IGS University Online Lecture Series

Geosynthetic Reinforced Pile Platforms for Soft Foundations

Rajagopal Karpurapu

Professor & PK Aravindan Institute Chair

Department of Civil Engineering

Indian Institute of Technology Madras

Chennai, India 600 036

gopalkr@iitm.ac.in

www. geosyntheticssociety.org

About the lecturer

The lecturer is a Professor of Civil Engineering & PK Aravindan Institute Chair at the Indian Institute of Technology Madras, Chennai, India. He has been teaching at the institute for more than 25 years in the areas of geotechnical and geosynthetics engineering. He had supervised the research work of several post-graduate students and doctoral candidates. He had provided consultancy services for several construction projects of retaining walls, steep embankments, ground improvement projects, foundations and tunnelling. He had served as the President of Indian Chapter of IGS during 2008-2010 & served as a council member of IGS during 2010-2018. He had received number of best paper awards for papers published in different journals. He is a member of several important committes related to geotechnical and geosynthetics in India.

Outline of the Lecture

- Introduction
- Geosynthetic Pile Platforms
- Arching Theory
- Centrifuge modelling of embankments
- Numerical analyses
- Case Histories
- Design Methods
- Summary

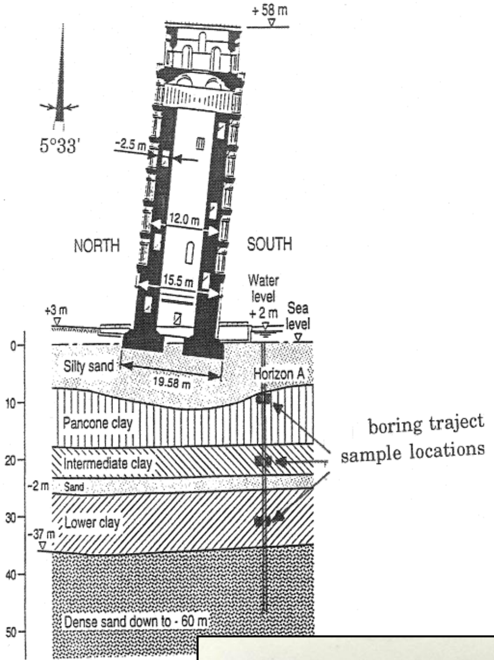
Outline of the Lecture

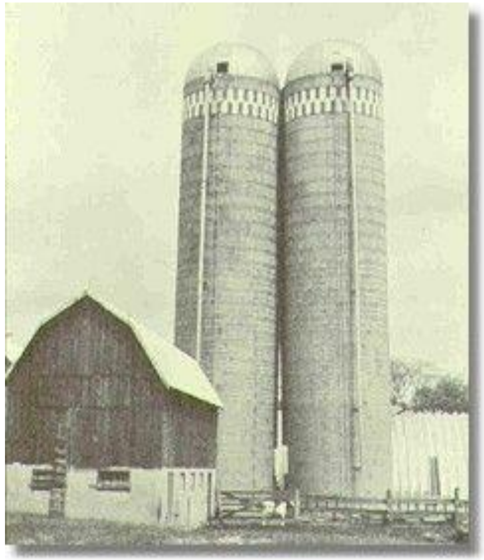
- **Introduction**
- Geosynthetic Pile Platforms
- Arching Theory
- Centrifuge modelling of embankments
- Numerical analyses
- Case Histories
- Design Methods
- Summary

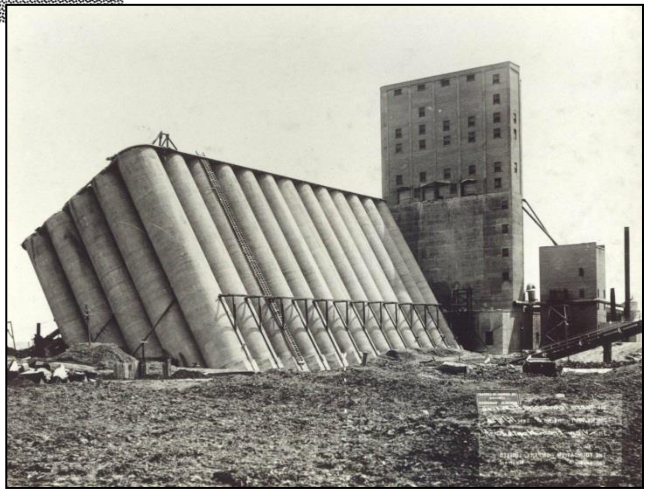
Problems with construction in soft clays

- Low bearing capacity
- Large total and differential settlements
- Large lateral flows leading to slip circle failure
- Consolidation settlements over time

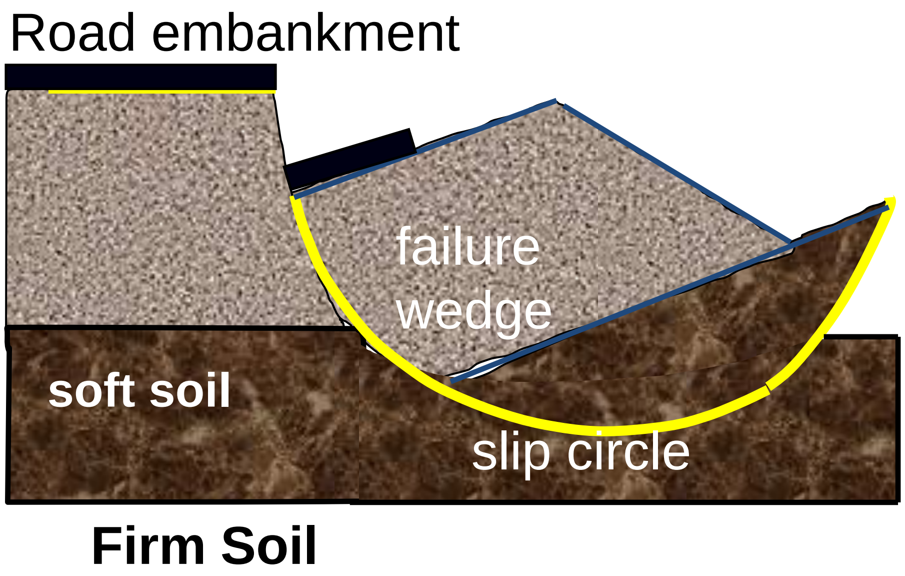
Problems with construction in soft clays







What happens when foundation soil flows laterally?



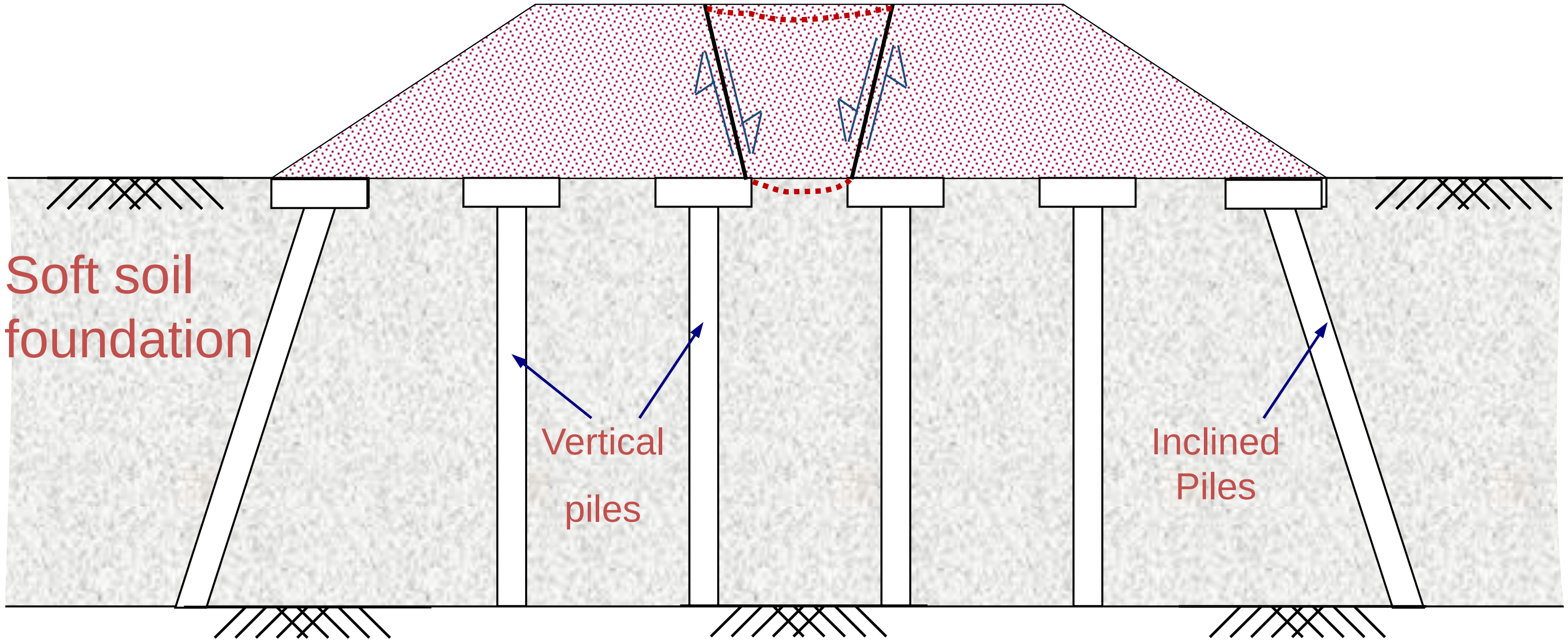
Solutions for soft foundation soils

- Remove & replace
- Pre-consolidation
- Grouting techniques
- Granular columns
- Geosynthetic encased granular columns
- Light weight fills for construction
- Basal reinforcement
- Pile elements to support embankment
- Geosynthetic reinforced pile platforms

