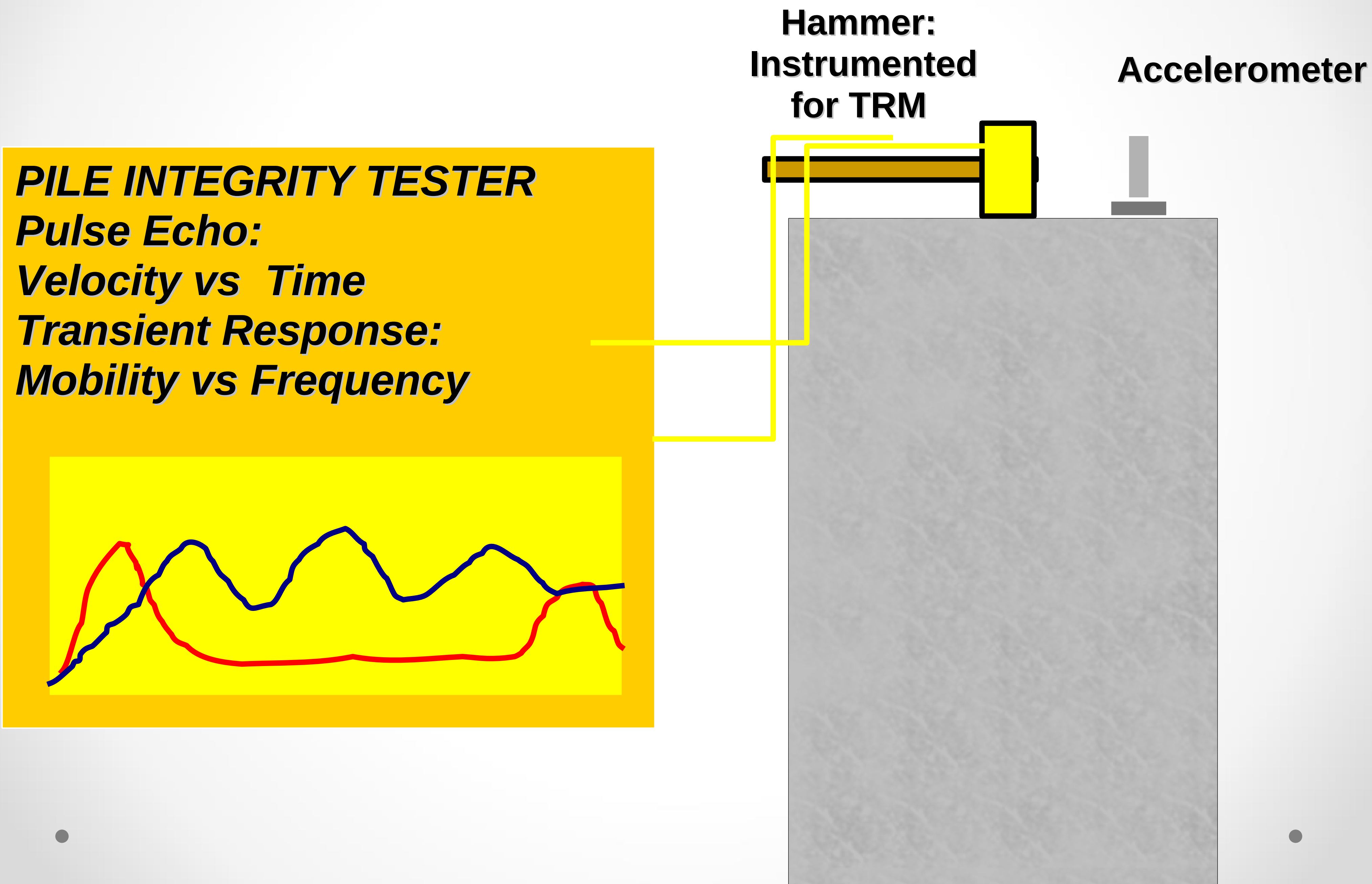
Displacement-Time Relationship



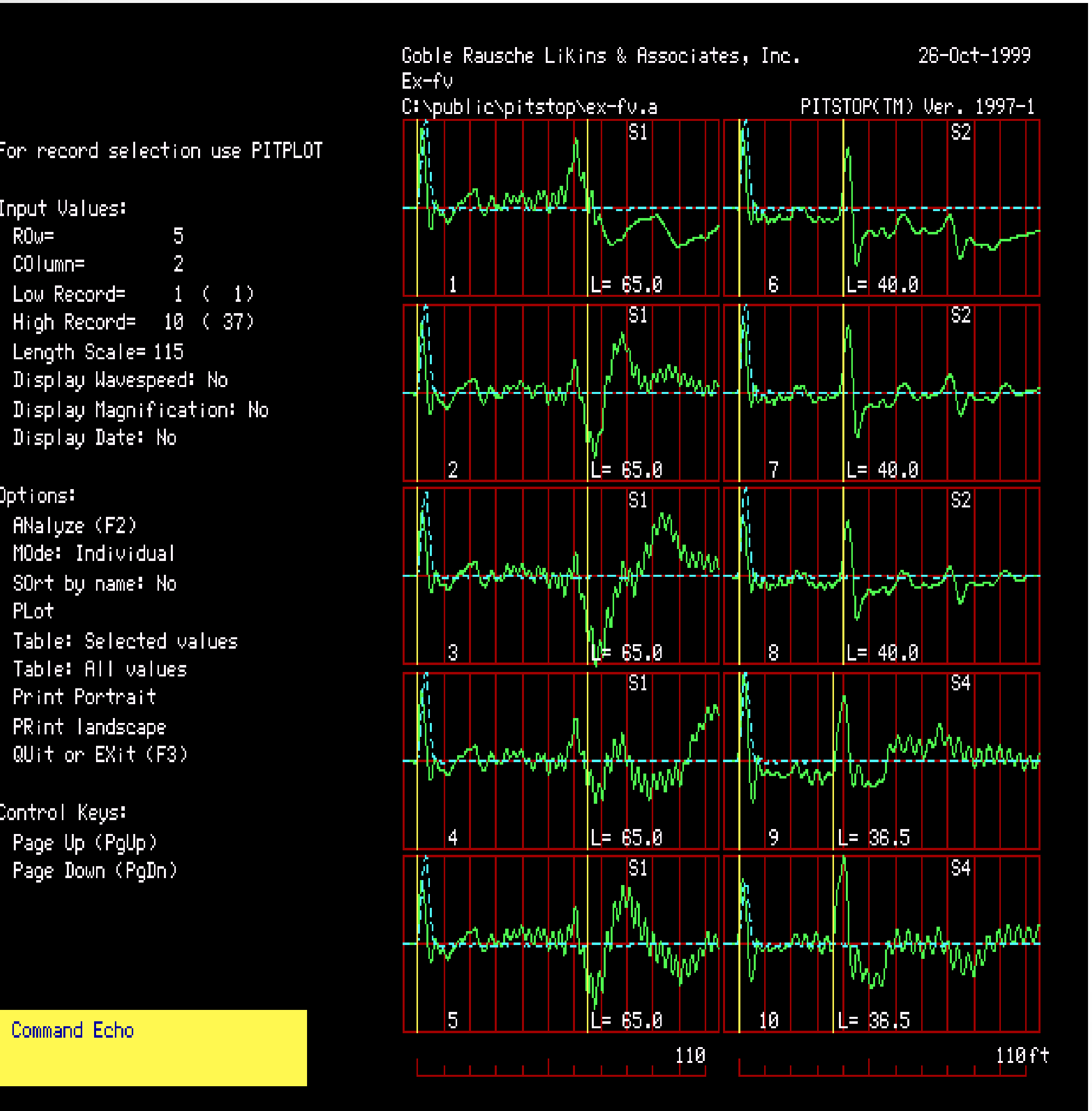
Stress-Time Relationship



