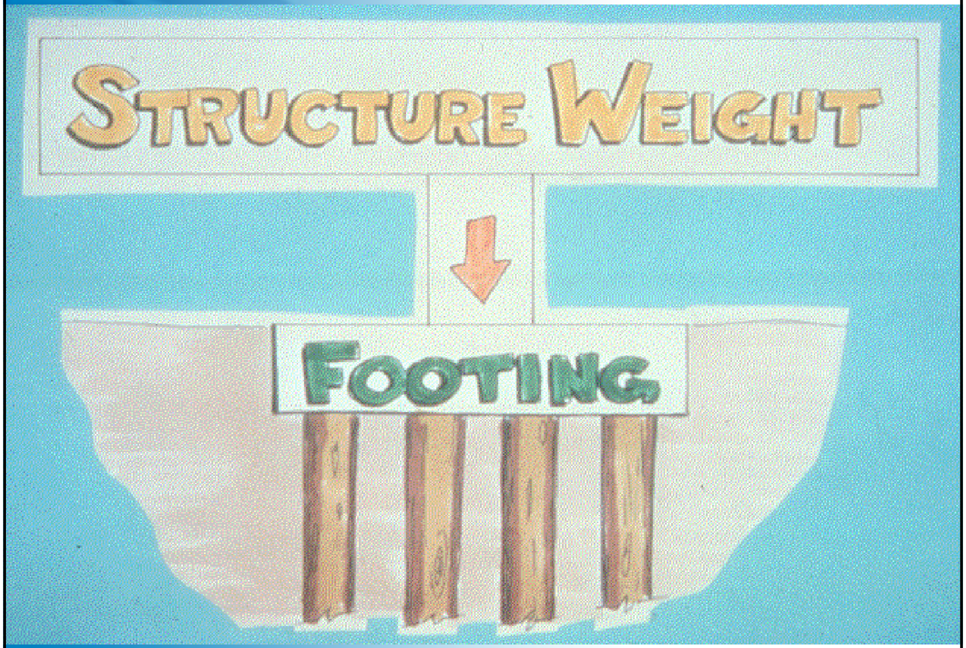
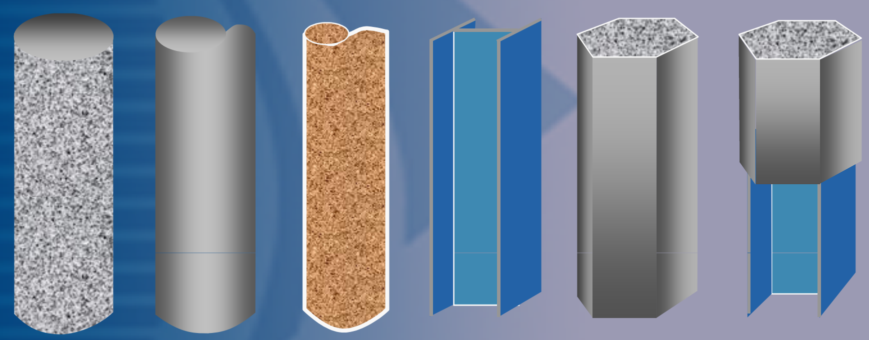
BORED PILE FOUNDATION

WHEN TO USE DEEP FOUNDATIONS (PILE)

- Upper soils are weak, structural loads are high; Required spread footings are too large
- Upper Soils are subject to scour or undermining
- Foundation must penetrate through water
- Need large uplift capacity
- Need large lateral load capacity



Types of Piling



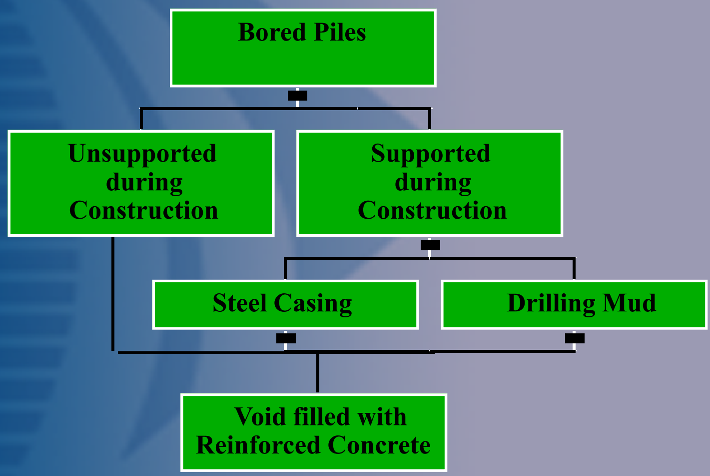
CLASSIFICATION OF PILE

- Function
- Method of installation
- Material
- Load Transfer mechanism
- As a Retaining Structures

TYPES OF DISPLACEMENT PILES

- Displacement
  - Large
    - Preformed
      - Solid Concrete, or Timber
      - Hollow tube Closed end Steel or Concrete
    - Formed in-situ
      - Tube former withdrawn void filled with concrete
  - Small
    - Hollow tube, or H-section Steel
    - Screw

TYPES OF BORED PILES



TYPES OF PILE FOUNDATIONS

- Compression Axial loaded pile
- Tension Pile
- Lateral Loaded Pile
- Void reduction/compaction purpose
- Root pile system for slope stabilization
- Friction Pile
- End Bearing and Friction

DRILLED SHAFT FOUNDATIONS

- ➔Mini Piles
- ➔Micro piles
- ➔Bored Piles
- ➔Caissons

Bridge Supported on Drilled Shafts

DRILLED SHAFTS VERSUS DRIVEN PILES

- Drilled Shafts/Advantages
  - cost of mobilizing/demobilizing a drill rig much lower than that for pile driving equipment (for hand dug caisson)
  - generates much less noise and vibration
  - opportunity to observe and verify soil conditions

DRILLED SHAFTS VERSUS DRIVEN PILES

- Drilled Shafts/Advantages (Continued)
  - Diameter/length can be changed easily to account for unanticipated conditions
  - Not hampered by presence of rock boulders
  - eliminates the need for a pile cap (for pile diameter larger than the column i.e., hand dug caisson)

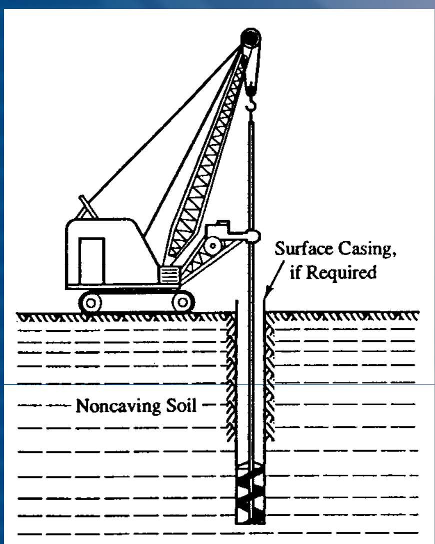
DRILLED SHAFTS VERSUS DRIVEN PILES

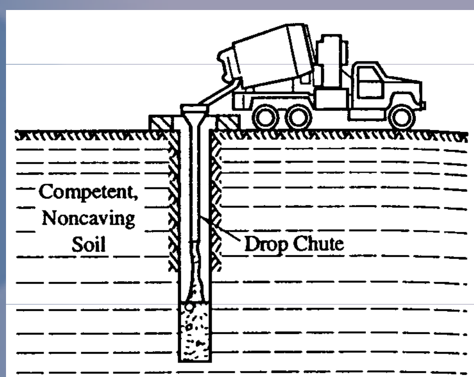
- Drilled Shafts/Disadvantages
  - Successful construction dependent on contractor's experience and skills
  - No soil displacement, therefore, lower skin friction
  - Does not densify soil near the tip
  - Full-scale load testing too expensive

DRILLED SHAFT CONSTRUCTION

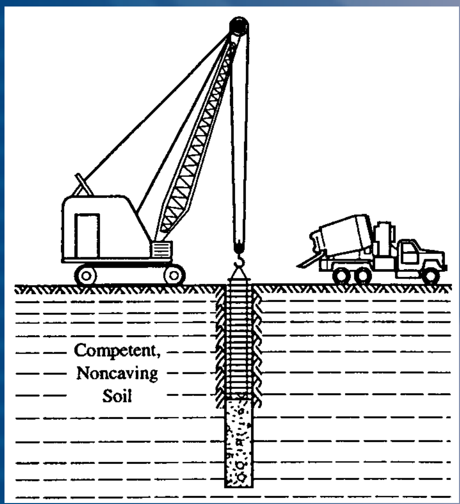
- Construction Procedure (Non-caving soil):
  - Excavate the shaft using a drill rig
  - Fill the lower portion with concrete
  - Place the prefabricated reinforcing cage
  - Fill the shaft with concrete

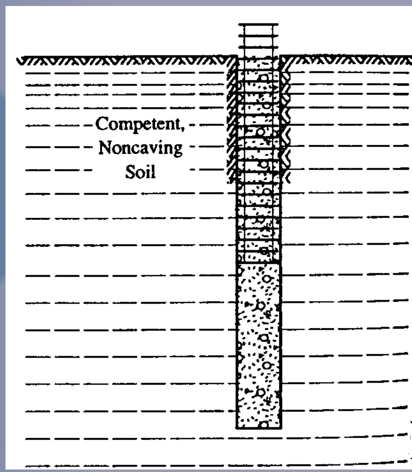
DRILLED SHAFT CONSTRUCTION





DRILLED SHAFT CONSTRUCTION

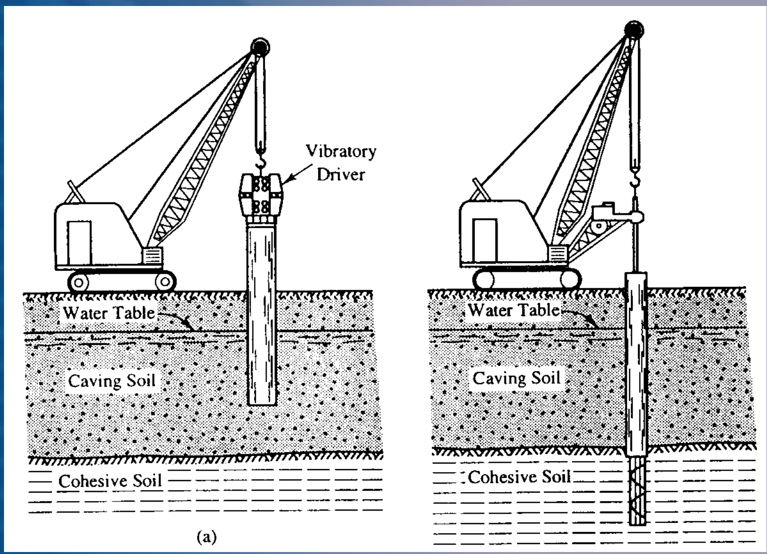




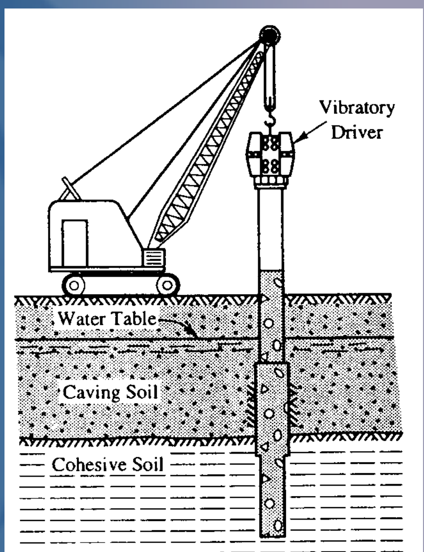
DRILLED SHAFT CONSTRUCTION USING CASING

- Construction in Caving soils (using casing)
  - Drill the hole as before until the caving soil stratum is encountered
  - Insert casing through the caving soil stratum
  - Drill through the caving soil stratus (inside the casing) into non-caving soil
  - Place reinforcement and concrete and then extract casing