Outline of the Lecture

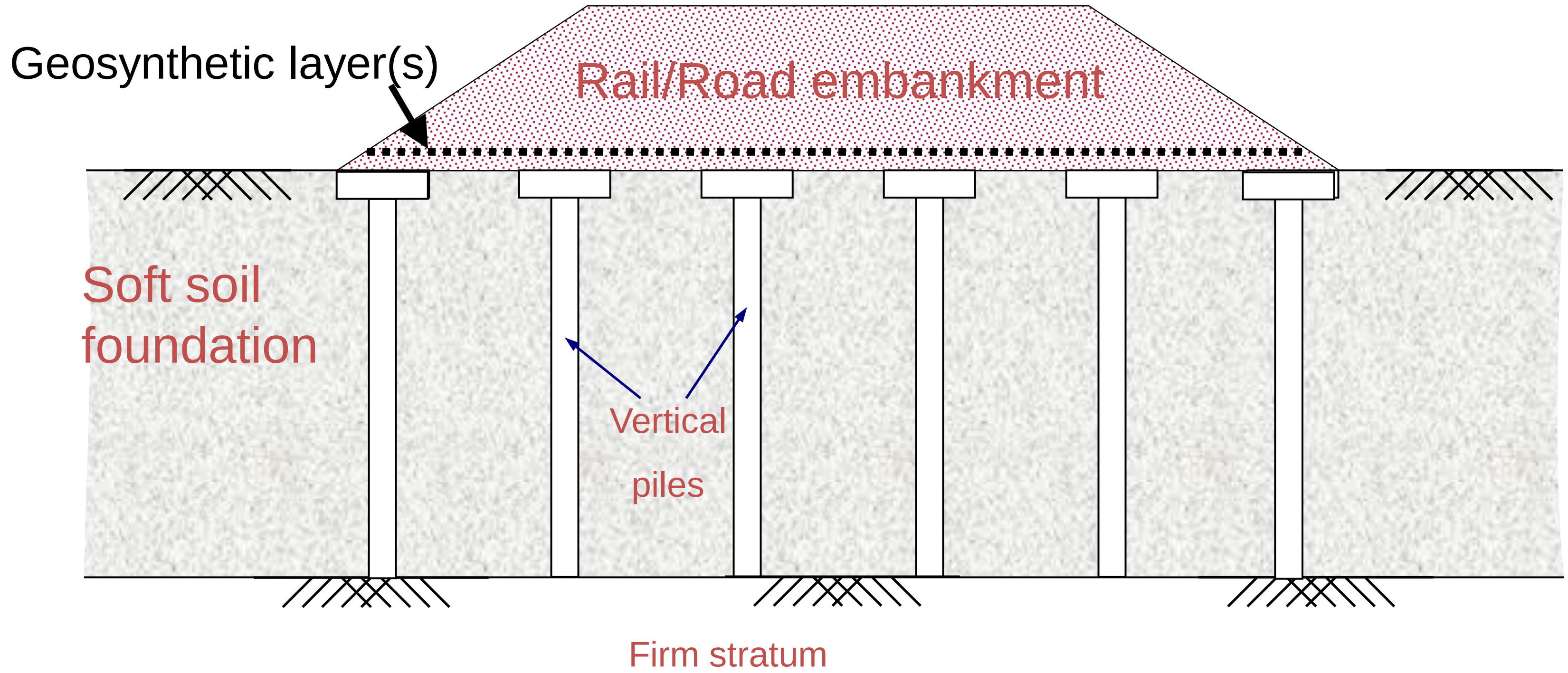
- Introduction
- **Geosynthetic Pile Platforms**
- Arching Theory
- Centrifuge modelling of embankments
- Numerical analyses
- Case Histories
- Design Methods
- Summary

Embankment with pile & pile cap support

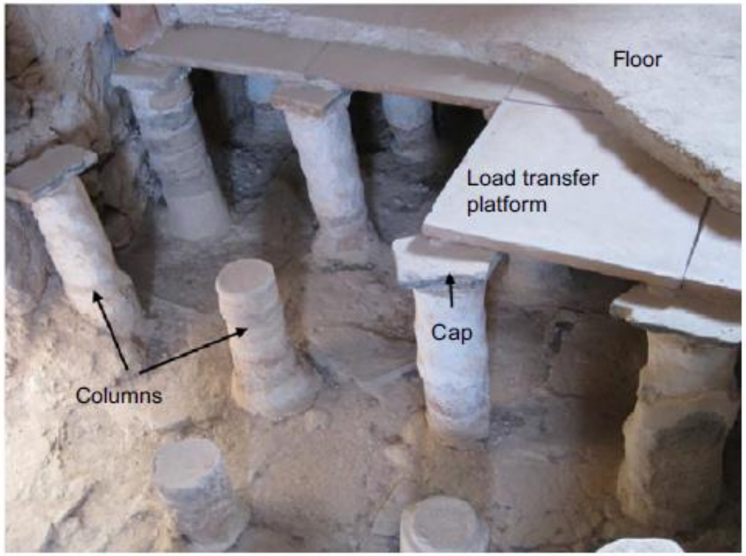


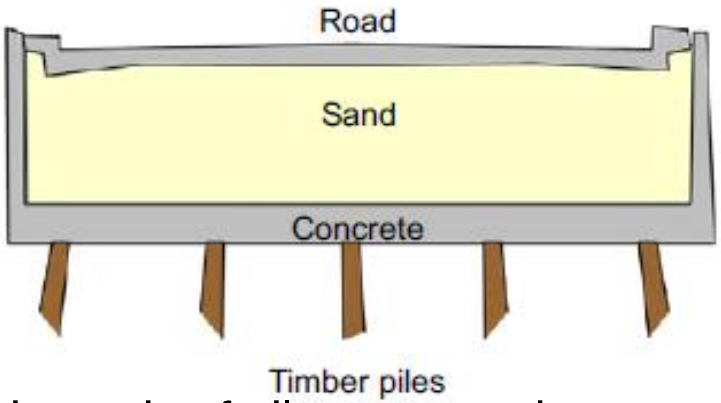
- Inclined piles at ends to support lateral thrust from embankment
- Large pile caps to reduce span between piles
- Differential deformations promote soil arching in embankment soil
- Larger fraction of embankment load transferred to piles than the soil resting directly above the pile caps

Geosynthetic reinforced piled embankment



- Geosynthetic reinforcement supports lateral thrust and piles can be vertical at ends
- Larger spacing between piles & smaller pile caps
- Lower settlements in soil at both surface and pile cap levels
- Larger load transfer into the piles
- Much lesser pressures on the foundation soil





Schematic of pile supported road embankment in Holland in 1930's

Column supported floor slab in a bath house in Masada, Israel (37-31 BCE)

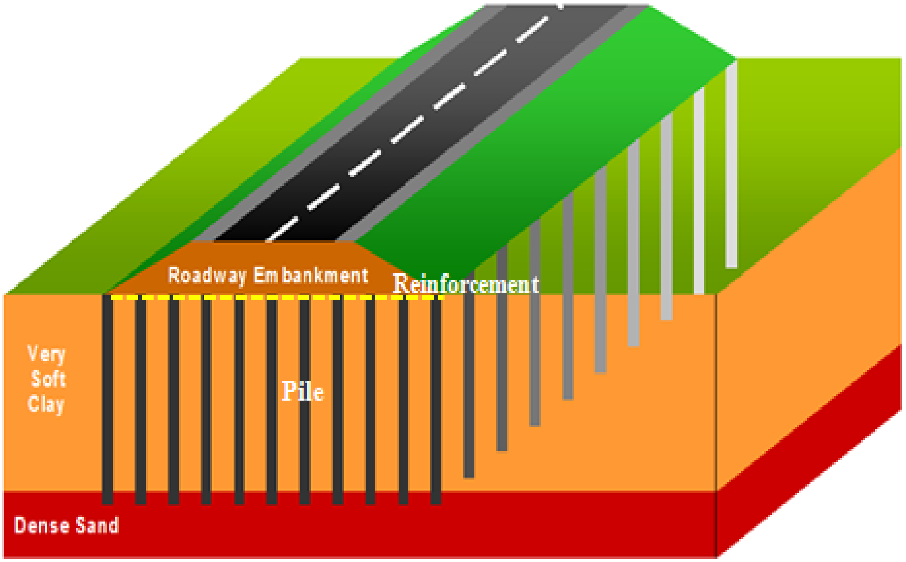
van Eekelen and Han (2020)

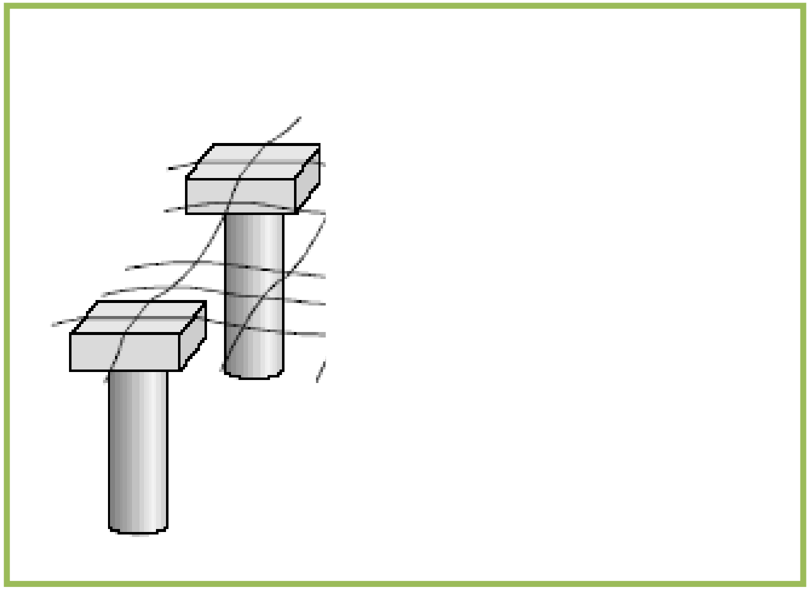
Coconut logs used as piles below bridge abutment in soft clays, Kerala PWD, India

Comparison between different treatment methods

Item of comparison	Traditional pre-consolidation	Vacuum assisted pre-consolidation	Granular columns	Geosynthetic pile platforms
Construction time	High	Medium	High	Short
Long term settlements	High	Medium	Medium	Low
Maintenance	High	Medium to high	Medium	Negligible
Construction cost	Low	High	Medium to high	High
Damage to nearby structures	Likely	Low	Limited	Negligible

Principle of Geosynthetic Reinforced pile platforms





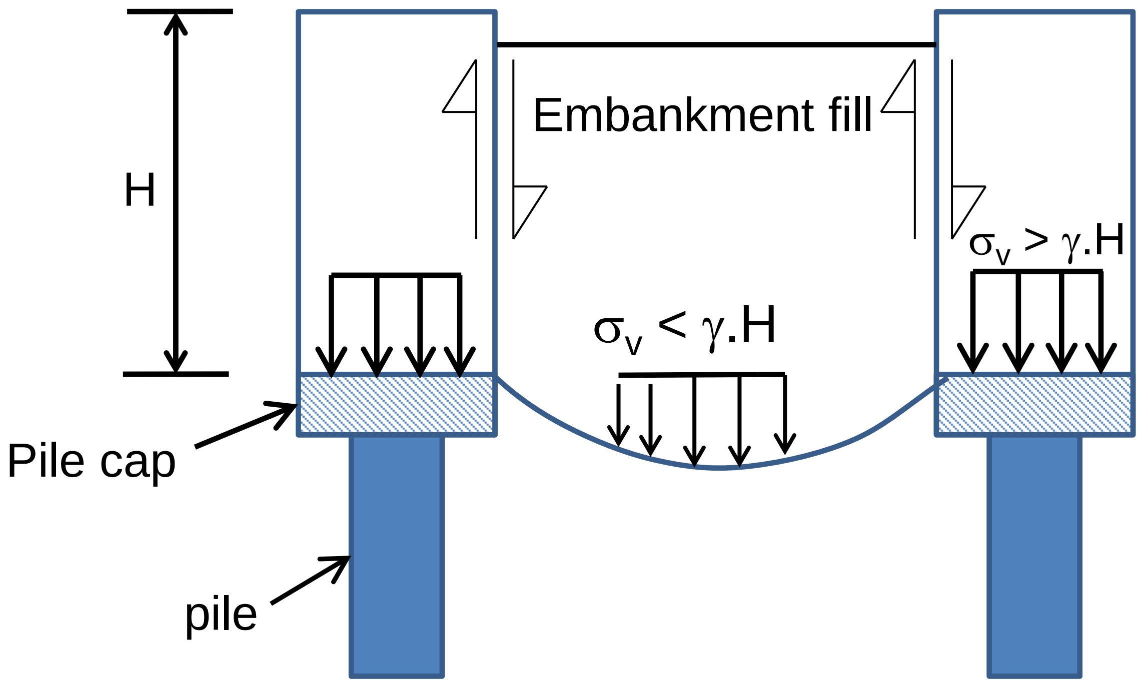
Schematic of geosynthetic pile platform
Smith (2008)