Pile Integrity Testing

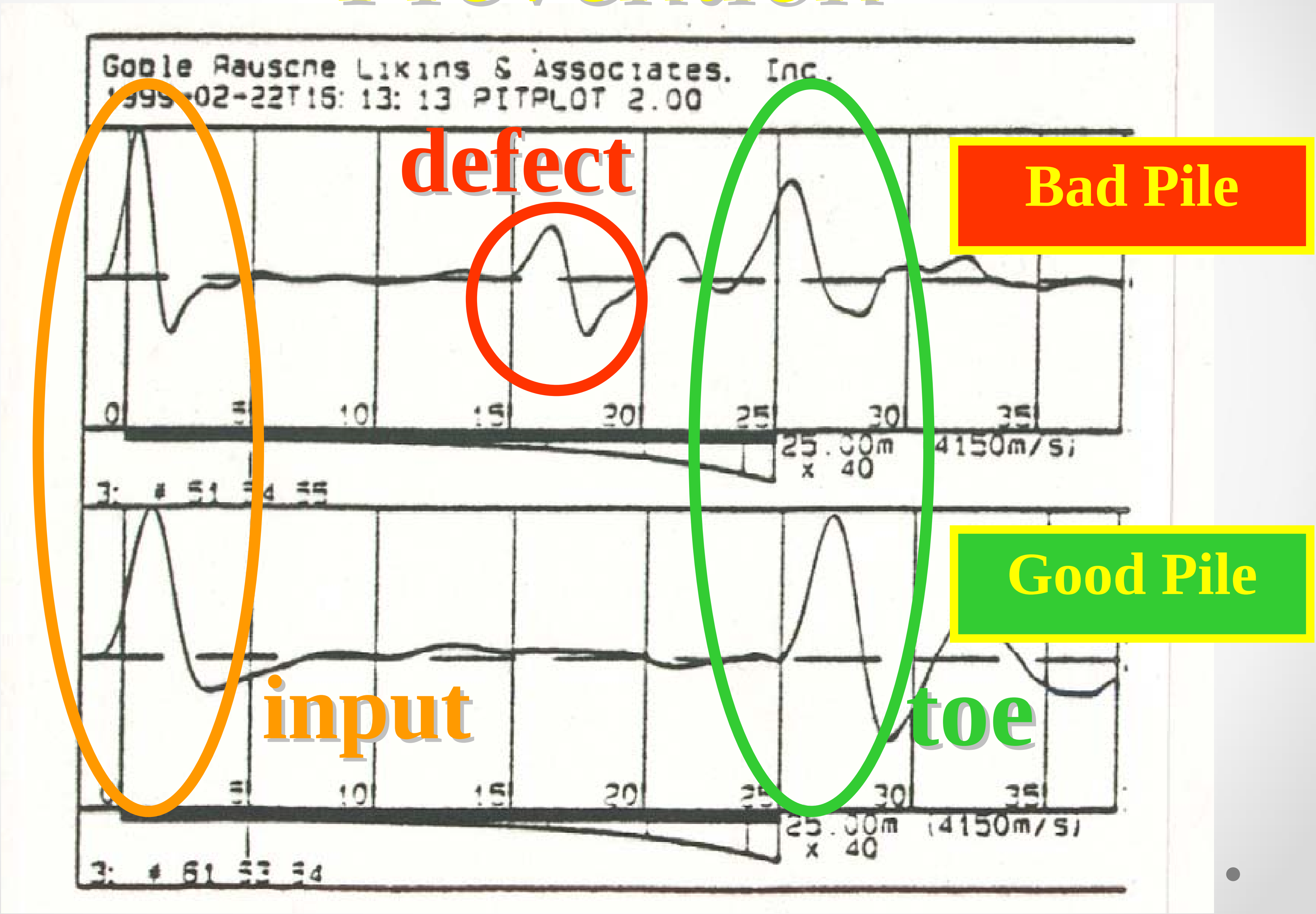


Pile Integrity Testing



- Fast, Inexpensive
- Mobile equipment, minimum site support
- Test many or even all piles on site
- No advance planning required
- Minimal pile surface preparation
- Finds major defects