DRILLED SHAFT CONSTRUCTION USING CASING



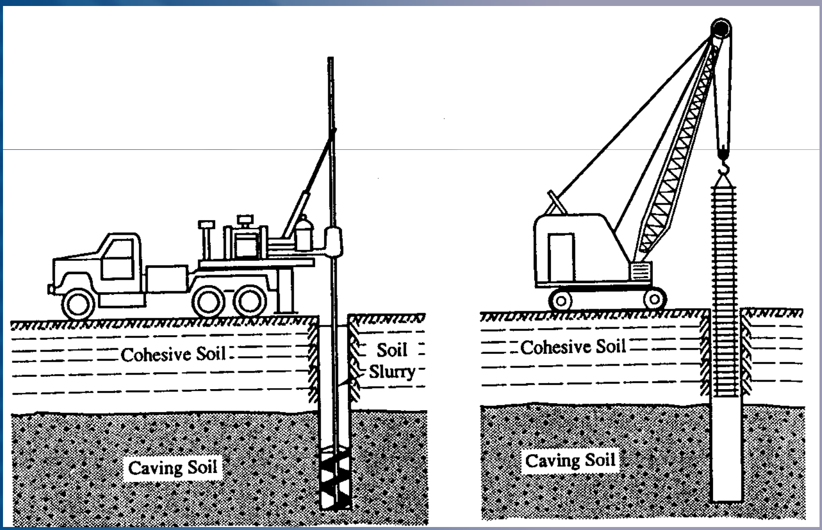
DRILLED SHAFT CONSTRUCTION USING CASING



DRILLED SHAFT CONSTRUCTION USING SLURRY

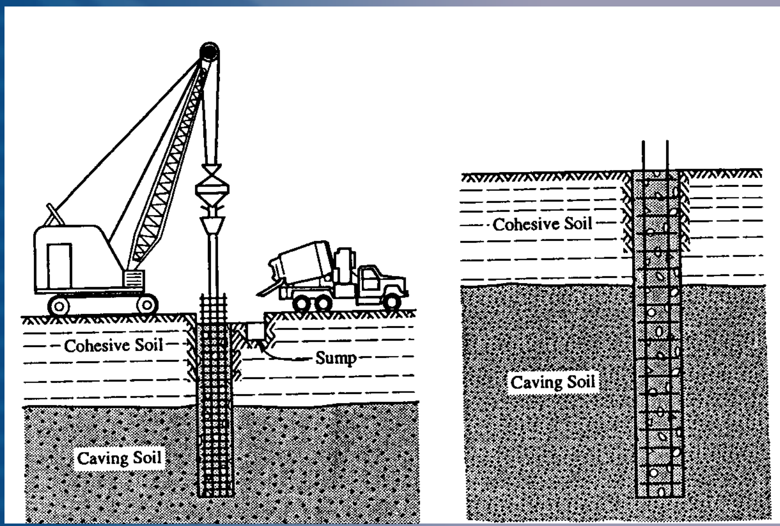
- Construction in Caving soils (using slurry)
  - Drill a starter hole (approx 10 ft deep)
  - Fill with slurry (bentonite+water)
  - Continue to drill through the slurry; keep adding slurry
  - Place reinforcing cage
  - Place concrete using a tremie pipe; slurry will get displaced
  - Messy Operation!