Tension membrane effect of reinforcement
Russel and Pierpont (1997)

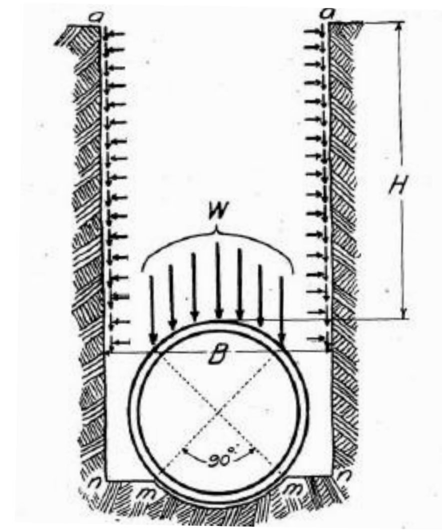
Outline of the Lecture

- Introduction
- Geosynthetic Pile Platforms
- **Arching Theory**
- Centrifuge modelling of embankments
- Numerical analyses
- Case Histories
- Design Methods
- Summary

Mechanism of load re-distribution due to arching

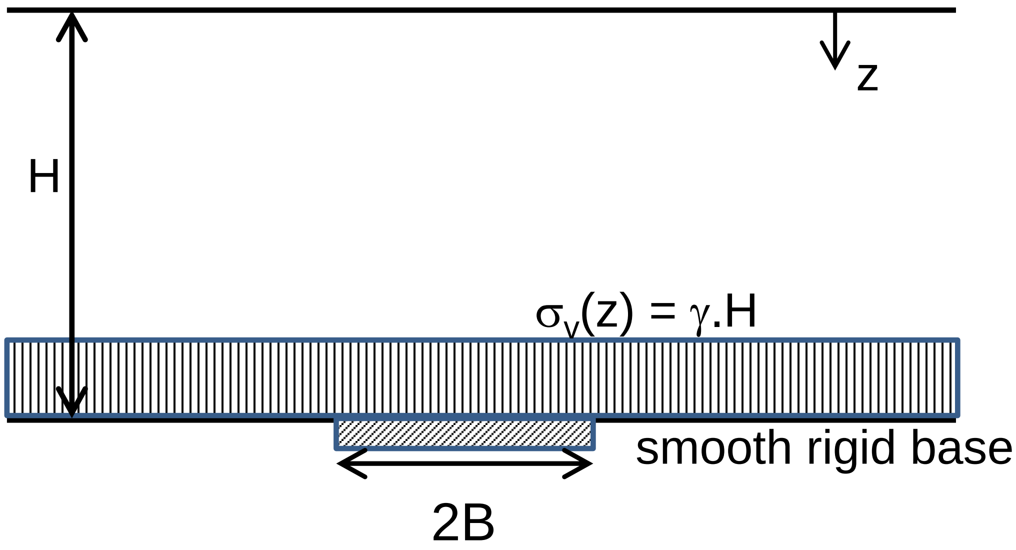


Lower pressures on soil and higher pressures on rigid supports



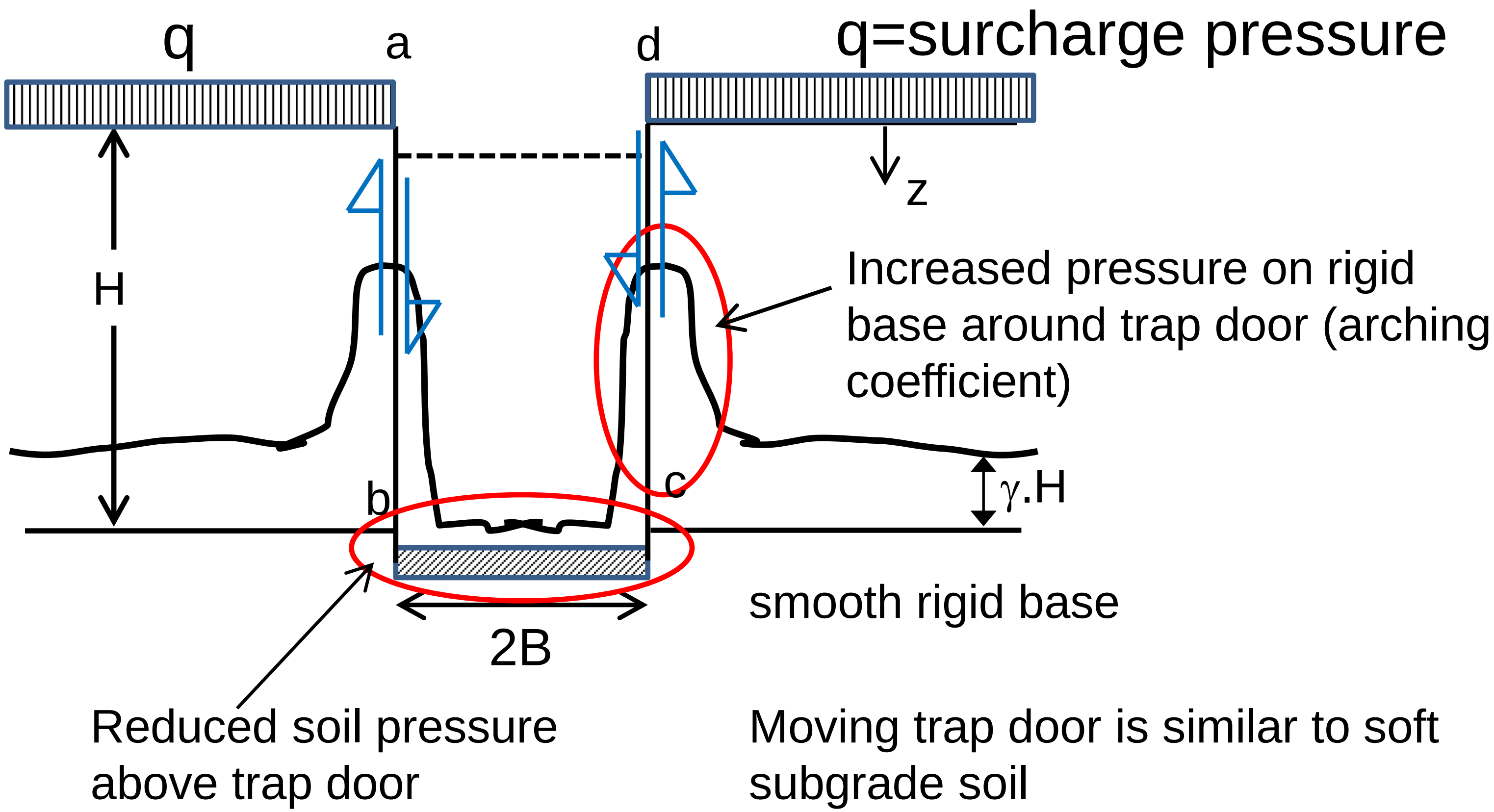
Reduced pressure on flexible pipe in a narrow trench

Trap door analogy for the soil arching



Uniform pressure at base before movement of trap door

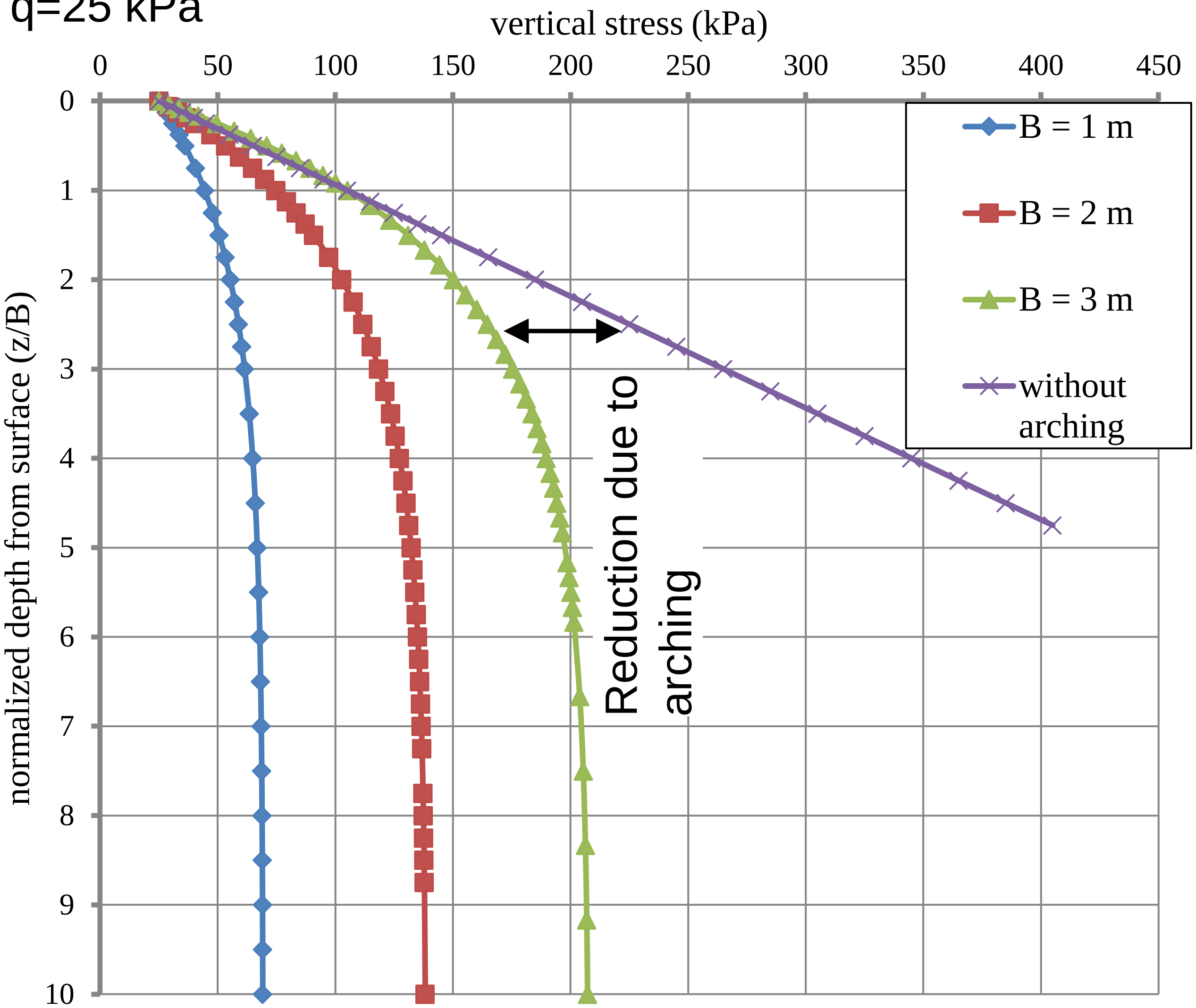
Pressure distribution when trap door moves down



$$\sigma_v(z) = \frac{\gamma . B}{K . tan\phi}\left[1 - e^{-K.tan\phi.(\frac{Z}{B})}\right] + q . e^{-K.tan\phi.(\frac{Z}{B})}$$