Better solution is Prevention



Crosshole Methods

Crosshole Acoustic Logging

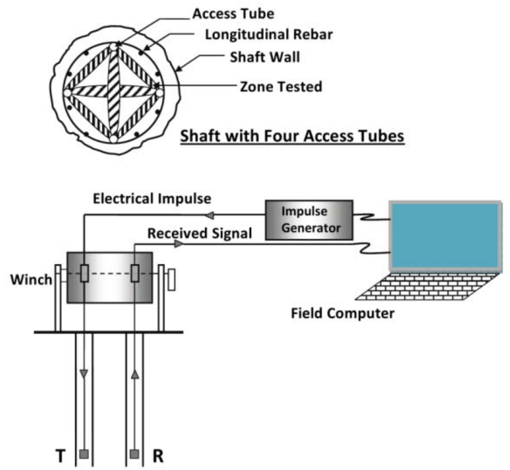


Figure 20-1 Diagram of Crosshole Acoustic Logging System (modified after Weltman, 1977)

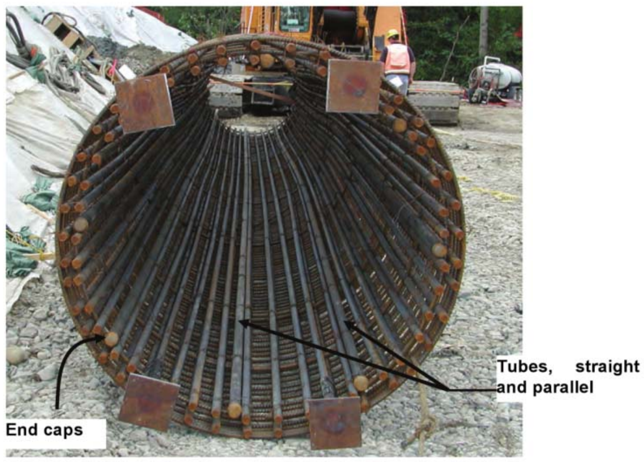


Figure 20-3 Reinforcing Cage with Steel Access Tubes for CSL Testing

Crosshole Tomography

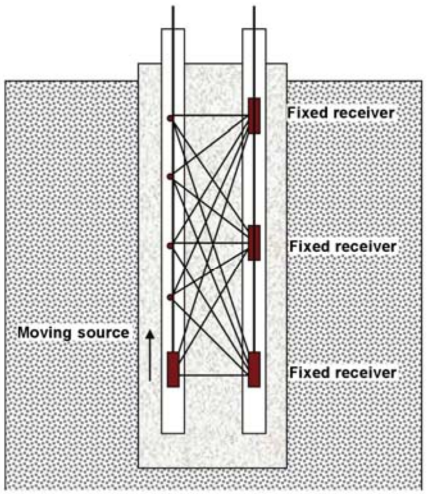


Figure 20-4 Crosshole Tomography Test (after Hollema and Olson, 2002)