DRILLED SHAFT CONSTRUCTION USING SLURRY

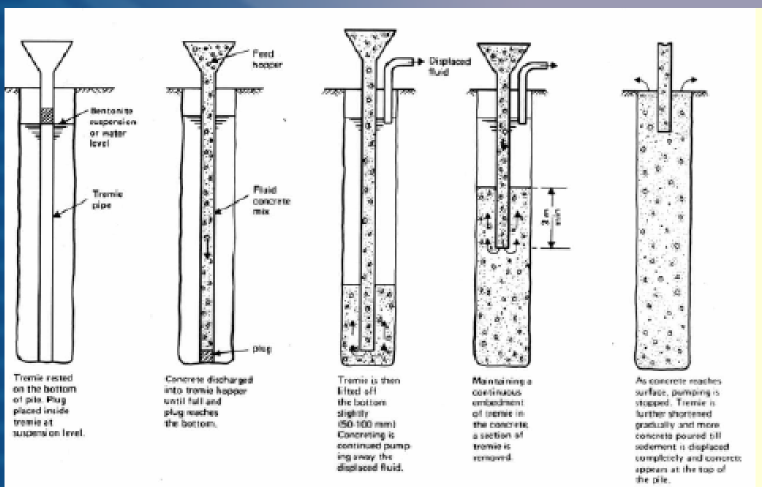


DRILLED SHAFT CONSTRUCTION USING SLURRY

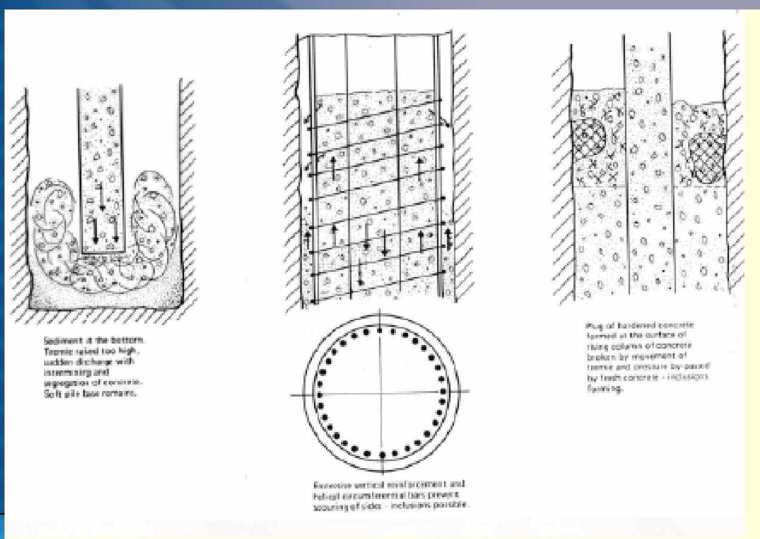
23



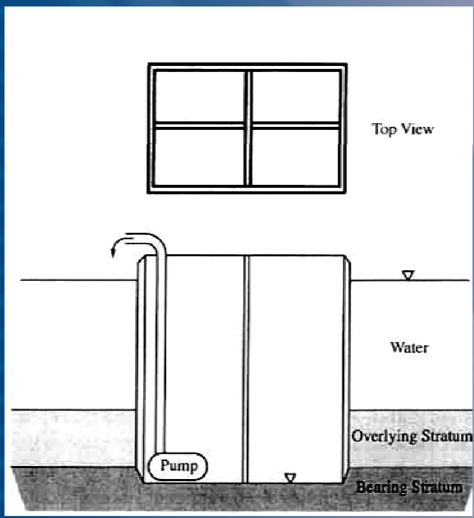
TREMIE CONCRETING

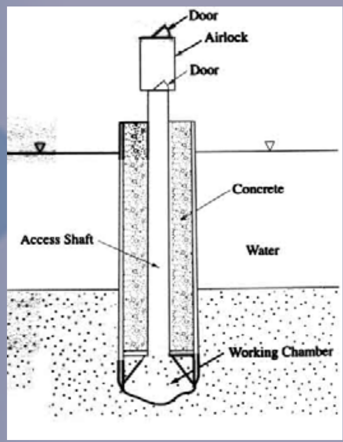


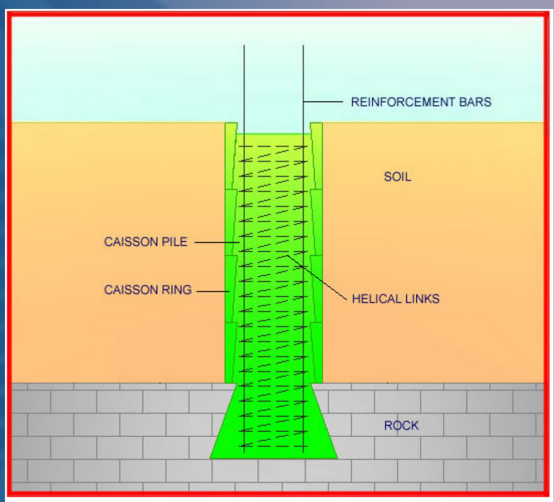
Tremie concreting



CAISSONS


Top View
Water
Overlying Stratum
Pump
Bearing Stratum


Door
Airlock
Door
Concrete
Access Shaft
Water
Working Chamber


REINFORCEMENT BARS
SOIL
CAISSON PILE
CAISSON RING
HELICAL LINKS
ROCK

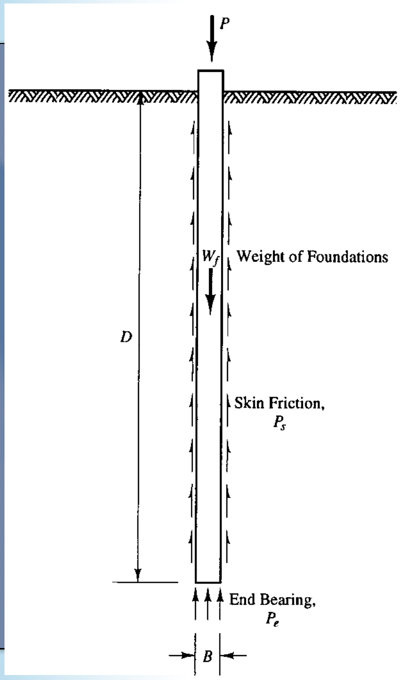
BORED PILE/ DRLLED SHAFT AS RETAINING STRUCTURE

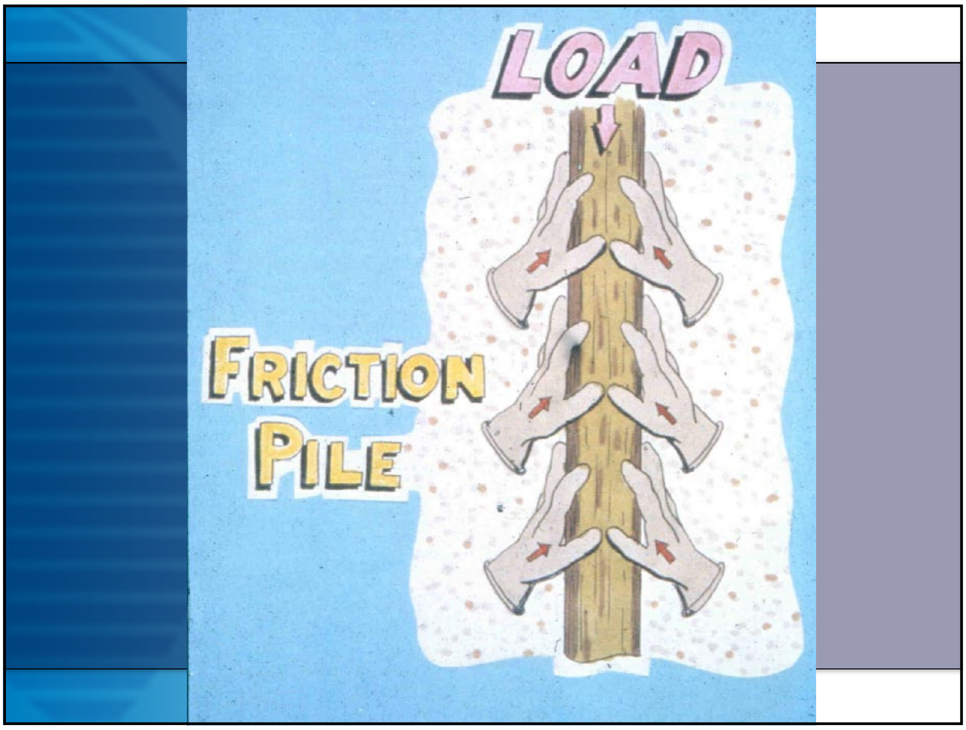
RETAINING STRUCTURES

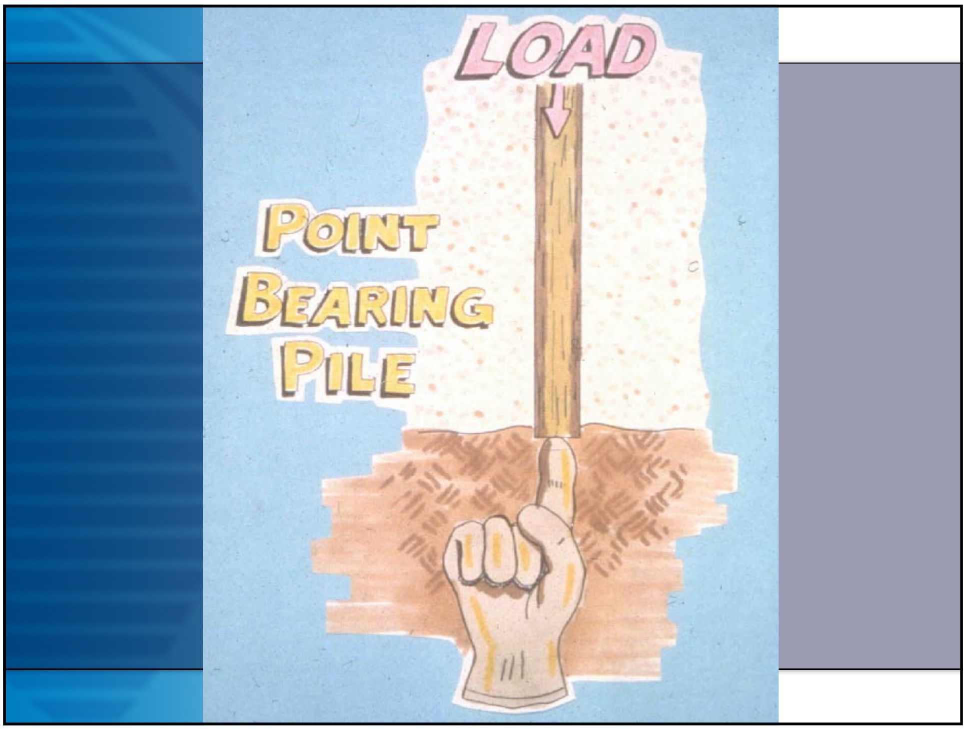
- Contiguous Bored Pile Wall
- Secant Bored Pile Wall
- Contiguous Hand Hug Caisson Wall

LOAD TRANSFER

Load Transfer




LOAD
FRICTION
PILE


LOAD
POINT
BEARING
PILE

SAFETY FACTOR AND PILE DESIGN

- In general – FOS = 2 ~ 3
- Subjected to:-
  - Information
  - Ground condition

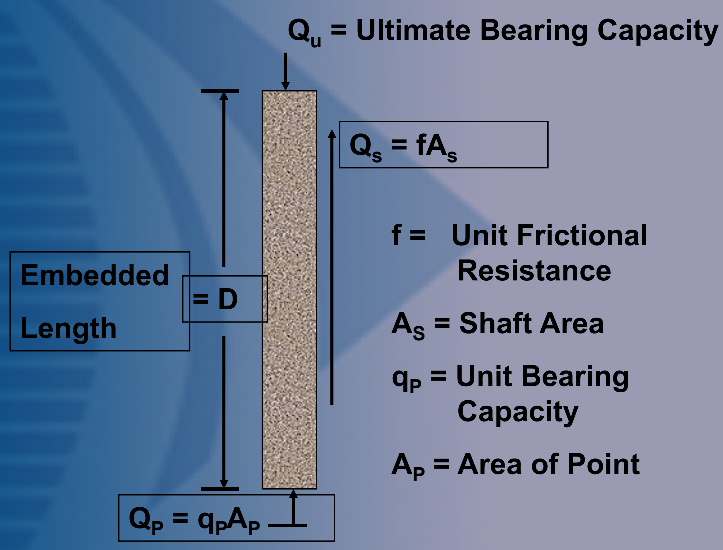
FACTOR OF SAFETY (PILES)

Construction Control Method[a]	Factor of Safety, *F*: Downward Loading (Hannigan et al., 1997)	Factor of Safety, *F*: Upward Loading
Static load test with wave equation analysis	2.00[b]	3.00[b]
Dynamic testing with wave equation analysis	2.25	4.00
Indicator piles with wave equation analysis	2.50	5.00
Wave equation analysis	2.75	5.50
Pile driving formula[c]	3.50	6.00

FACTOR OF SAFETY (DRILLED SHAFTS)

Design Information			Factor of Safety, *F*	
Static Load Test	Soil Conditions	Site Characterization Program	Downward Loading	Upward Loading
Yes	Uniform	Extensive	2.00[a]	3.00[a]
Yes	Erratic	Average	2.50	4.00
No	Uniform	Extensive	2.50	5.00
No	Uniform	Average	3.00	6.00
No	Erratic	Extensive	3.00	6.00
No	Erratic	Average	3.50	6.00

Ultimate Bearing Capacity - Static Formula Method ($Q_u = Q_p + Q_s$)



SINGLE PILE

- Allowable
- Qa= Qp/ FOS + Qs/FOS
- Qa = (Qp +Qs) / FOS

END BEARING

- $Q_p = f_p A_p = (CN_c + \gamma\ L\ N_q + 0.5\gamma B N_\gamma)\ A_p$
- Where
- fp = max unit resistance of base
- Ap = Base Area
- L = Pile length
- In general, End Bearing is ignored bored pile, but full end bearing in Hand Dug Caisson

SHAFT RESISTANCE

- $Ps = \int_0^L fs \text{ as } dL$

Where
fs = Max unit shaft eresistance

= Ca + σh tan φa

As = perimeter area of pile
Σh = normal stress against Pile
Φa = friction angle along pile shaft

PILE IN CLAY (UNDRAINED)

- **Φ = 0, Nq=1, N** γ = 0, c= Cu
- Qu = Qp + Qs
  = fpAp + fs As = Cu Nc Ap + ca As
  - From Skempton Chart Nc=9

"α" METHOD

- $F_s = c_a = \alpha C_u$
- Depending on type of piles, ground, pile length, method of installation.
- High plasticity pile: O.Cons. And Nor. Cons. α = 1

- Medium to low plasticity

Cu ksf	α
<0.5	1
0.5 – 1.5	1 – 0.5
> 1.5	0.5

- α reduces with increase of Over consolidation ratio because
- A) Over consol. Give less confining preesure to pile
- B) contraction cause less/improper contact of soil and pile

α-Method ($f_s = \alpha\, s_u$)

- API Function

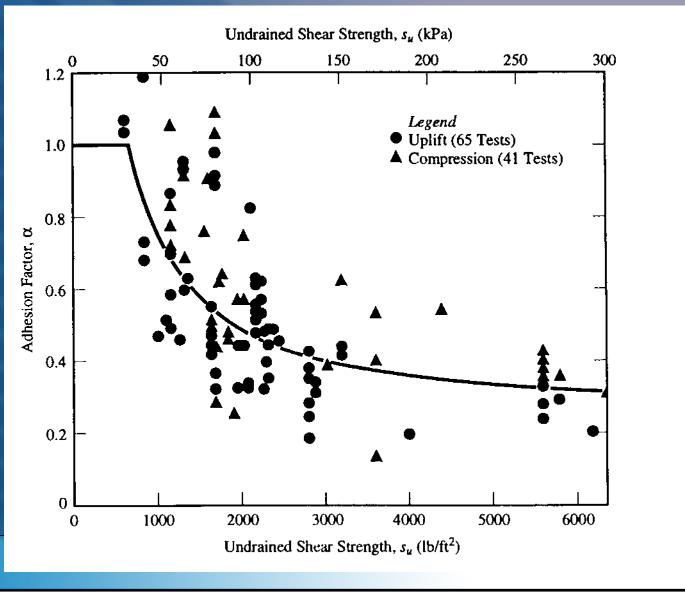
For s_u <25 kPa (500 psf) $\alpha = 1.0$

For 25 kPa (500 psf) < s_u <75kPa (1500psf)

$$\alpha = 1.0 - 0.5\left(\frac{s_u - 500\,psf}{1000\,psf}\right)$$

For s_u > 75 kPa (1500 psf) $\alpha = 0.5$

α-Method ($f_s = \alpha\ s_u$); Drilled Shafts



ESTIMATING UNIT-SIDE FRICTION RESISTANCE, F_S

- Effective Stress Analysis (β-Method)
  - Sands
  - Gravels
  - Silts and Clays

$$\boxed{f_s = \beta\ \sigma'_z}$$

β-METHOD (SANDS)

For large displacement piles, Bhushan(1982)

$$\beta = 0.18 + 0.65D_r$$

β-METHOD (SANDS)

For drilled shafts with $N_{60} \geq 15$, O'Neill & Reese (1999)

$$\beta = 1.5 - 0.135\sqrt{z} \qquad 0.25 \leq \beta \leq 1.20 \qquad \text{(English)}$$

$$\beta = 1.5 - 0.245\sqrt{z} \qquad 0.25 \leq \beta \leq 1.20 \qquad (SI)$$

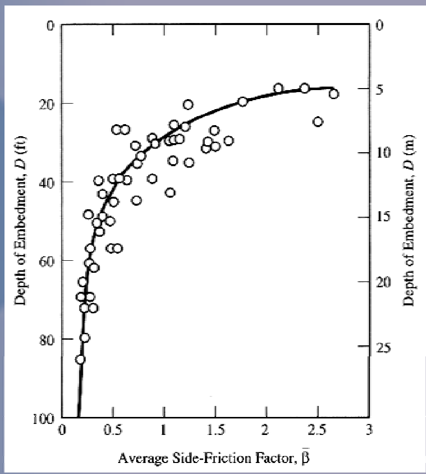
Subject to maximum value of f_s of 4000 psf (190 kPa)

If N_{60}<15 then multiply above β by $N_{60}/15$

β-METHOD (SANDS)

For Auger-Cast Piles, Neely (1991)

Do not divide into layers



$$\overline{f_s} = \overline{\beta}\overline{\sigma'} \le 140\ kPa\ (2800\,psf)$$

β-METHOD (GRAVELS)

Rollins, Clayton, and Mitchell (1997)

For 50% or more gravel size particles

$$\beta = 3.4\ e^{-0.026z} \quad 0.25 \le \beta \le 3.00 \quad \text{(English)}$$

$$\beta = 3.4\ e^{-0.085z} \quad 0.25 \le \beta \le 3.00 \quad (SI)$$

For 25-50% gravel size particles

$$\beta = 2.0 - 0.061z^{0.75} \quad 0.25 \le \beta \le 1.80 \quad \text{(English)}$$

$$\beta = 2.0 - 0.15z^{0.75} \quad 0.25 \le \beta \le 1.80 \quad (SI)$$

SPT METHOD (MEYERHOF)

Unit end bearing $q_p = 40N\,(D_b/B) <= 400N$

Unit friction $f_s = s\,N_{ave}$

$= 2\,N_{ave}$

For Mayerhof method s= 2

Pile capacity design in soil in accordance to MEYERHOF (1976)

Nos. of typical Soil layer		=	4	layers			
(reduced from SPT)							
N value fo layer	1	=	6		Depth up to =	7	m
N value fo layer	2	=	15		Depth up to =	12	m
N value fo layer	3	=	60		Depth up to =	20	m
N value fo layer	4	=	80		Depth up to =	30	m
N value fo layer	No valid!	=	-		Depth up to =	-	m
Recommended pile size dia.		=	700	mm			
Area of pile base		=	0.385	m²			
Perimeter of pile		=	2.20	m			
Safety factor		=	4				
Working Load		=	3000	kN			
type of pile (driven = d; borepile = b)		=	b				