Effect of different parameters on soil pressures

q=25 kPa



Pressure reduction due to arching:

- Higher for smaller width of trap door (or lower spacing between the piles)
- More evident at deeper depths (or larger embankment heights)

Effect of arching

- Arching reduces vertical pressures on soft subgrades
- Pressure reduction is more for closer spacing between piles
- Pressure reduction is more for larger embankment heights
- Larger pressure reductions can be achieved by geosynthetic reinforcement
- Tension membrane effect of geosynthetic layer helps in further reducing pressure on subgrade soil & increase pressure on pile caps

Performance measures commonly used to evaluate the GRPES

Efficacy- Portion of the embankment weight carried by the piles Hewlett & Randolph (1988)

$$E = \frac{p}{s^2 \gamma H}$$

Soil Arching Ratio or Stress Reduction Ratio (SRR) – degree of arching (McNulty1965).

$$\rho = \frac{p_b}{\gamma H + q_0}$$

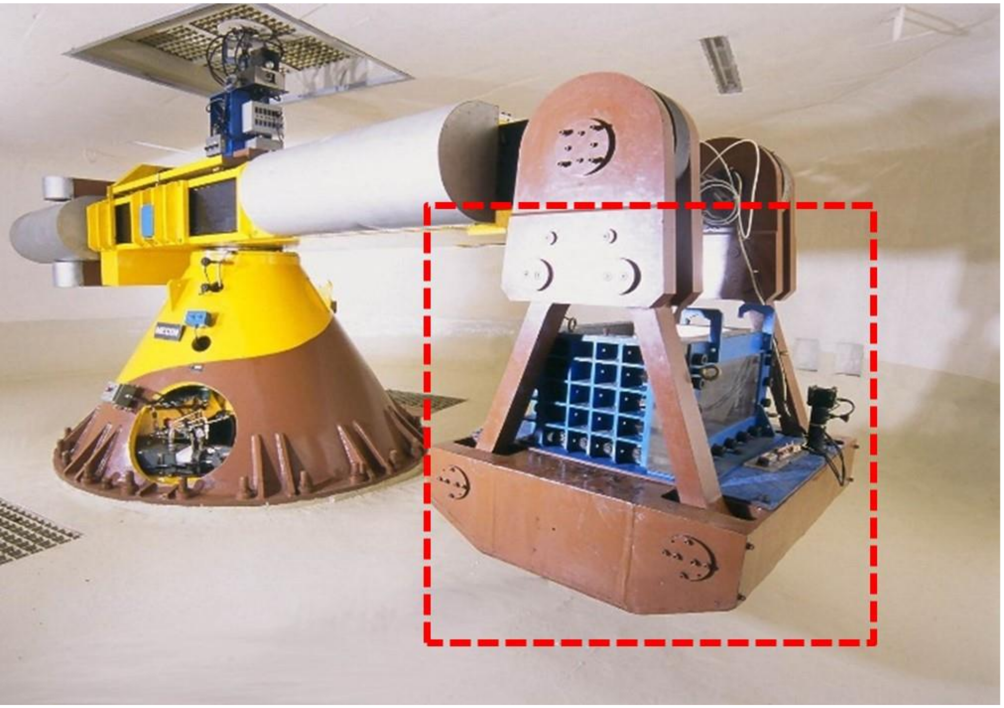
Stress Concentration Ratio (SCR)-measure of the degree of the load transfer to the piles (Han and Gabr 2002).

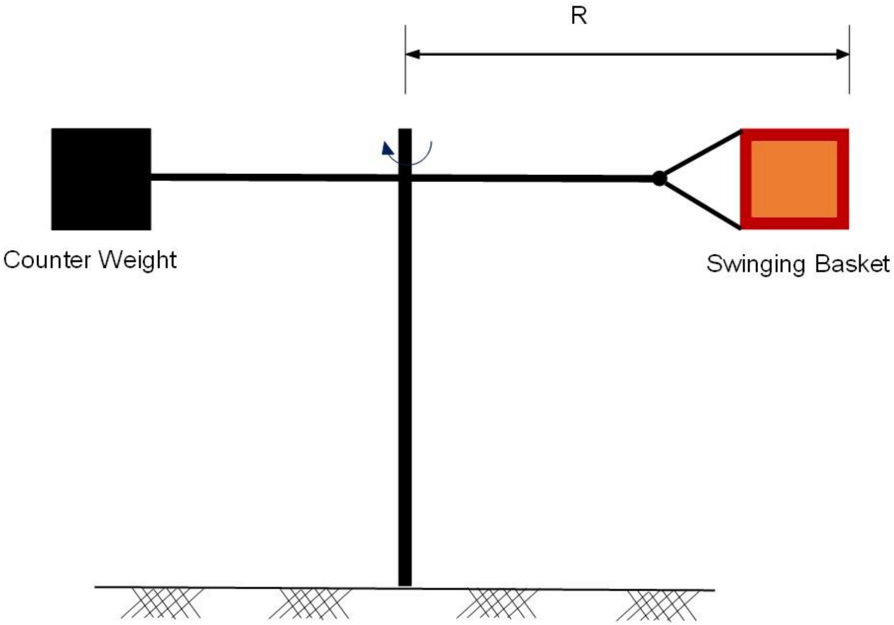
$$n = \frac{\sigma_c}{\sigma_s}$$

Outline of the Lecture

- Introduction
- Geosynthetic Pile Platforms
- Arching Theory
- **Centrifuge modelling of embankments**
- Numerical analyses
- Case Histories
- Design Methods
- Summary

Centrifuge model tests of embankments





Scaling considerations

Parameters	Units	Prototype	Model
Soil Parameters			
Unit weight (γ)	kN/m^3	1	N
Cohesion (c)	kN/m^2	1	1
Angle of internal friction (φ)	Degrees	1	1
Rate of construction	Days	1	$1/N^2$
Stress (σ)	kN/m^2	1	1
Strain (ε)	%	1	1
Settlement (δ)	m	1	1/N
Geogrid Parameters			
Length (a_l, a_t, b_l, b_t, t)*	m	1	1/N
Cross-sectional area of rib, A	m^2	1	$1/N^2$
Percentage Open Area, f	%	1	1
Tensile load, T	kN/m	1	1/N
Geogrid strain, ε	%	1	1
Secant Stiffness, Jg	kN/m	1	1/N
Pull-out force, F	kN	1	$1/N^2$
Pull-out stress, τb	kN/m^2	1	1
Soil-geogrid friction angle	Degrees	1	1
Soil-geogrid interface stiffness, Jsg	kN/m	1	1/N

Parameters	Units	Prototype	Model
Pile Parameters			
Pile diameter, a	m	1	1/N
Pile length, l	m	1	1/N
Axial Rigidity, (AE)	kN	1	$1/N^2$

Geometric dimensions of different piles

END BEARING PILE

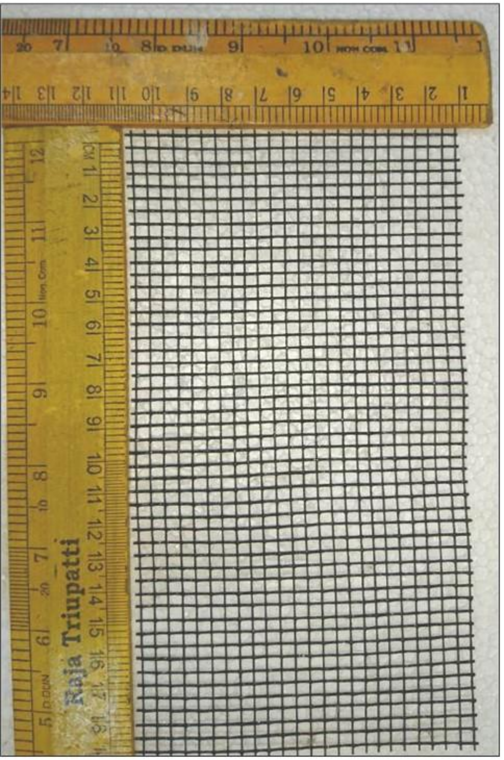
FLOATING PILE

Properties of Pile

Parameter	Unit	Model	Prototype
External diameter of pile	mm	12.7	600
Wall thickness	mm	2	
Pile cap dimension	mm	40×40×8	1600×1600×320
Axial Stiffness	MN	4.706	7530

/

GEOGRID



Properties of geogrid

Parameter	Geogrid property
Material type and composition	PET+PVC
Aperture size in longitudinal and transverse directions (mm)	3.5 x 3.5
Thickness of longitudinal rib, $(t_r)l$ (mm)	0.7
Thickness of transverse rib, $(t_r)t$ (mm)	0.4
a,b Tensile load (kN/m) at:	
2% strain	2.2 (88)
5% strain	3.2 (128)
10% strain	5.6 (224)
a,b Secant modulus, J_g (kN/m) upto 5% strain	64 (2560)
a, b Ultimate tensile load, T_g (kN/m)	11 (440)
a, b Ultimate tensile strain ε_g (%)	15
Percentage open area, f (%)	75.8

a - Wide width tensile tests according to ASTM D4595-2010

b - In longitudinal direction (i.e. laid direction in model tests), PET- Polyethylene, PVC-Polyvinyl chloride;

Tests	Geogrid	Pile length (mm)	Pile Spacing (mm)	Depth of soft soil (mm)	Embankment height (mm)
UE	Nil	Nil	Nil	150	150
GR	Yes	Nil	Nil	150	150
GP	Yes	164 (L#)	76.2 (6d*)	150	150
GF	Yes	126 (0.75L)	76.2 (6d)	150	150

* d is the diameter of the pile cap
# L is the length of the pile.