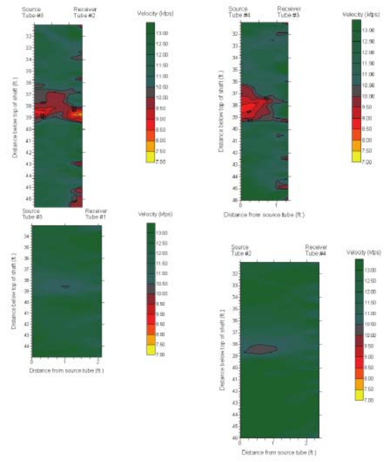
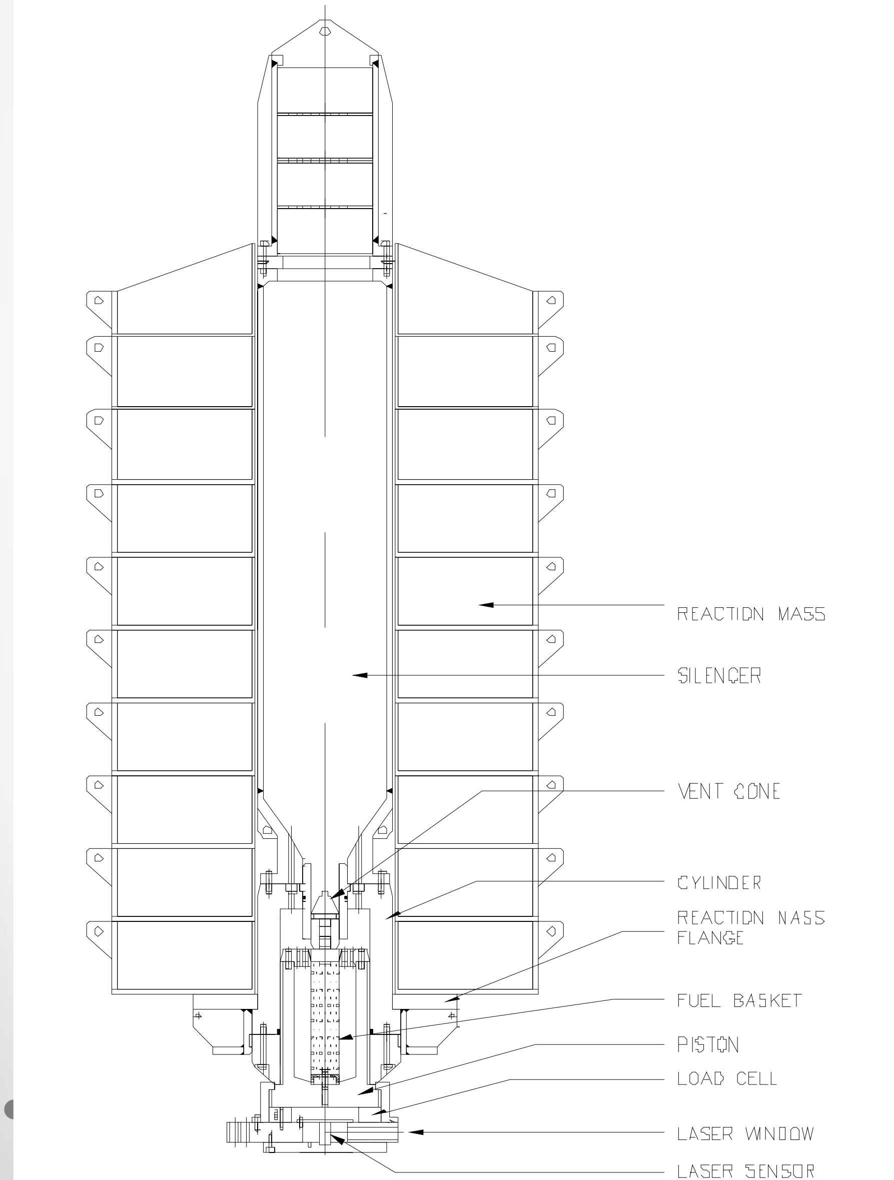


Figure 20-5 2-D Tomograms for a Shaft with Four Access Tubes (Hollema and Olson, 2002)