Depth	SPT	SPT ave.	Area of base	Area of shaft	Qbase	Q[illegible]	Qu	Q[illegible]	Lp [illegible]
m	N	N ave.	m²	m²	kN	kN	kN	kN	m
1.5	6	6	0.385	3.299	304.8	19.8	324.6	81.1	114
3.0	6	6	0.385	6.597	304.8	79.2	384.0	96.0	114
4.5	6	6	0.385	9.896	304.8	118.8	423.5	105.9	114
6.0	6	6	0.385	13.195	304.8	158.3	463.1	115.8	114
7.5	15	8	0.385	16.493	762.0	257.3	1019.3	254.8	45
9.0	15	9	0.385	19.792	762.0	356.3	1118.2	279.6	45
10.5	15	10	0.385	23.091	762.0	455.2	1217.2	304.3	45
12.0	15	11	0.385	26.389	762.0	554.2	1316.2	329.0	45
13.5	60	16	0.385	29.688	3048.0	950.0	3998.0	999.5	11
15.0	60	20	0.385	32.987	3048.0	1345.9	4393.8	1098.5	11
16.5	60	24	0.385	36.285	3048.0	1741.7	4789.7	1197.4	11
18.0	60	27	0.385	39.584	3048.0	2137.5	5185.5	1296.4	11
19.5	60	30	0.385	42.883	3048.0	2533.4	5581.4	1395.3	11
21.0	80	33	0.385	46.181	4064.0	3061.2	7125.1	1781.3	9
22.5	80	36	0.385	49.480	4064.0	3589.0	7652.9	1913.2	9
24.0	80	39	0.385	52.779	4064.0	4116.7	8180.7	2045.2	9
25.5	80	41	0.385	56.077	4064.0	4644.5	8708.5	2177.1	9
27.0	80	44	0.385	59.376	4064.0	5172.3	9236.3	2309.1	9
28.5	80	45	0.385	62.675	4064.0	5700.1	9764.1	2441.0	9

SUB SURFACE INVESTIGATION

- Based on the following criterion:-
  - Soil Type
  - Method of Analysis

DATA REQUIRED FROM SI

- SPT – Meyerhof Method
- Vane shear test, UU – α Method
- CPT (mainly for soft soil)

SEC 14.6 GROUP EFFECTS

- Piles/Auger-cast piles may be installed in groups
- Why?
  - Single pile does not give sufficient capacity
  - Low degree of precision in "spotting"
  - Multiple piles provide redundancy
  - Lateral soil compression produced by pile groups is greater-therefore total capacity higher

SEC 14.6 GROUP EFFICIENCY

Load Capacity of Pile Group (P_{ag})

$= \eta$ x N x Capacity of single pile (P_a)

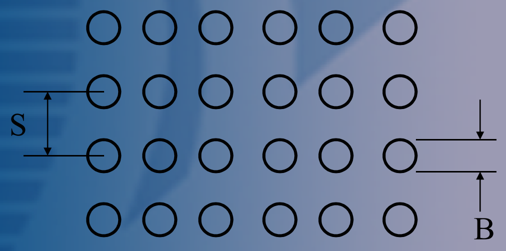
η depends on:

- soil type (sands or clays)
- pile diameter/pile spacing ratio
- construction procedures (pre-drilling, jetting etc.)
- elapsed time since pile driving
- mode of failure
- Pile slenderness (l/d)

Group Efficiency, η

- Converse-Labarre Formula

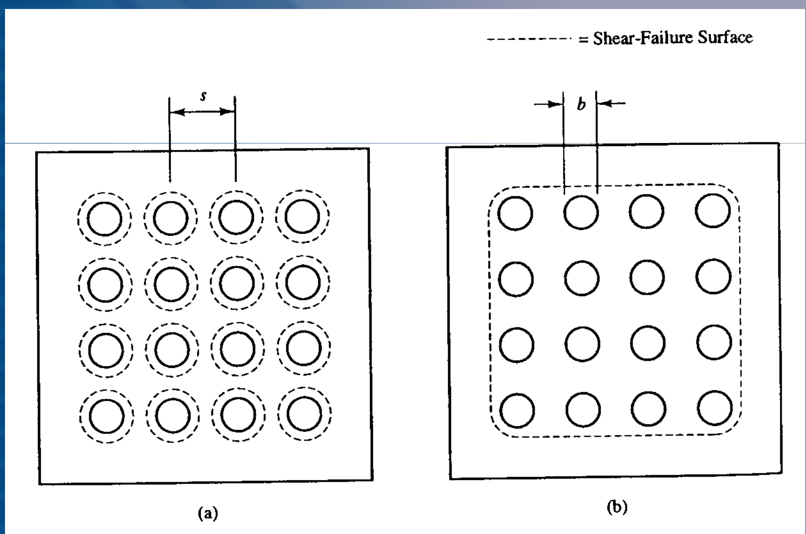
$$\eta = 1 - \theta\left[\frac{(n-1)m + (m-1)n}{90\, m\, n}\right]$$



m = 4

n = 6

INDIVIDUAL VS. BLOCK FAILURE



WHEN BLOCK FAILURE GOVERNS…

$$\eta = \frac{2s(m+n)+4B}{\pi mnB} \leq 1$$

- Note that the above equations have been widely used but are not based on any hard data

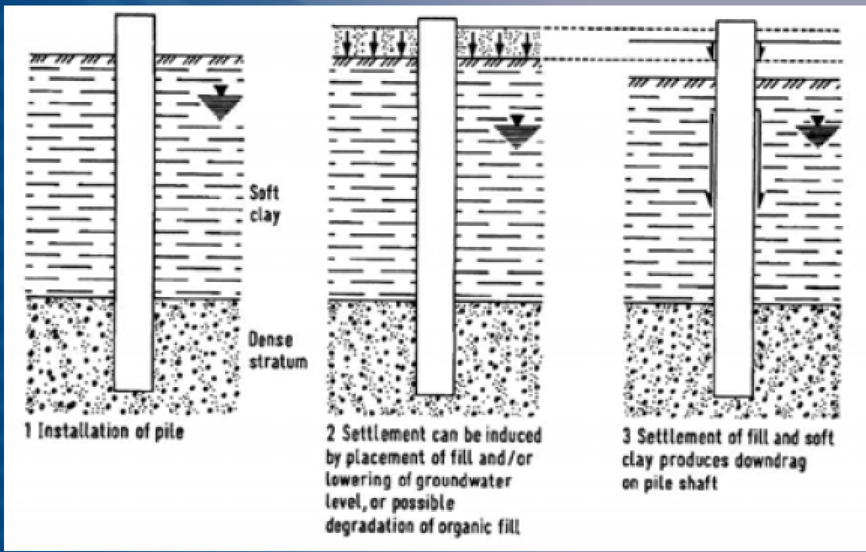
---

- $Q_p = BL\,(C_b N_c + \gamma L N_q + 0.5\gamma B N_\gamma)$
- $P_s = 2D\,(B+L)\,(c^* + \sigma_h \tan\Phi_a')$
- Where;
- C_b = cohesion at base
- C^* = average cohesion along pile shaft.

- η=(Block resistance)/ Sum of Single pile resistance)

NEGATIVE SKIN FRICTION

- Due to consolidation of recent filled material or any original soil that subjected to consolidation process.



- Distribution depending on: -
- A) relative movement between soil and pile.
- Relative movement of compressible soil below pile foundation
- Elastic compression of pile
- Degree of consolidation of soil stratum
- N.S.F take place when soil downward movement > pile

- Neutral point – point where relative movement =0
- Ration of neutral points depth of pile in compressible soil ~ 0.75.
- N.K.F will cause pile to be overloaded and may cause distress to structures above the foundation.

BJERRUM METHOD

- Fsn= Kσv′ tanφ′ +χPe
- Where

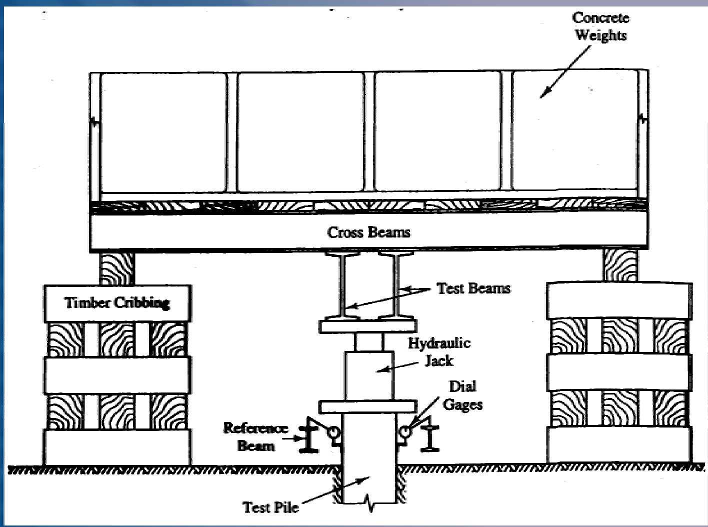
- Fsn= unit N.S.F
- σv′ = effective filled pressure
- φ′ = effective friction angle
- K = coeeficient of lateral earth pressure
- χ = factor depending on rate of loading
- Normally χPe is small and ignored.

Tyep of soil	φ’	K	Unit N.K.F
Silty	30º	0.45	0.25 σv’
Low plasticity	20º	0.5	0.2 σv’
High plasticity	15º	0.55	0.15 σv’

SETTLEMENT OF DEEP FOUNDATIONS

- Settlement of deep foundations, when designed based on axial load capacity considerations, is typically less than 0.5 in
- Pile groups may have larger settlements, but still within acceptable limits
- Therefore, in practice engineers generally do not perform settlement analysis for deep foundations
- However, settlement analysis may be necessary in certain special situations

FULL SCALE STATIC LOAD TESTS



FULL SCALE STATIC LOAD TESTS

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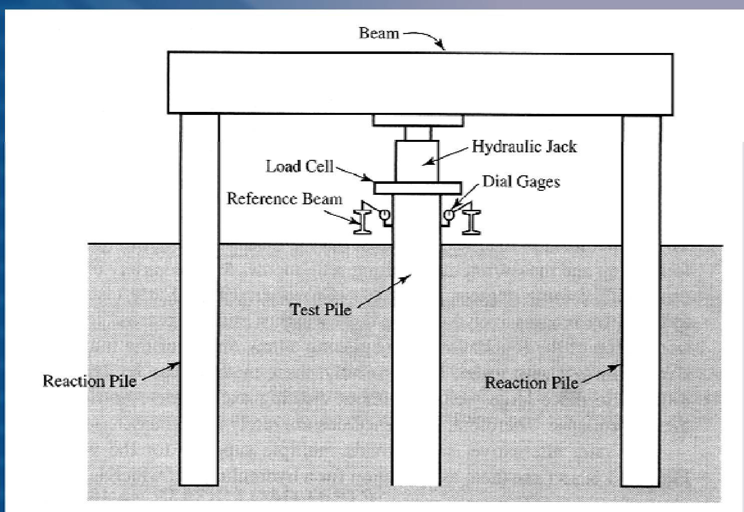
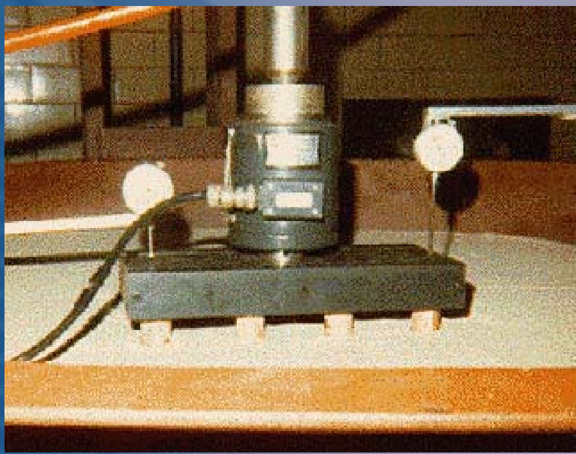
FULL SCALE STATIC LOAD TESTS