Geometry considered	Model dimensions (mm)	Prototype dimensions at 40g (m)
Thickness of hard layer	20	0.8
Thickness of soft soil	150	6
Thickness of working platform	10	0.4
Height of embankment	150	6
Base width of embankment	435	17.4
Crest width of embankment	210	8.4
Side slope	1.5H:1V	1.5H:1V
Area Ratio	19.3 %	19.3 %

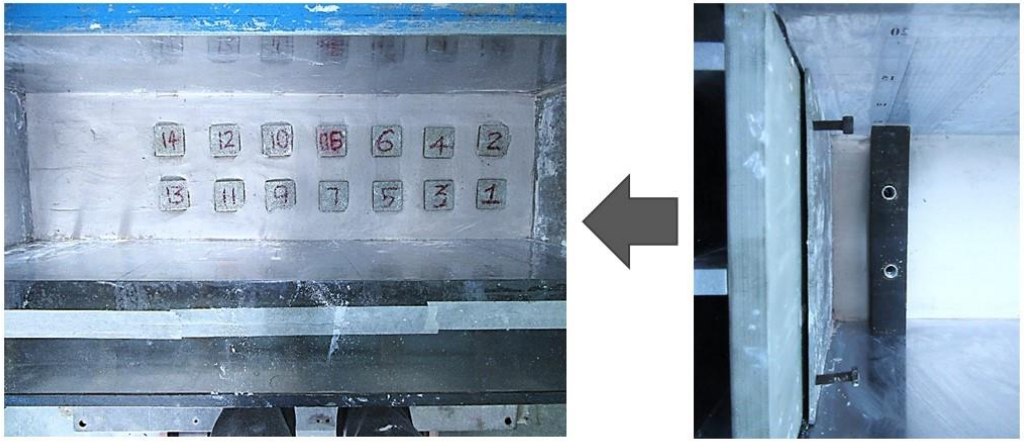
Test Legend		S01	S02	S03	S04
Height of embankment (m)		0.15 (6)	0.15 (6)	0.15 (6)	0.15 (6)
Sand wt. (kg)	C1	9	11	12.1	13.7
	C2	3.3	4	4	4.8
	C3	3.8	3.45	3.45	0
Time taken for embankment construction		80s (1.481days)	80s (1.481days)	160s (2.962days)	192s (3.56days)
Remarks				Closure plate holes reduced to 6 mm	Closure plate holes reduced to 6 mm

Clay bed preparation

Consolidation of clay bed

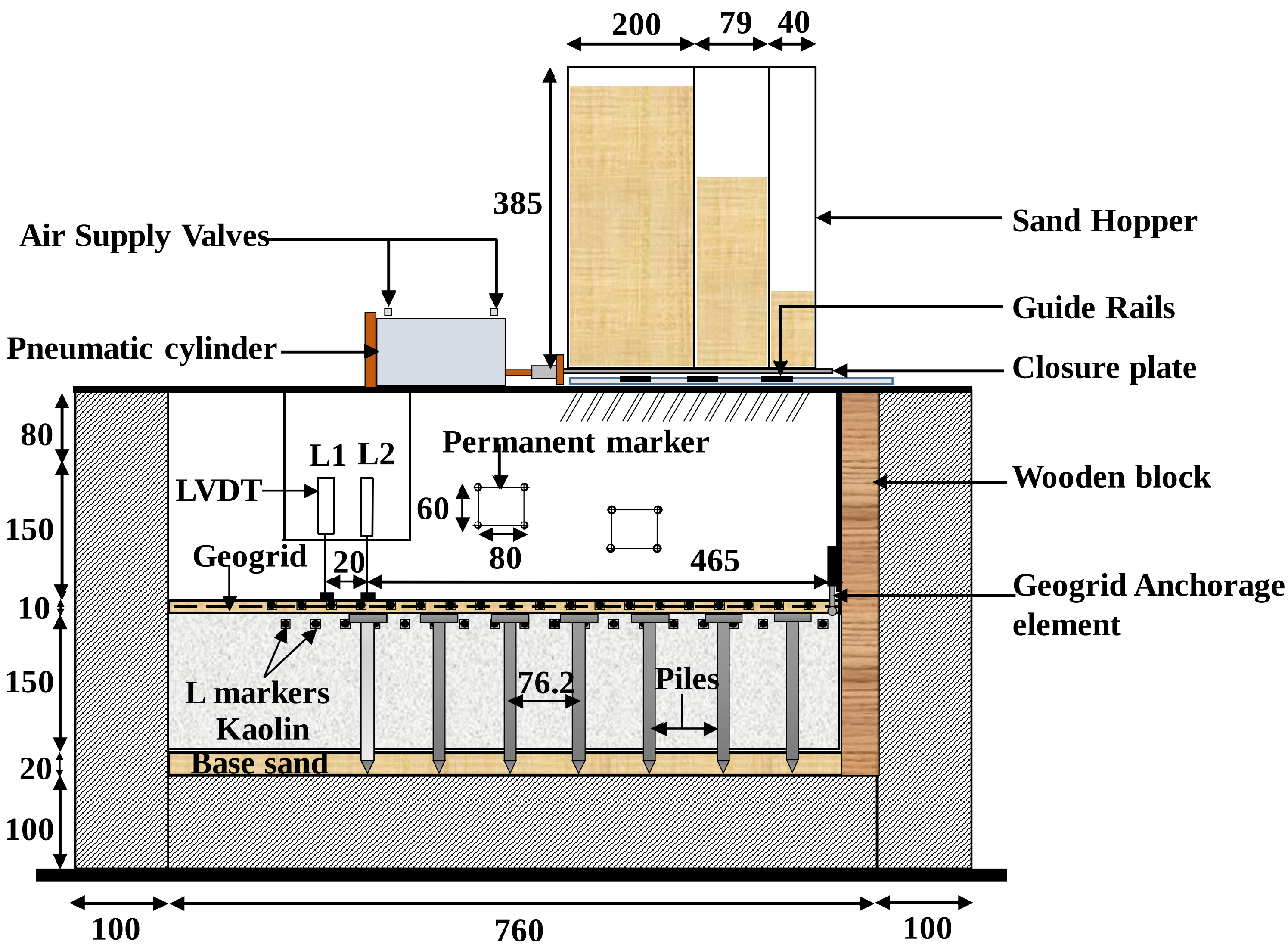
Tilting the container to put seeding for image analysis