Statnamic Tests

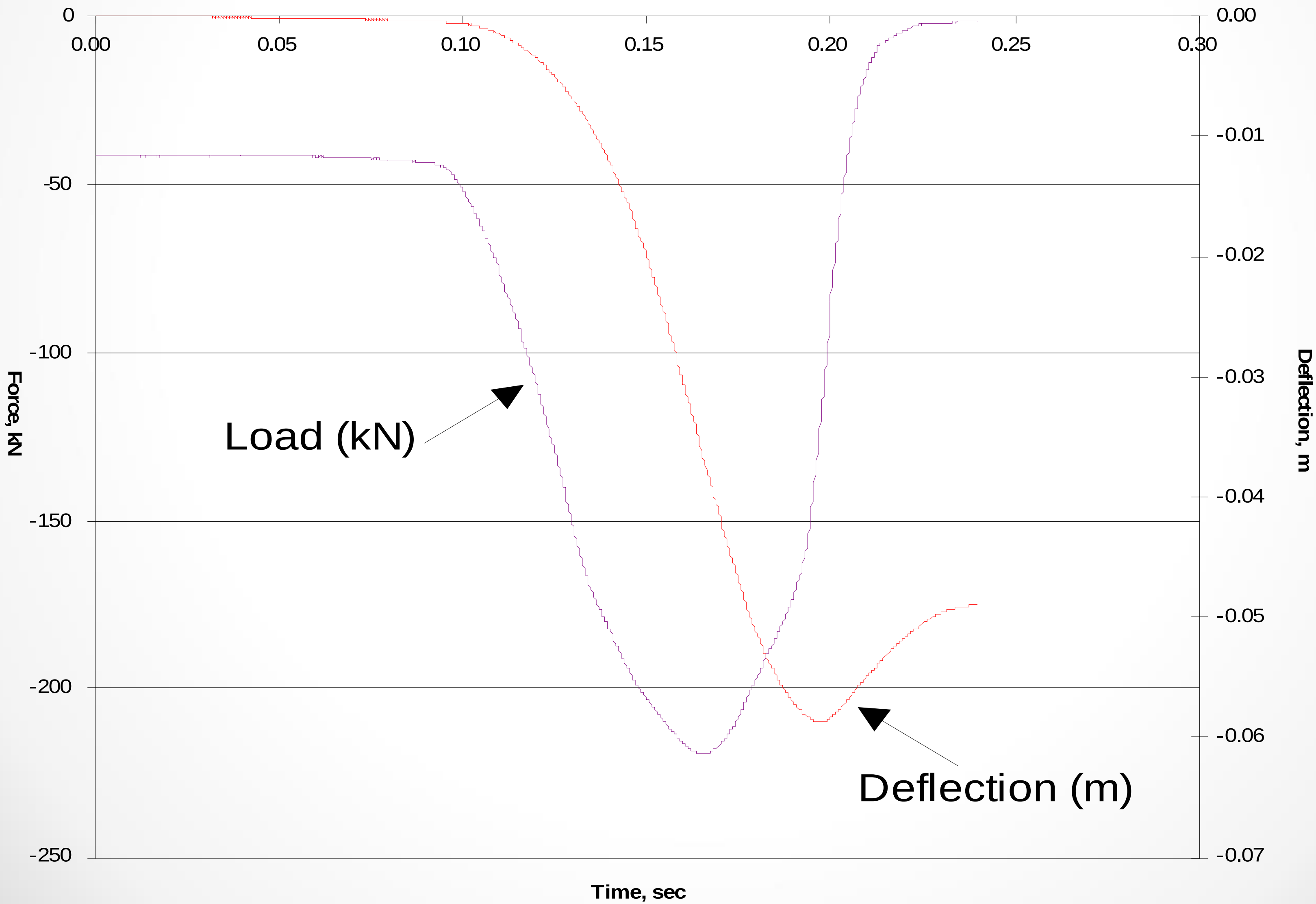
http://www.youtube.com/watch?v=IHnbd-QGdaw