FULL SCALE STATIC LOAD TESTS

FULL SCALE STATIC LOAD TESTS

FULL SCALE STATIC LOAD TESTS

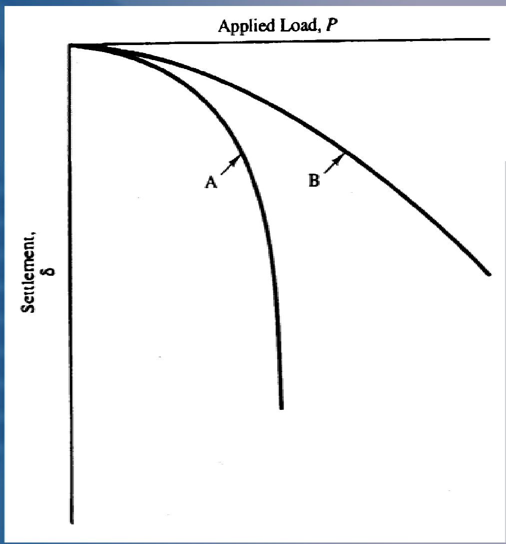


FULL SCALE LOAD TESTS

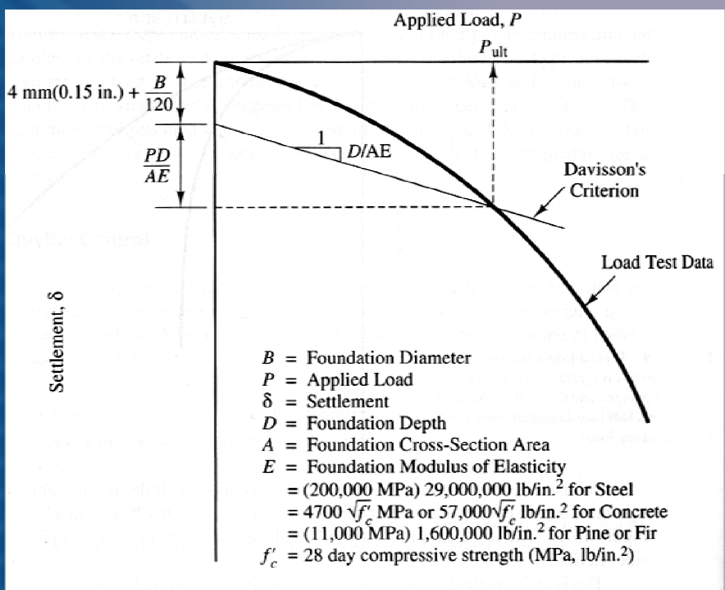
- Controlled Stress Tests
  - Maintained Load Tests (ML Tests)
    - Slow ML Tests (hold load for 1-2 hours)
    - Quick ML Tests (hold load for 2.5-15 minutes)
- Controlled Strain Tests
  - Constant Rate of Penetration Test
  - Constant Settlement Increment Test

INTERPRETATION OF TEST RESULTS

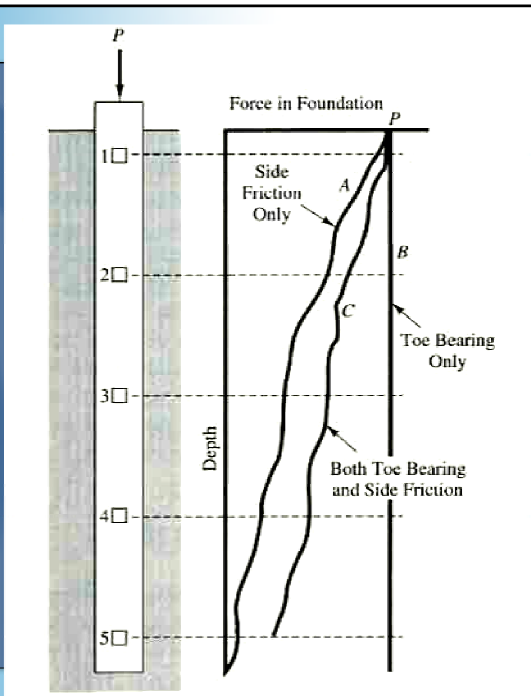
85



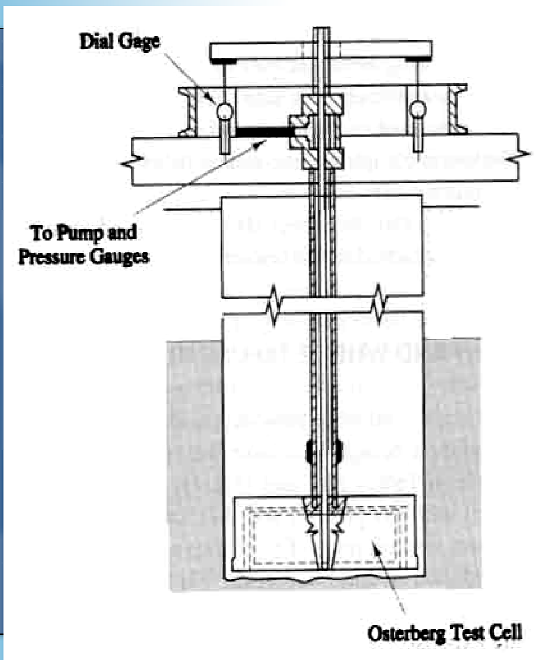
DAVISSON'S METHOD



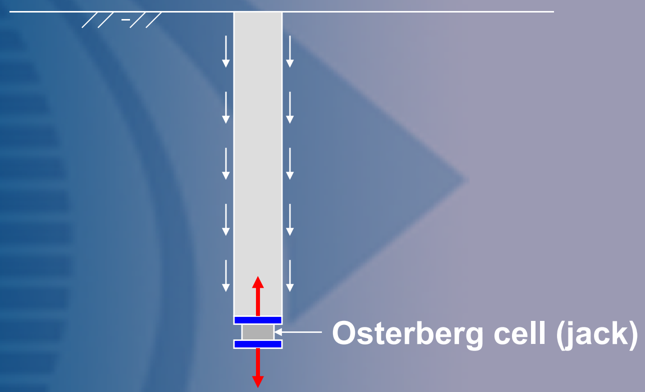
Use of Strain Gauges



Osterberg Load Tests



OSTERBERG-CELL PILE LOADING TEST



Advantage: separate tip resistance from side resistance

Disadvantage: expensive

O-CELL PILE LOADING TEST

Reed et al

O-CELL PILE LOADING TEST

Reed et al

O-CELL PILE LOADING TEST

Reed et al

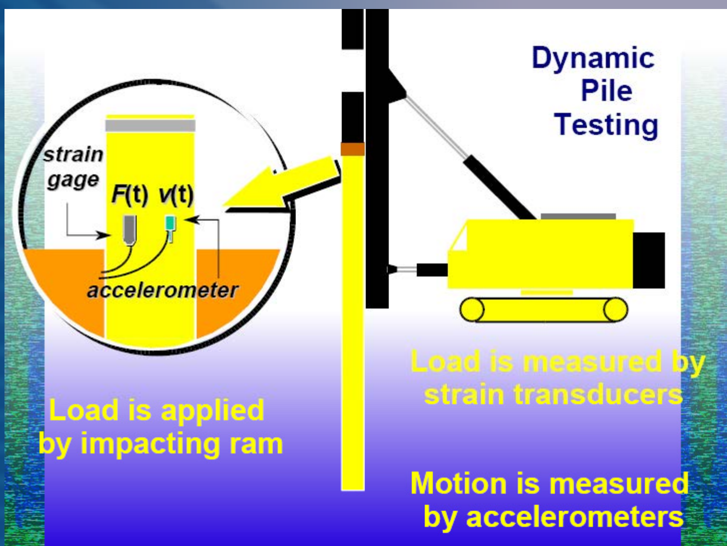
O-CELL PILE LOADING TEST

O-CELL PILE LOADING TEST

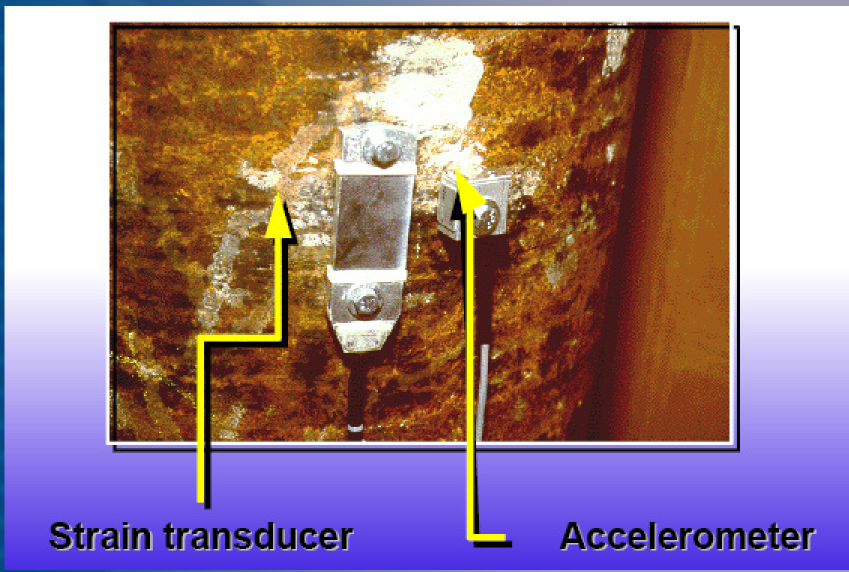
STRAIN GAUGE ATTACHED ON REBAR

Reed et al

PILE DRIVING ANALYZER (PDA)



STRAIN AND ACCELERATION MEASUREMENT



INSTALLING GAUGES

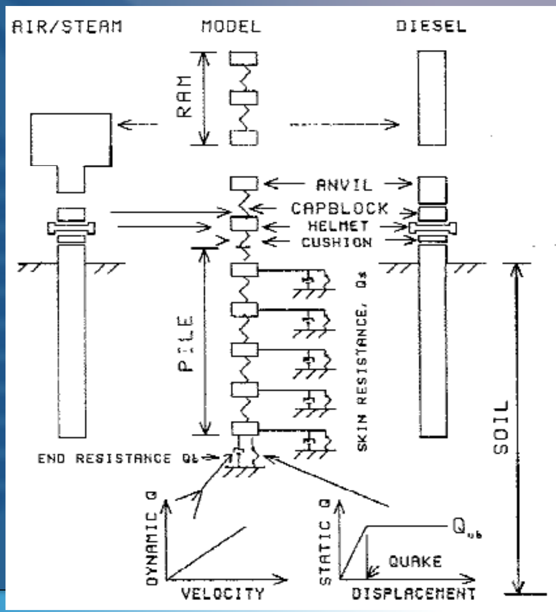
CalTran

PDA TESTING

CalTran

PDA READOUT

PILE DRIVING AND WAVE PROPAGATION THEORY



PILE DRIVING FORMULAS

Method	Equation for P_u (kips)	FS
Gates	$27\sqrt{E_h E_r}\,(1-\log_{10} S)$	3
Pacific coast Uniform Building Code	$\frac{12E_h E_r C_{p1}}{S+C_{p2}}$, $C_{p2}=\frac{W_r + C_p W_p}{W_r + W_p}$, $C_{p2}=\frac{P_u}{AE_p}$ C_p = 0.25 for steel piles or 0.10 for other piles Initially assume C_{p2} = 0 and compute P_u; reduce P_u by 25%, compute C_{p2}, then recompute P_u; Compute a new C_{p2}, compute P_u until P_u used = P_u computed	4
Danish	$\frac{12E_h E_r}{S+C_d}$, $\sqrt{\frac{144E_h E_r L}{2AE_p}}$ inches	3-6
Engineering News Record	Drop Hammers $\frac{12W_r h}{S+1.0}$	6
	Other Hammers $\frac{24W_r h}{S+1.0}$	6

A=area of pile cross section (ft^2), E_h=hammer efficiency; E_p=pile modulus of elasticity (ksf) E_r=manufacturer's hammer-energy rating (or $w_r h$) (kips-ft); h=height of hammer fall (ft); L=pile length (inches); S=average penetration in inches/blow for last 5 to 10 blows for drop Hammers and 10 to 20 blows for other hammers; W_r=weight of striking parts of ram (kips); W_p=weight of pile including pile cap, driving shoe, capblock and anvil for double-acting Steam hammers (kips)

PILE INTEGRITY TEST LOW STRAIN(PIT)

- It detects potentially dangerous defects such as major cracks, necking, soil inclusions or voids
- PIT Testing is performed with a hand held hammer, a sensitive accelerometer and the PIT Tester
- A compressive wave is generated by tapping the pile head with a hammer.
- When the downward compression wave encounters a change in cross section or in concrete quality, it generates an upward tension wave that is obtained at the pile top

PIT

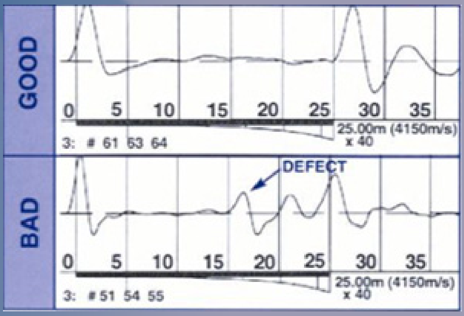
- The velocity recorded along with the subsequent reflections from the pile top or pile discontinuities are graphically displayed
- The effectiveness of the system is limited to pile length not exceeding 30 to 60 pile diameters