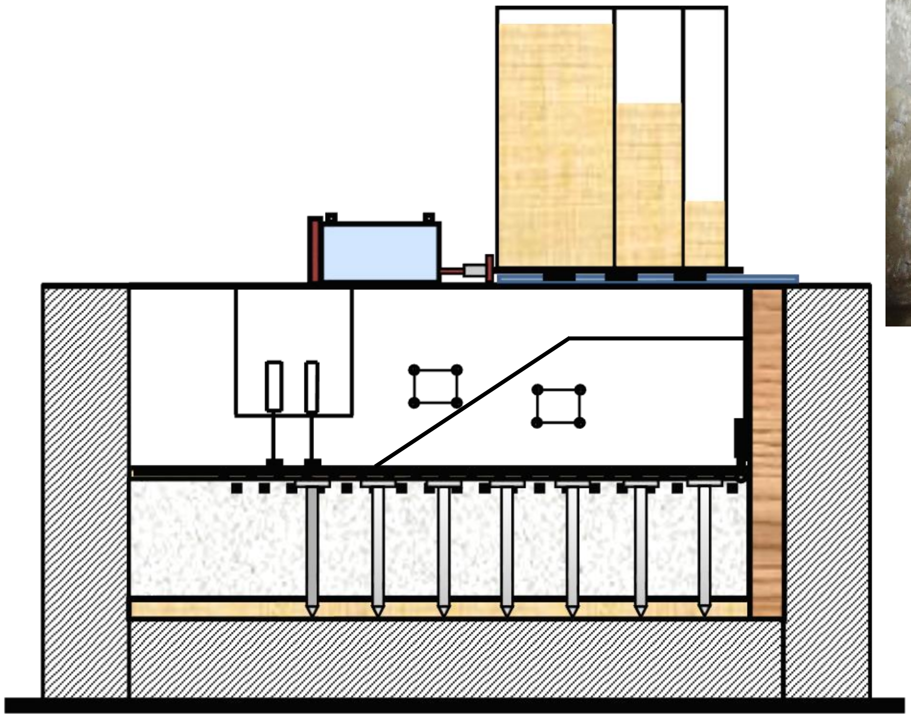
Seeding for PIV analysis

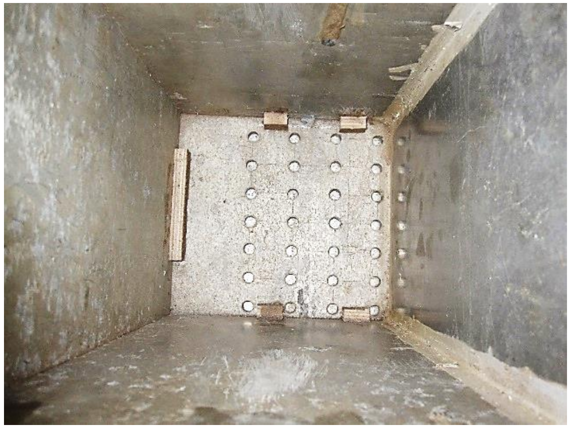


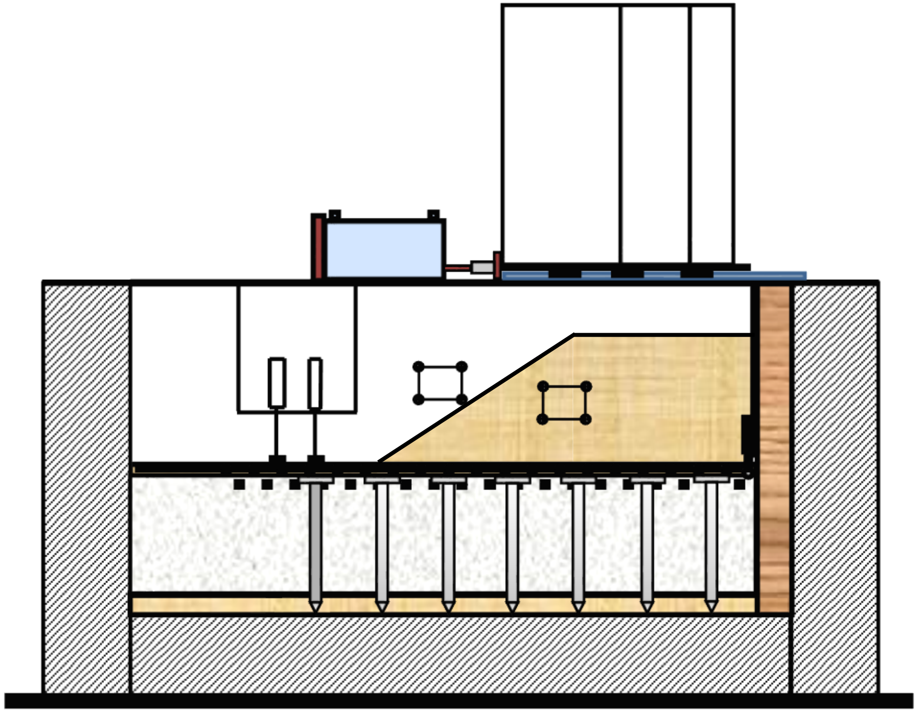
Pile Installation

Schematic of in flight construction of embankment

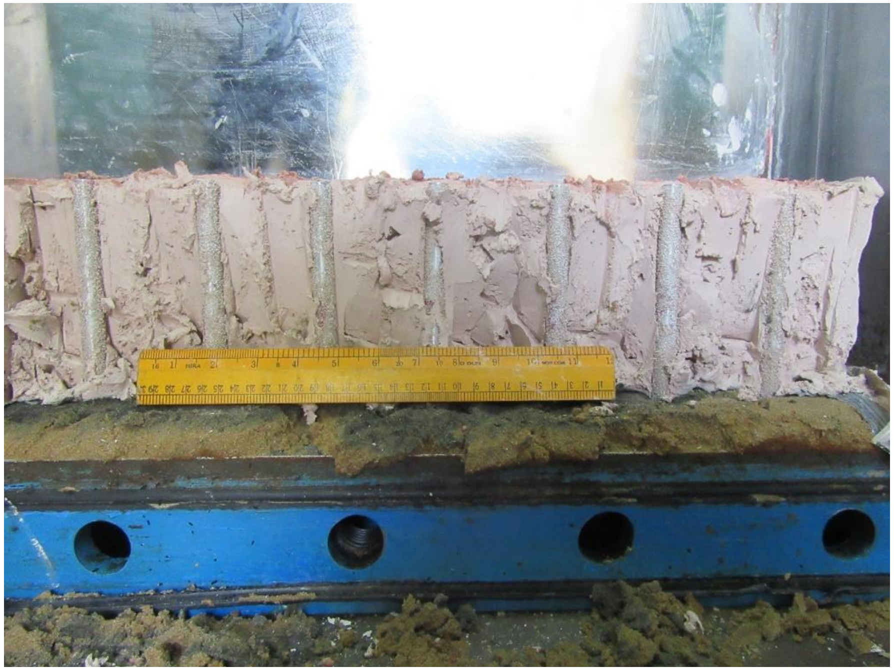








Post Investigation

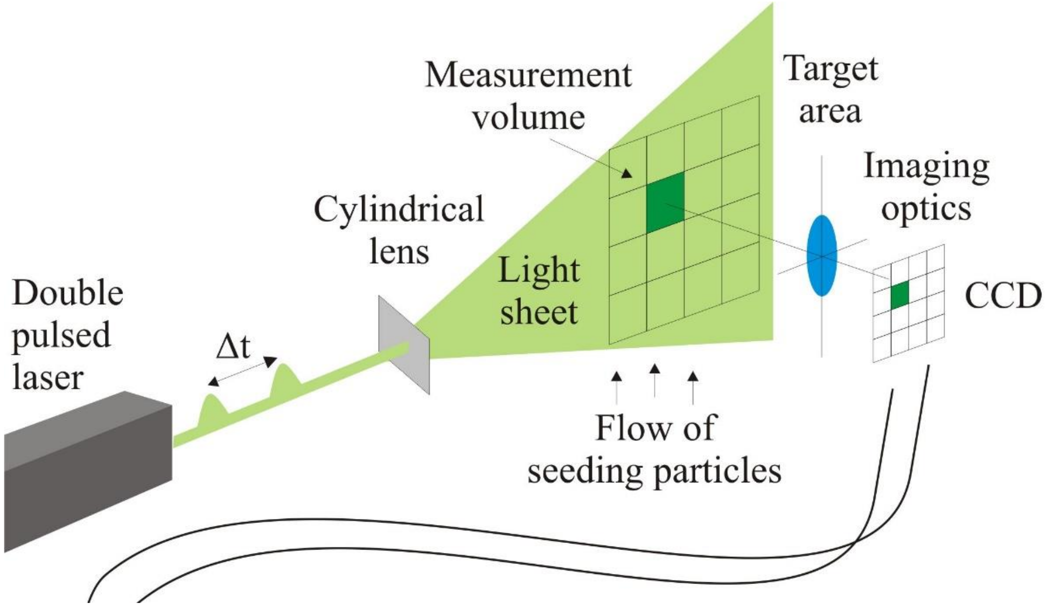


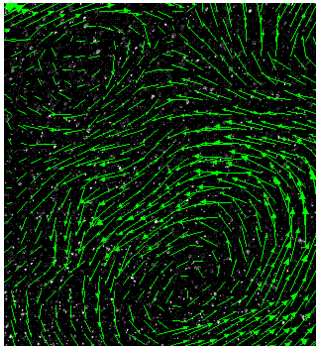
End bearing pile

Floating pile

Image Analysis

- PIV- Particle Image Velocimetry
- Tracking of “seed particles” or “tracers”
- Aerodynamics, fluid mechanics, microbiology, or microfluidic devices among others.



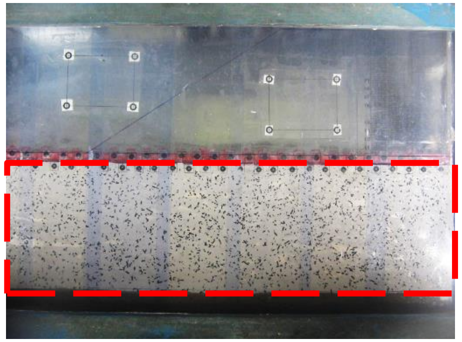


Typical results of displacement vectors from PIV analysis

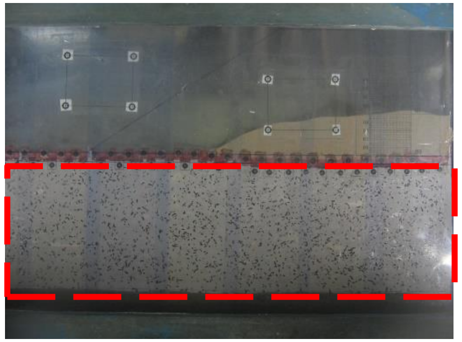
Limitations of the present work

- The effect of creep is ignored in the tests.
- The tensile force in geogrid reinforcement and the loads on the piles could not be measured during the tests.
- Staged embankment construction was not performed.
- Rate of embankment construction cannot be controlled.
- Particle size effects are ignored.
- The effect of pile installation at Ng is not considered.