Statnamic Device and Principles

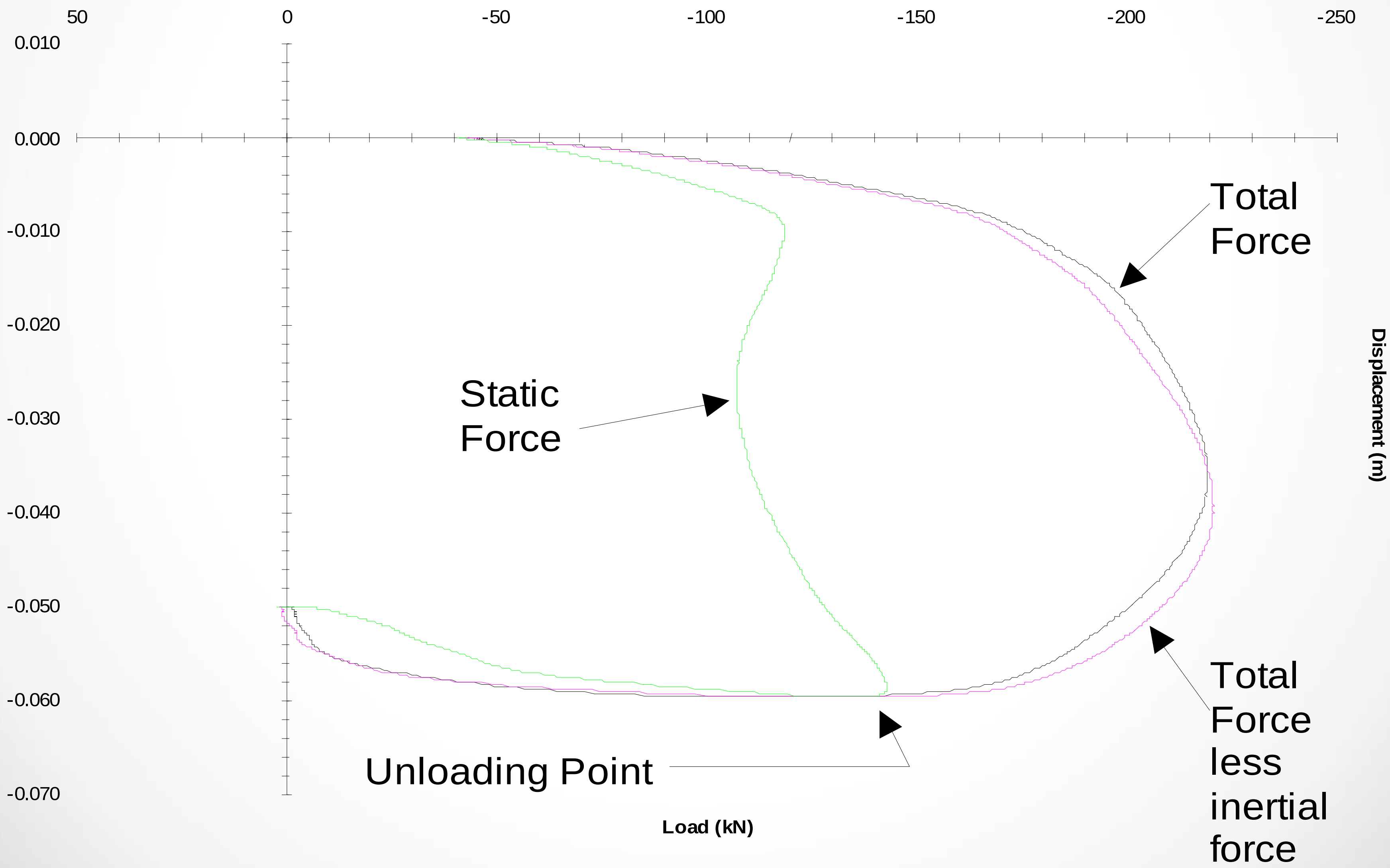


- Controlled explosion detonated; loads the pile over longer period of time than impact dynamic testing
- Upward thrust transferred to reaction weights
- Laser sensor records deflections; load cell records loads

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

Questions