PIT

- BENEFITS:

  * No advance planning
  * Quick and Economical
  * Can verify every pile on site

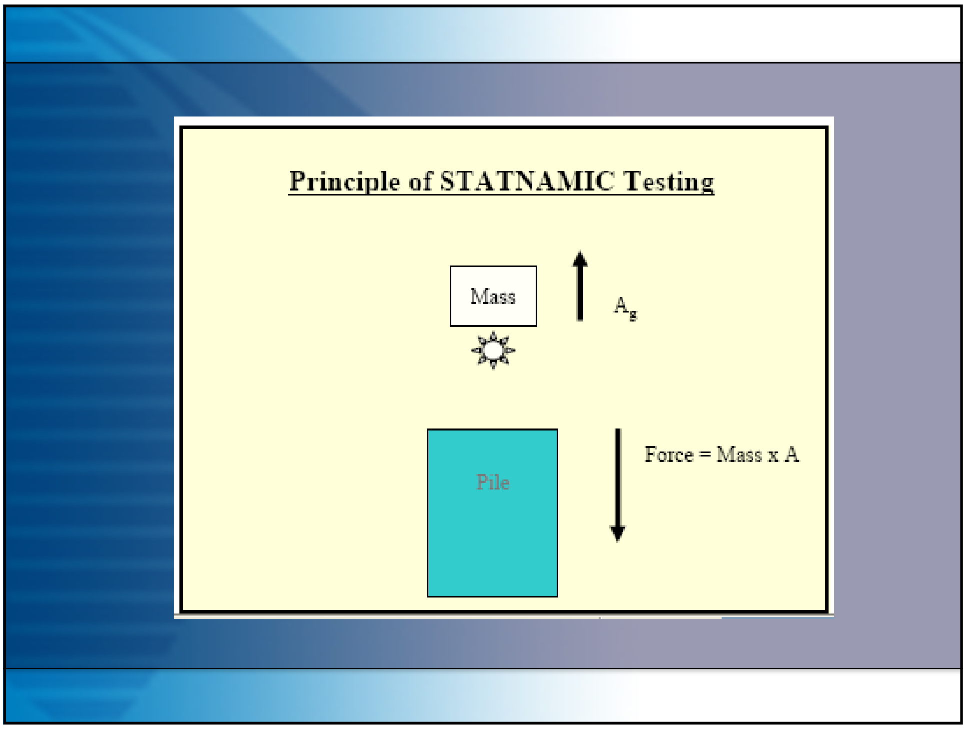
PIT

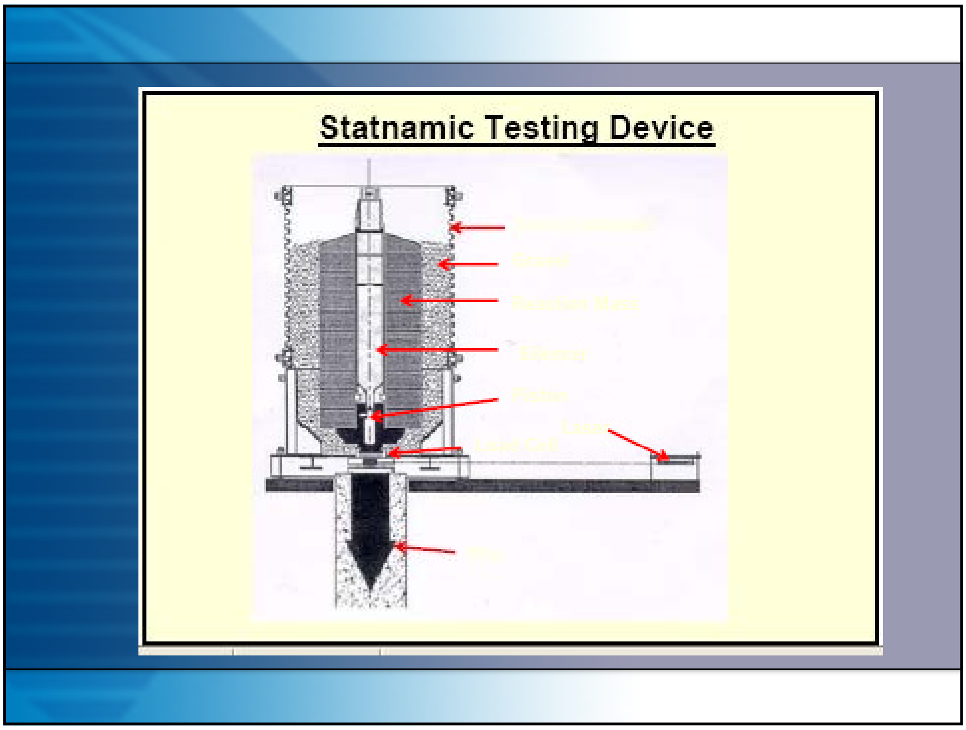


STANAMIC TEST (STATIC + DYNAMIC)

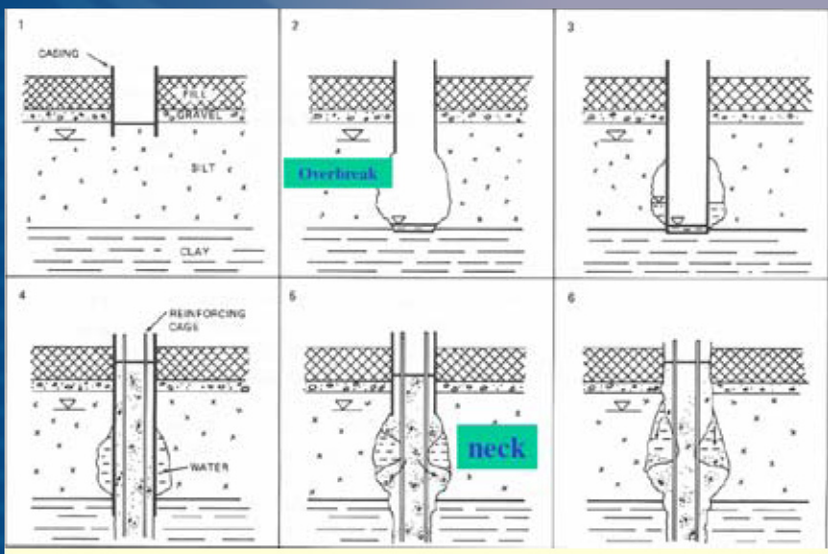
- Using the action and reaction concept
- Is an electronic explosive method used to measure pile capacity, combining Static and dynamic loading
- The explosion is contained so that it pushed the reaction mass up and the pile downward.

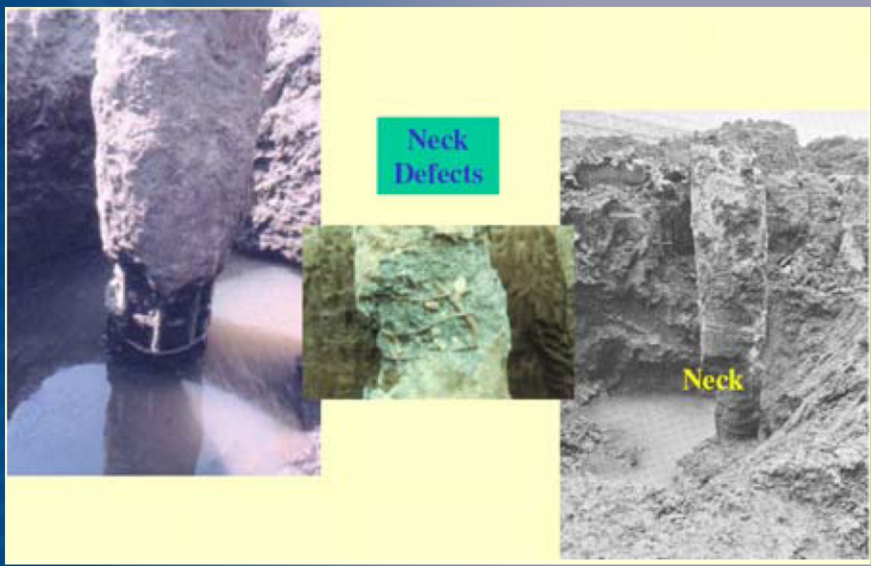
- Developed tp allow "cost effective load test of high capacity piles
- Required mobilisation of a reaction mass typically 5% of the required test load

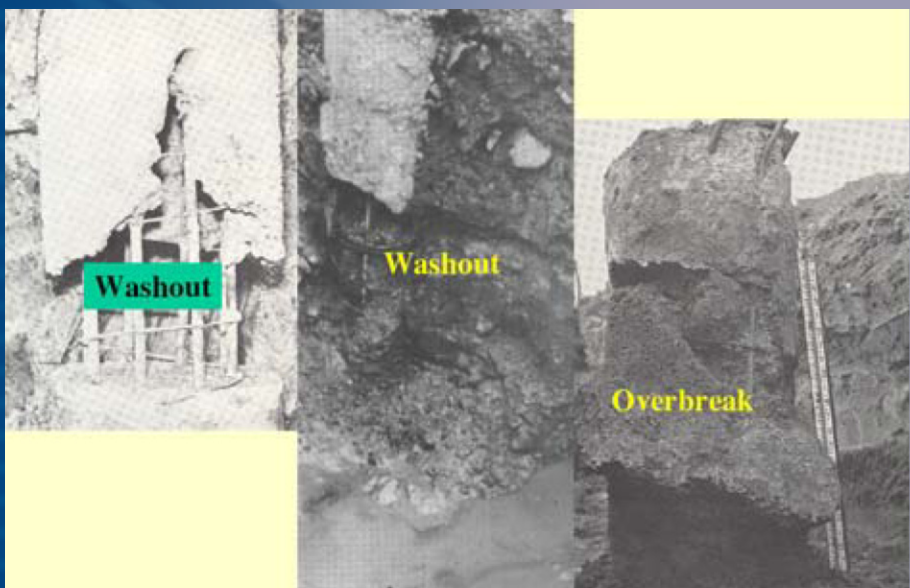

Principle of STATNAMIC Testing
Mass
A_g
Pile
Force = Mass x A

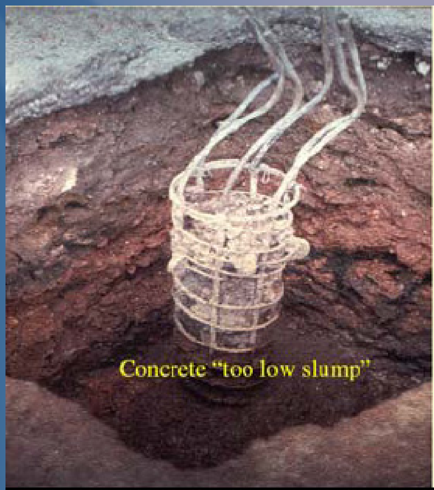

Statnamic Testing Device

COMMON DEFECTS IN BORED PILE




Neck
Defects
Neck


Washout
Washout
Overbreak



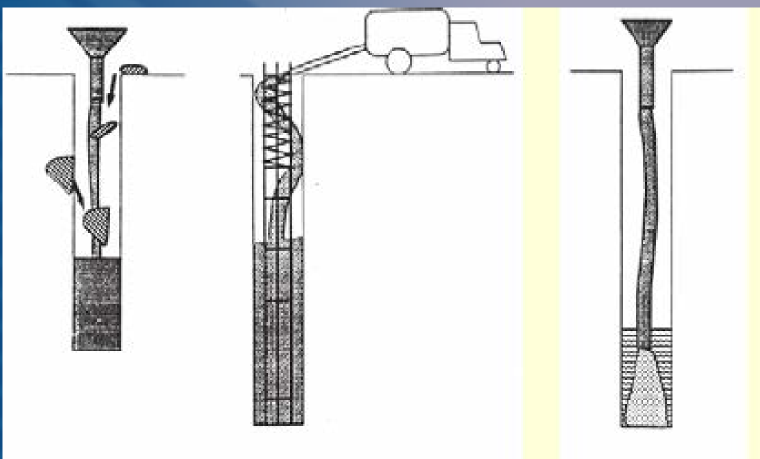


Figure 6-13: Pile Defects
Cave In

Figure 6-14: Pile Defects
Concrete Segregation

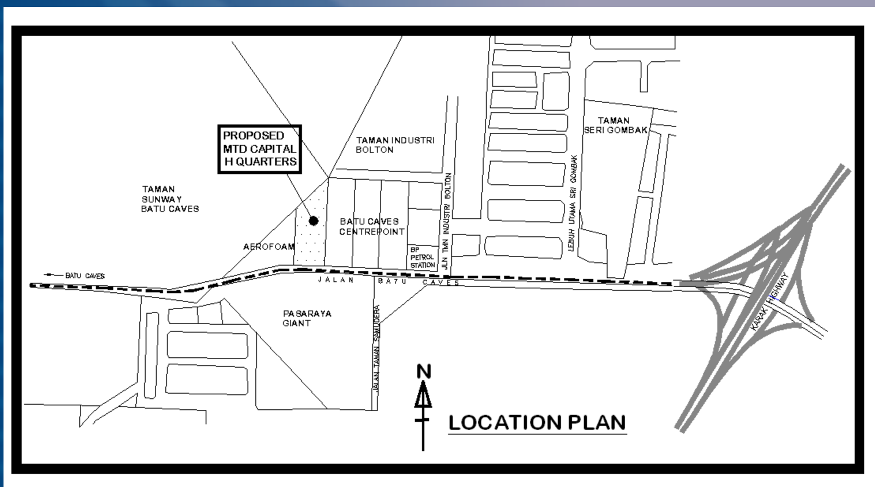
Figure 6-16: Pile Defects
Water in the Hole