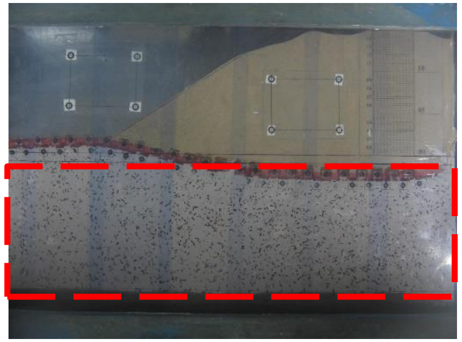
Image Analysis



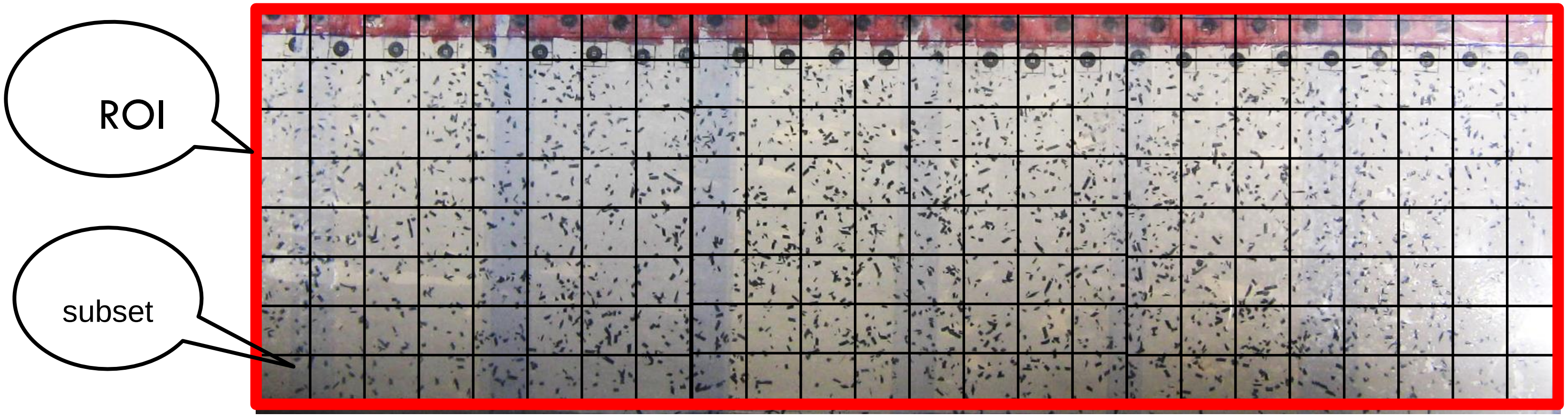
Reference Image



Intermediate image

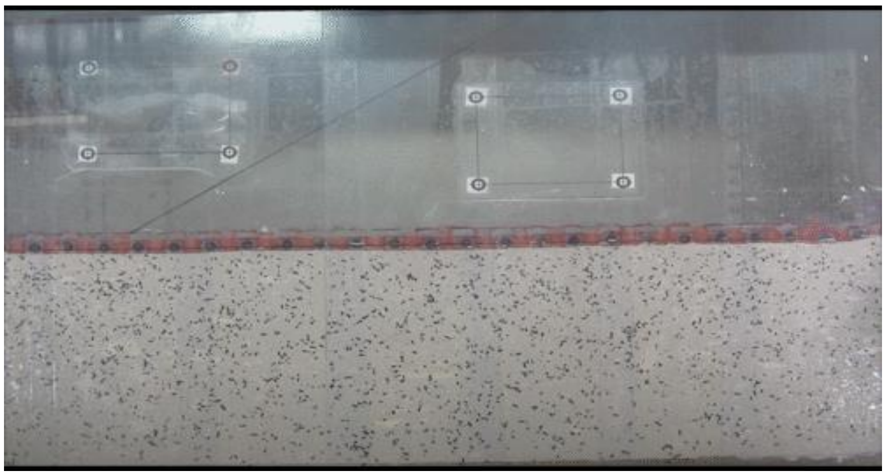


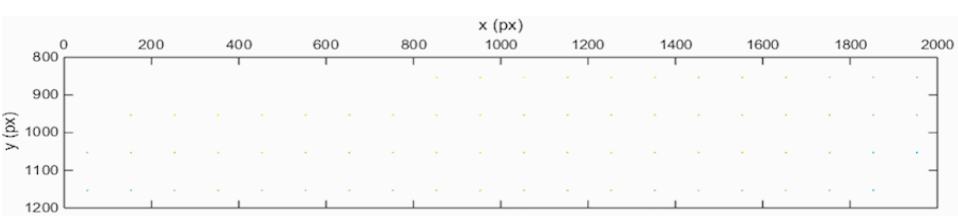
Final Image

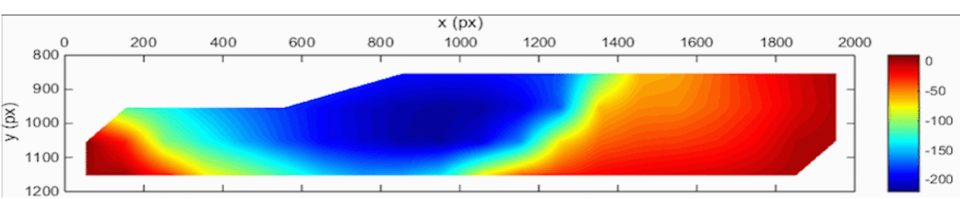


Results

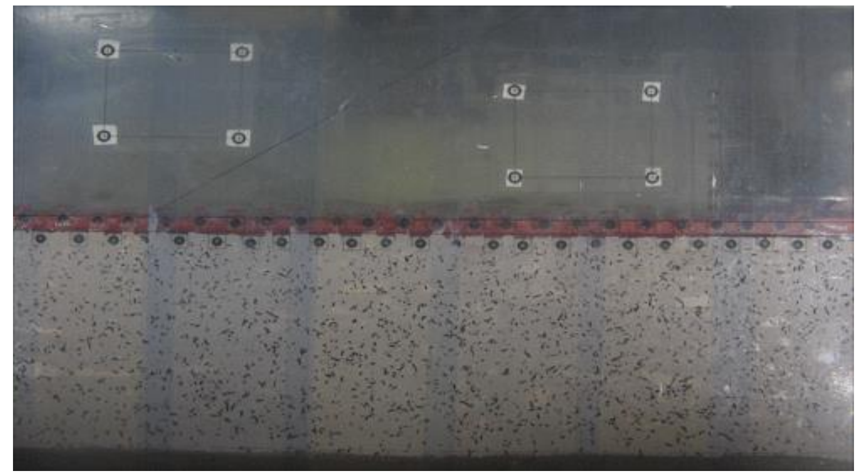
UNREINFORCED

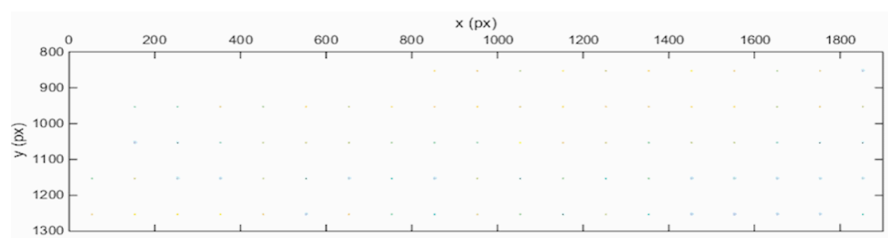


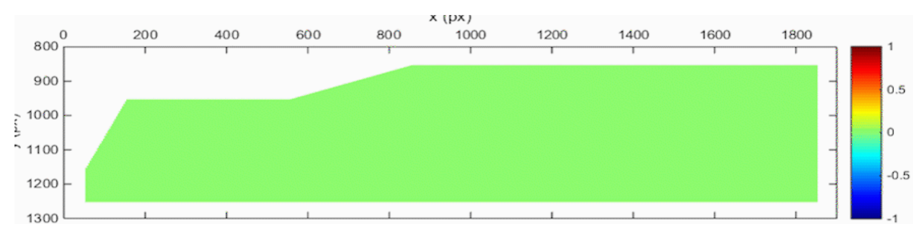




GEOGRID REINFORCED

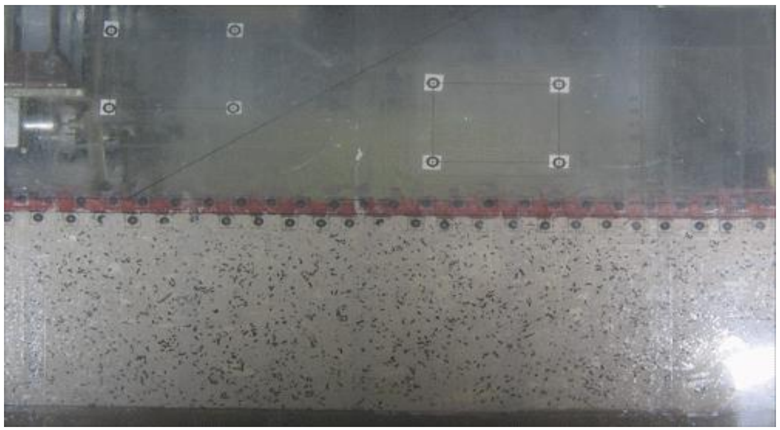


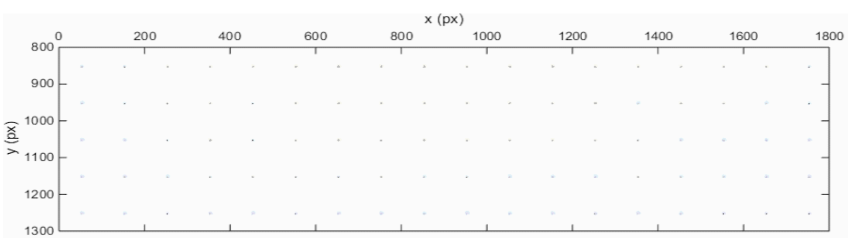


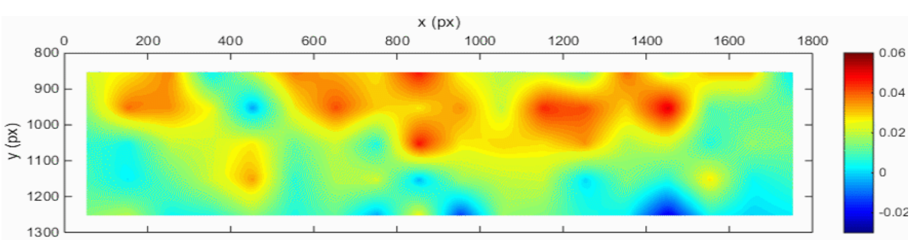


Results

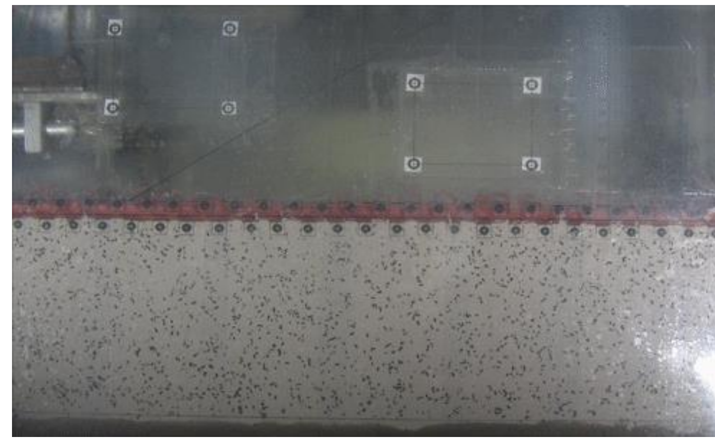
GEOGRID REINFORCED PILE – End Bearing

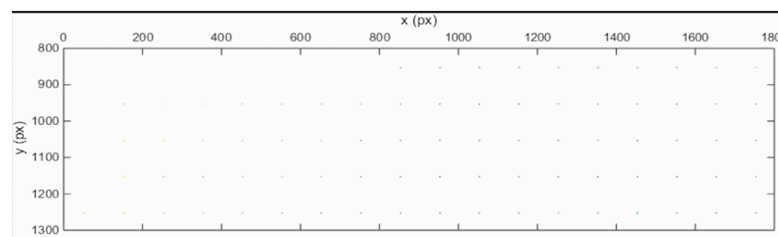


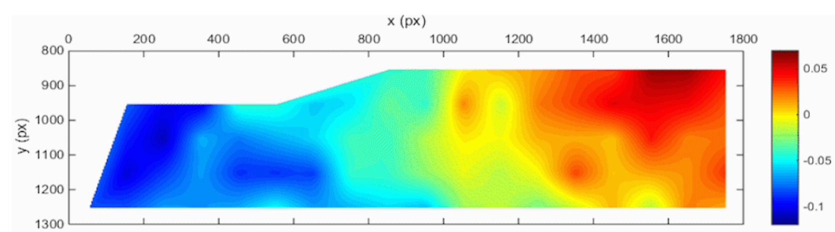




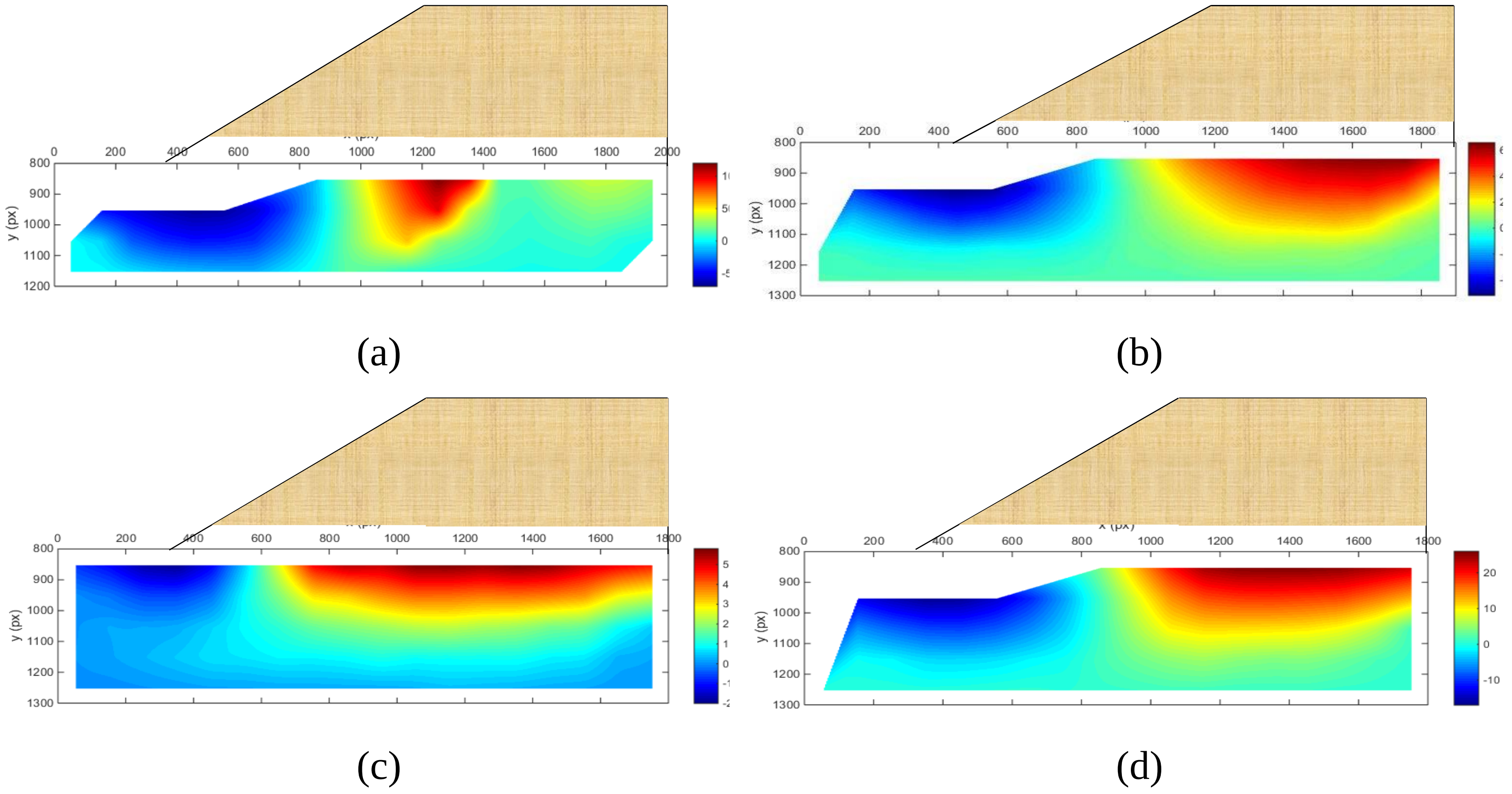
GEOGRID REINFORCED PILE – Friction





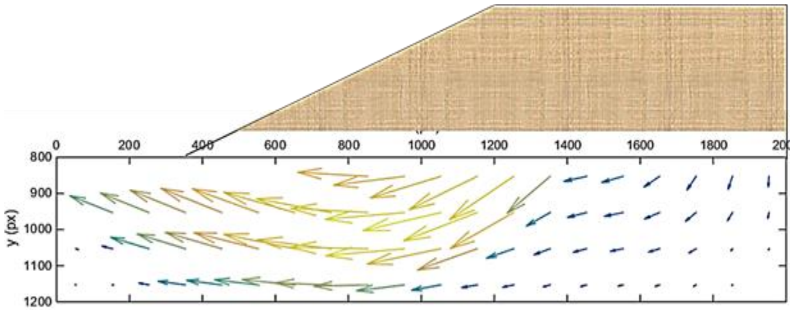


Results

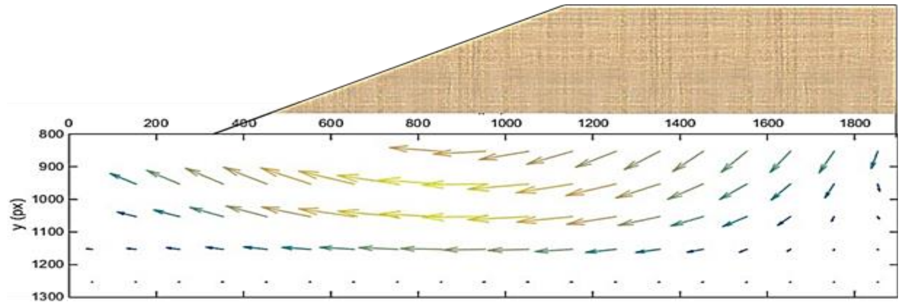


Variation of vertical displacement contours for all tests at the end of embankment construction (a) Unreinforced embankments (UE) (b) Geogrid reinforced embankments (GR) (c) Geogrid reinforced embankments with end bearing piles (GP) (d) Geogrid reinforced embankments with floating piles (GF).

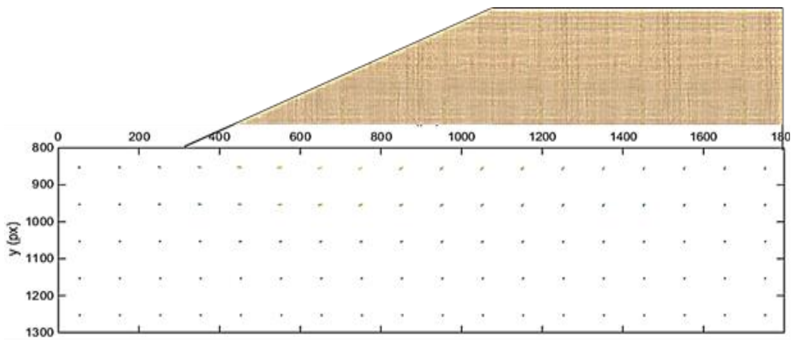
Results



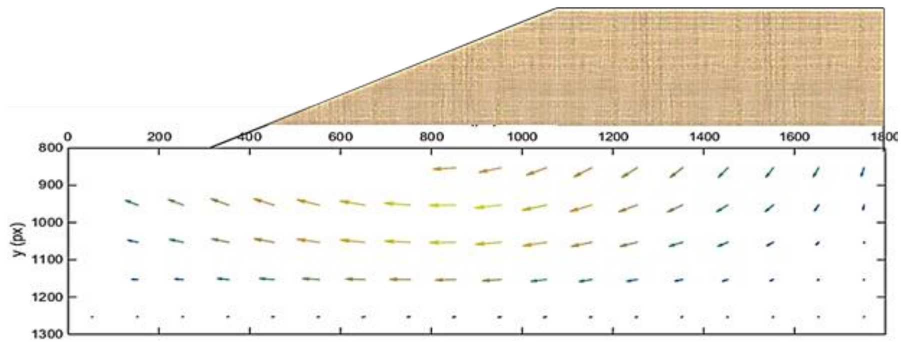
(a)



(b)



(c)



(d)

Comparison of displacement vectors for all models at the end of embankment construction (a) Unreinforced embankments (UE) (b) Geogrid reinforced embankments (GR) (c) Geogrid reinforced embankments with end bearing piles (GP) (d) Geogrid reinforced embankments with floating piles (GF).