CASE STUDIES

CASE STUDY 1:- MTD HEAD QUARTERS

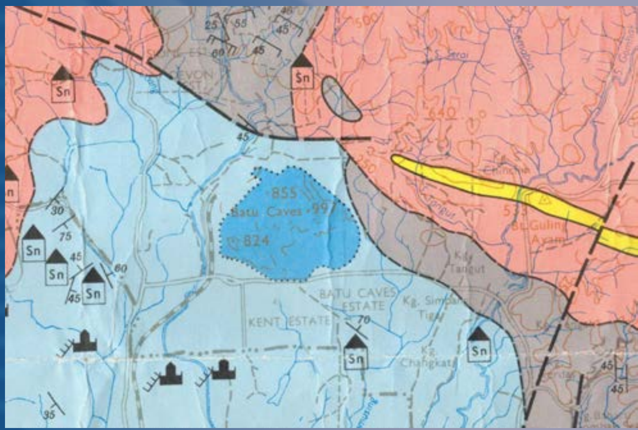
INTRODUCTION

- The proposed development is located at Lot 1381, Mukim Batu, Daerah Gombak, Selangor Darul Ehsan which consists of an existing MTD Capital site office. The existing site office will be removed/relocated prior to the construction of the proposed project. It is bounded by Batu Caves Centrepoint on the east, Rezab Pusat Silat Batu Caves on the north, Aerofoam factory on the west and Giant Hypermarket on the south.
- 14 storey's high rise.



GEOLOGICAL FORMATION

- Kuala Lumpur Limestone Formation



FOUNDATION PROBLEMS IN LIMESTONE FORMATION

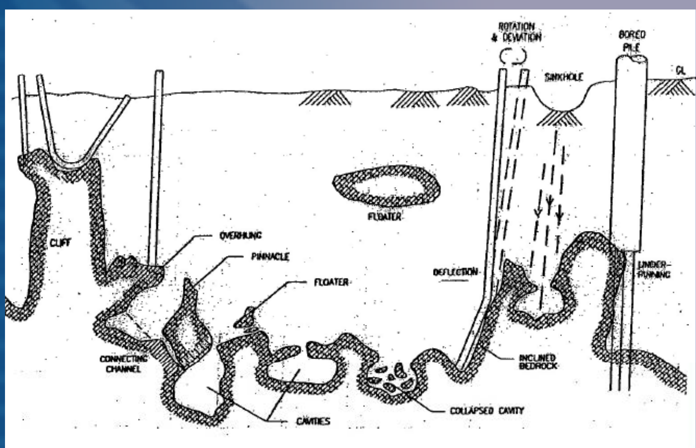
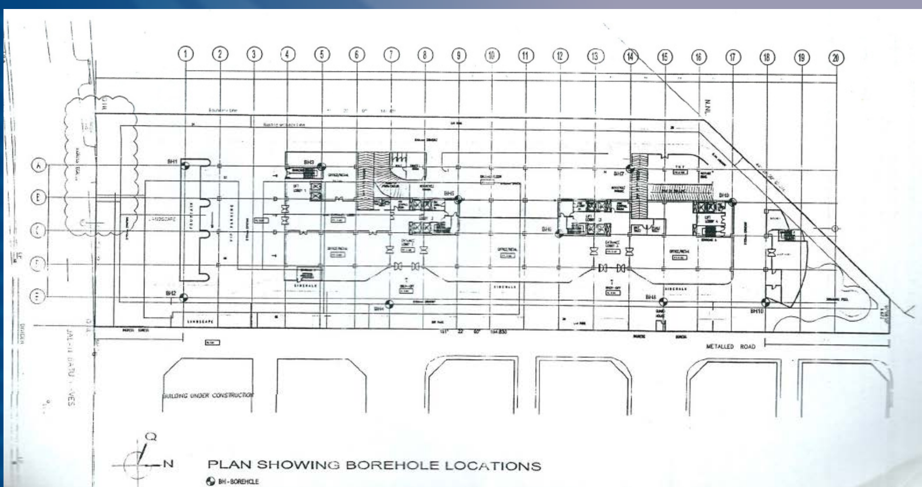


FIG 4: PILING PROBLEMS IN LIMESTONE AREA

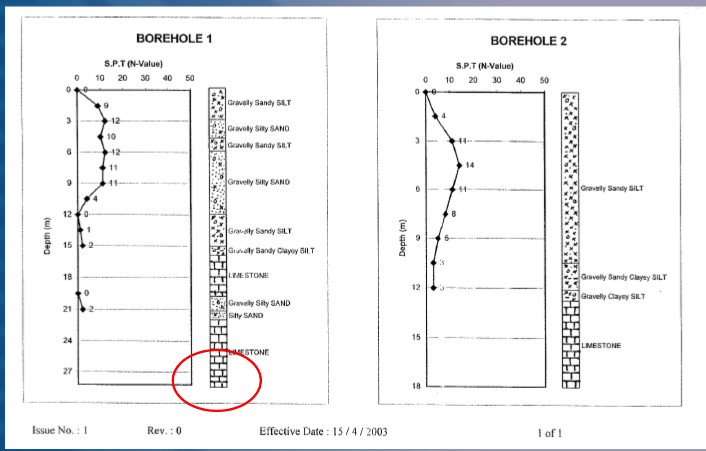
SITE INVESTIGATION

- 10 nos of boreholes were sunk
- Provision for Rock Probing during construction

SI LAYOUT



TYPICAL BORELOGS



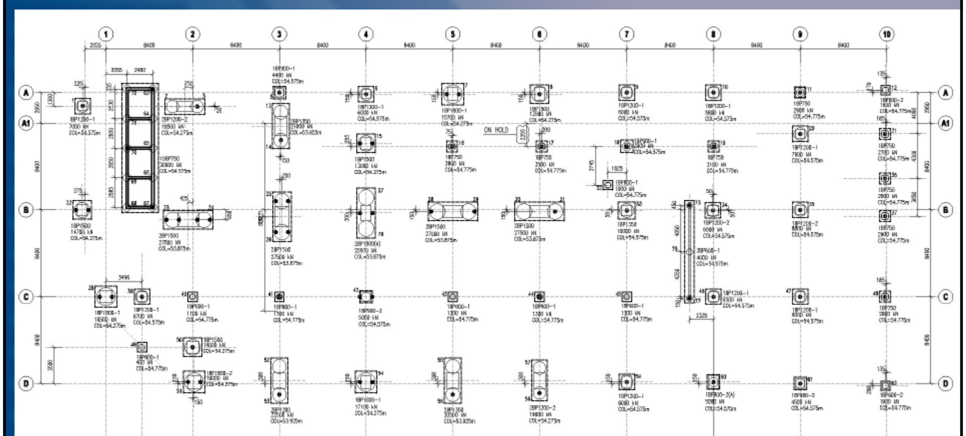
PROPOSED FOUNDATION

- Initial Proposal:- Micropile
- Change to Bored Pile foundation
- Pile Sizes:-
  - 600mm dia.
  - 750mmdia
  - 900mm dia
  - 1200mm dia
  - 1350mm dia
  - 1500mm dia
  - 1800mm dia

BORED PILE, $F_y = 460\ N/mm^2$

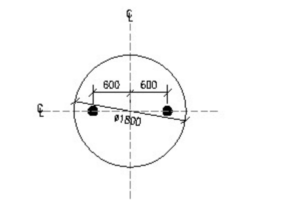
PILE MARKING	PILE DIAMETER (mm)	CONCRETE GRADE	WORKING LOAD (kN)	REINFORCEMENT				SOCKET LENGTH (m)
				TOP 12m STEEL CAGE		REMAINING STEEL CAGE		
				MAIN BARS	HELICAL LINK	MAIN BARS	HELICAL LINK	
BP600-1	600mm ø	35 N/mm^2	1,450	8 T20	R10-250mm C/C	4 T20	R10-250mm C/C	0.5m
BP600-2	600mm ø	35 N/mm^2	2,450	8 T20	R10-250mm C/C	4 T20	R10-250mm C/C	1.1m
BP750	750mm ø	35 N/mm^2	3,850	12 T20	R10-250mm C/C	6 T20	R10-250mm C/C	1.4m
BP900-1	900mm ø	35 N/mm^2	4,450	11 T25	R10-250mm C/C	6 T25	R10-250mm C/C	1.1m
BP900-2	900mm ø	35 N/mm^2	5,550	11 T25	R10-250mm C/C	6 T25	R10-250mm C/C	1.7m
BP1200-1	1200mm ø	35 N/mm^2	7,900	19 T25	R10-200mm C/C	10 T25	R10-200mm C/C	1.4m
BP1200-2	1200mm ø	35 N/mm^2	9,850	19 T25	R10-200mm C/C	10 T25	R10-200mm C/C	2.2m
BP1350	1350mm ø	35 N/mm^2	12,500	24 T25	R10-200mm C/C	12 T25	R10-200mm C/C	2.5m
BP1500	1500mm ø	35 N/mm^2	15,450	18 T32	R10-200mm C/C	9 T32	R10-200mm C/C	2.8m
BP1800-1	1800mm ø	35 N/mm^2	17,800	26 T32	R10-200mm C/C	13 T32	R10-200mm C/C	2.1m
BP1800-2	1800mm ø	35 N/mm^2	22,250	26 T32	R10-200mm C/C	13 T32	R10-200mm C/C	3.3m

LAYOUT PLAN



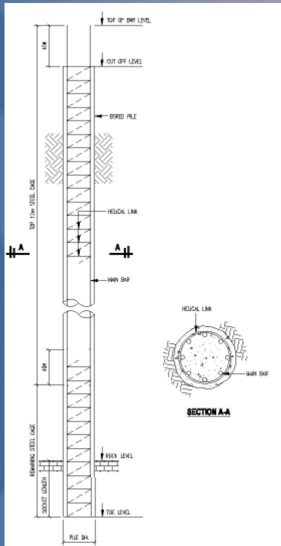
ROCK AND CAVITY PROBING

- Large diameter 2 nos.



TYPICAL PROBE POSITION FOR 1800Ø PILE
SCALE : 1=50

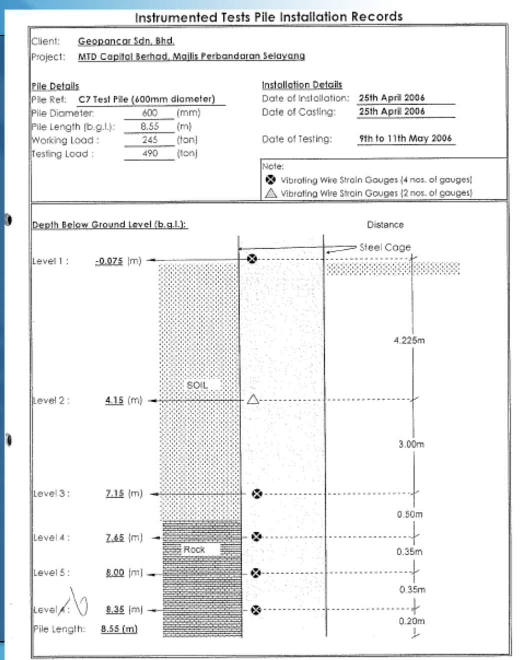
BORED PILE DETAILING



TYPICAL BORED PILE LATERAL SECTION

TEST CARRIED OUT

- 2 nos of Instrumented test pile were carried out
- Strain Gauges were installed at various locations to determine the skin friction and rock socket of the pile.
- Tests were carried out using reaction anchors



Instrumented Tests Pile Installation Records

Client: Geopancar Sdn. Bhd.
Project: MTD Capital Berhad, Majlis Perbandaran Selayang

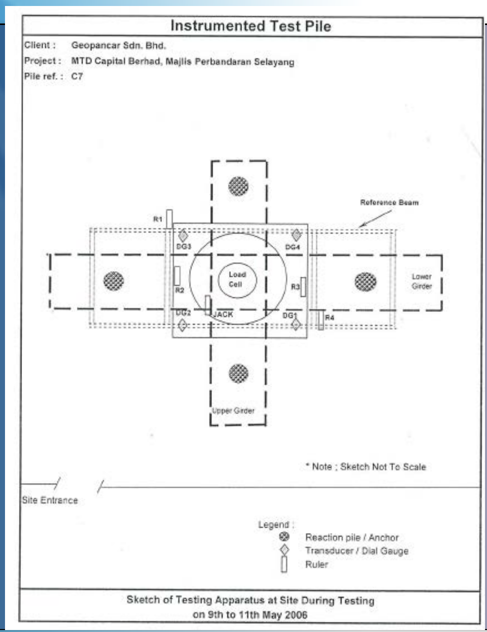
Pile Details

Pile Ref: C7 Test Pile (600mm diameter)
Pile Diameter: 600 (mm)
Pile Length (b.g.l.): 8.55 (m)
Working Load : 245 (ton)
Testing Load : 490 (ton)

Installation Details

Date of Installation: 25th April 2006
Date of Casting: 25th April 2006
Date of Testing: 9th to 11th May 2006

Note:
⊗ Vibrating Wire Strain Gauges (4 nos. of gauges)
△ Vibrating Wire Strain Gauges (2 nos. of gauges)



Case Study 2:- Caisson pile

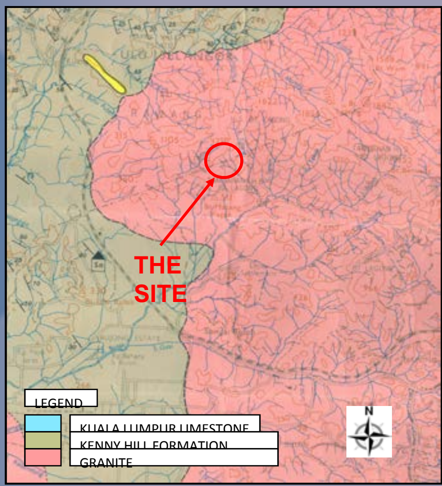
PROPOSED DEVELOPMENT OF 9 UNIT BUNGALOW AT PHASE 2C, TEMPLER PARK, MUKIM RAWANG, DAERAH GOMBAK, SELANGOR DARUL EHSAN UNTUK TETUAN KUMPULAN HARTANAH SELANGOR BERHAD

INTRODUCTION

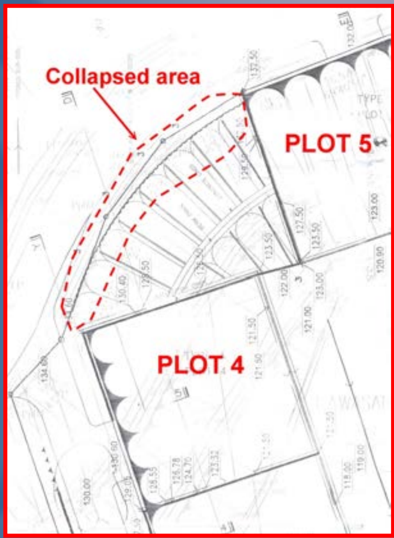
- Mohd Asbi & Associates were invited to provide geotechnical assessment and foundation design for the bungalows.
- Bungalows were designed seated on slope.

GEOLOGICAL FORMATION

- Granite formation

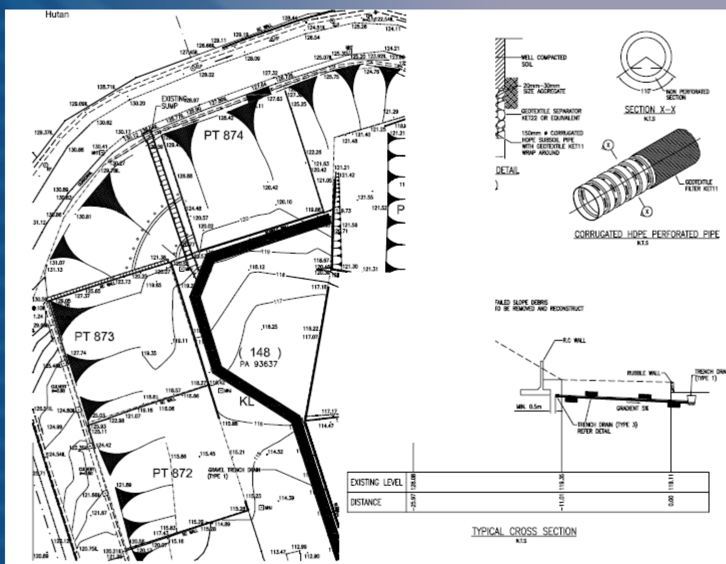


- During the preliminary assessment, slopes in the development were found unstable due to high ground water table.
- Water seepages were found at the toe of these slopes and the slopes were found wet.


Collapsed area
PLOT 5
PLOT 4

SLOPE REMEDIAL

- Deep trench drain was proposed at the toe to stabilized the slope.
- After the slope remedial works, MAA was again requested to study on the foundation system of these bungalows
- A total of two (2) boreholes, thirty six (36) Mackintosh probes and four (4) trial pits were carried out



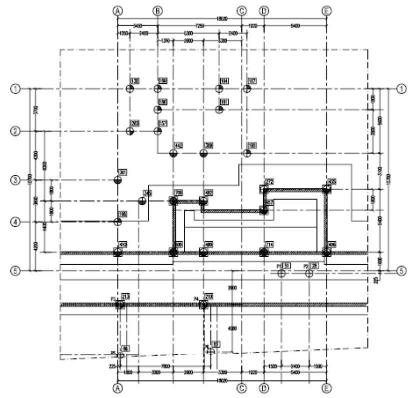
FOUNDATION SELECTION

- In order to withstand any lateral load by soil creep, which may potentially cause distress to the bungalow, hand dug Caissons are proposed to be adopted as the foundation system for this development
- 1m hand dug caisson was proposed with various length (to cater for different capacity

FOUNDATION

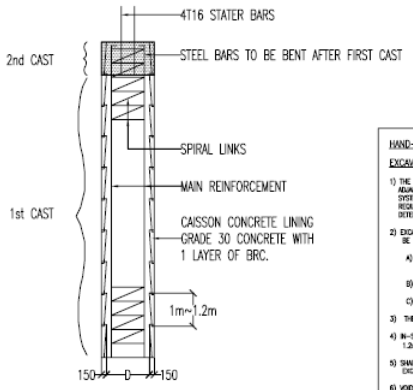
- 1m hand dug caisson was proposed with various length (to cater for different capacity
- The designed lengths were also taking into consideration of potential slip and the termination depth.
- The caisson piles were designed as full end bearing pile (deep footing)

TYPICAL FOUNDATION LAYOUT



TYPE	SIZE DIA. (mm)	GRADE	ESTIMATED LENGTH (m)	ALLOWABLE CAPACITY (kN)	REINFORCEMENT	SPIRAL
⊕	1000	40	3	120	20T20	T12-200
⊕	1000	40	4.5	200	20T20	T12-200
⊕	1000	40	6	300	20T20	T12-200
⊕	1500	40	4.5	500	20T25	T12-200
⊕	1500	40	7.5	900	20T25	T12-200

NOTE:
THIS DRAWING IS ONLY FOR PLOT 3



TYPICAL CAISSION DETAIL

D = CAISSION PILE DIAMETER

HAND-DUG CAISSONS

EXCAVATION FOR HAND-DUG CAISSONS

1) THE CONTRACTOR SHALL ADOPT A METHOD OF CONSTRUCTION THAT WILL NOT CAUSE SETTLEMENT OR DISTURBANCE ANY KIND TO ADJACENT STRUCTURES, PAVEMENTS, PUBLIC OR PRIVATE SERVICES. THE CONTRACTOR SHALL ESTABLISH AN APPROVED MONITORING SYSTEM AND TAKE REGULAR READINGS AND PREPARE AND SUBMIT REPORTS TO THE ENGINEER IN THE FORMAT AND QUANTITY AS REQUESTED. THE CONTRACTOR SHALL MODIFY THE METHOD OF CONSTRUCTION IF THE EFFECTS OF GROUND MOVEMENT ARE DETECTED IN ANY SUCH STRUCTURES, PAVEMENTS AND SERVICES.

2) EXCAVATION FOR HAND-DUG CAISSONS SHALL BE CARRIED OUT USING MANUAL METHODS OR POWER TOOLS. BLASTING SHALL NOT BE USED UNLESS PERMITTED BY THE ENGINEER. IF BLASTING IS PERMITTED:

A) THE POSITION OF BLAST HOLES AND THE SIZE OF CHARGES SHALL BE SUCH THAT SHATTERING OF ROCK BEYOND THE CAISSON IS MINIMISED,

B) THE ROCK FACE SHALL NOT BE SHATTERED WITHIN THE TOE-IN OR BELL-OUT ZONE AT THE BOTTOM OF THE CAISSON, AND

C) THE CAISSON OPENING SHALL BE COVERED TO PREVENT THE PROJECTION OF FRAGMENTS OF MATERIAL.

3) THE STABILITY OF EXCAVATIONS FOR HAND-DUG CAISSONS SHALL BE MAINTAINED WHERE NECESSARY BY LININGS.

4) IN-SITU CONCRETE TAPERED RINGS USED AS PERMANENT LINERS SHALL BE AT LEAST 100 MM THICK AND SHALL NOT EXCEED 1.2m DEEP. THE RINGS SHALL BE CONSTRUCTED WITH WELL-COMPACTED CONCRETE OF GRADE 30/20 OR GREATER.

5) SHAFT LININGS SHALL BE PLACED AS SOON AS PRACTICABLE AND NOT MORE THAN 24 HOURS AFTER EACH INCREMENT OF EXCAVATION IS COMPLETE.

6) VOIDS BETWEEN THE LINING AND FACE OF THE EXCAVATION SHALL BE FILLED WITH CONCRETE OF THE SAME GRADE AS THE LINING OR WITH OTHER MATERIALS AGREED BY THE ENGINEER.

7) ANY UNSTABLE LAYERS OF SUBSOIL ENCOUNTERED SHALL BE STABILIZED BY GROUTING OR SIMILAR METHODS. NO FURTHER EXCAVATION WILL BE PERMITTED UNTIL THE STABILIZATION WORKS ARE COMPLETED.

SEALING AND SCALING OF HAND-DUG CAISSONS

1) LEAKAGE OF GROUNDWATER THROUGH LINERS OR INTO UNLINED SHAFTS OF HAND-DUG CAISSONS SHALL BE STOPPED BY A METHOD AGREED BY THE ENGINEER.

2) LOOSE ROCK ON THE FACES OF UNLINED SHAFTS SHALL BE SCALED OFF AND REMOVED BEFORE CONCRETING.