Results

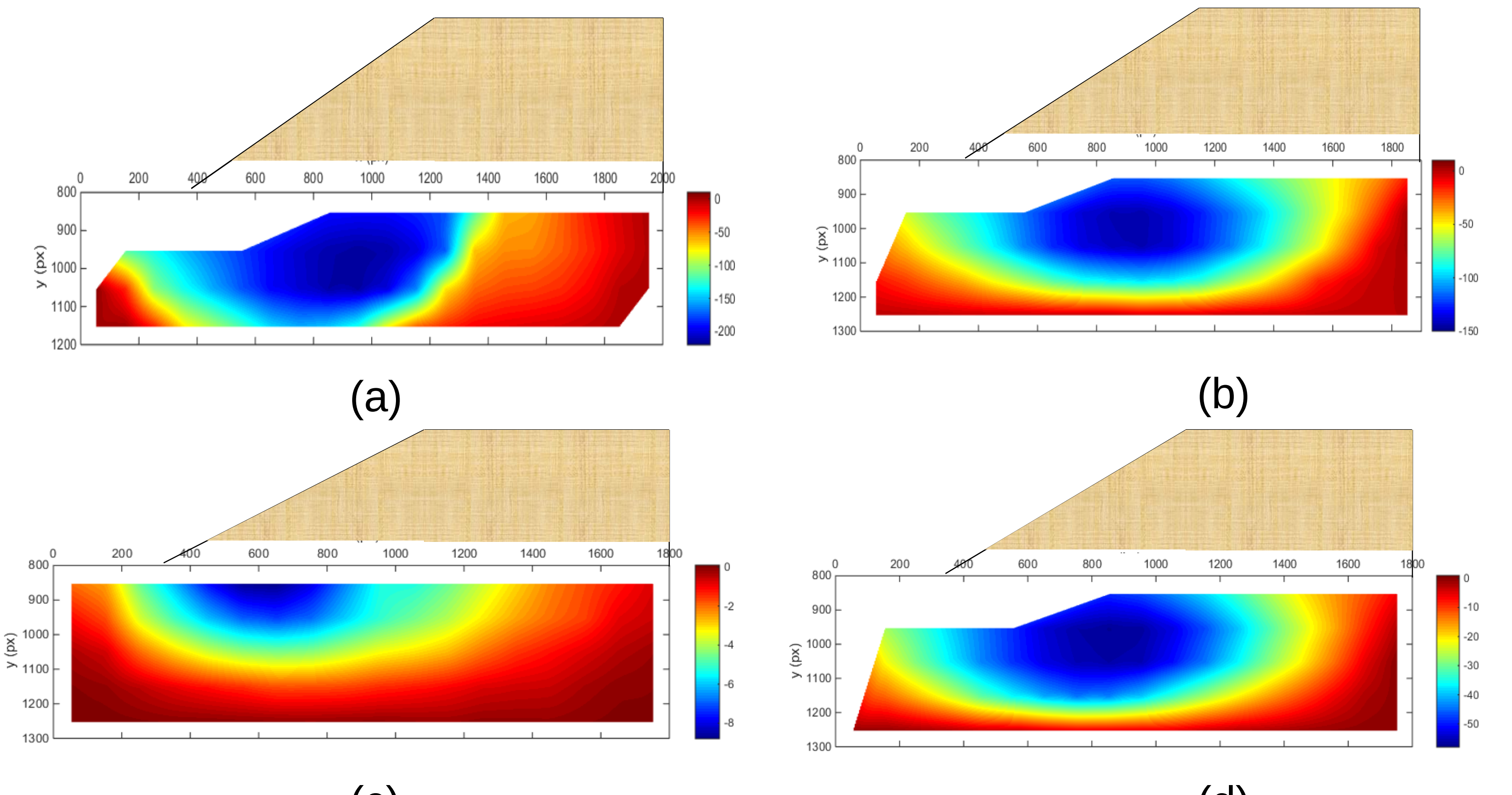


Figure 14: Comparison of horizontal displacement contours for all models at the end of embankment construction (a) Unreinforced embankments (UE) (b) Geogrid reinforced embankments (GR) (c) Geogrid reinforced embankments with end bearing piles (GP) (d) Geogrid reinforced embankments with floating piles (GF).

Observations from centrifuge model tests

- Without ground treatment, embankment undergoes large differential settlements and ground heave
- With geogrid basal reinforcement, lateral flows are reduced to some extent
- GRPES treatment with end bearing piles has restricted the settlements and lateral flows to a large extent
- GRPES with floating piles has resulted in larger settlements but has not shown any failure

Other significant studies

Reference	Embankment	Foundation	Reinforcement	Theory/conclusions
Marston et al. (1913)	Sand	Compacted soil	-	BS8006 recommendations
Kempfert et al.(1999)	Sand	Peat	Scaled geogrid	Simplified design procedures don't reproduce the stresses efficiently.
Hewlett and Randolph (1988)	Sand	Foam rubber chips	-	Resulted in the hemispherical vault arching approach
Zaeske (2001)	Sand	Peat	Scaled geotextile	Multi-shell arching approach (EBGEO 2010)
Britton and Naughton (2008)	Sand	Trap door	-	Validated the critical-height arching approach
Van Eekelen et al. (2012)	Sand	Soaked rubber foam	Scaled Geotextiles and geogrids	Consolidation resulted in increase in the load in pile and geosynthetic

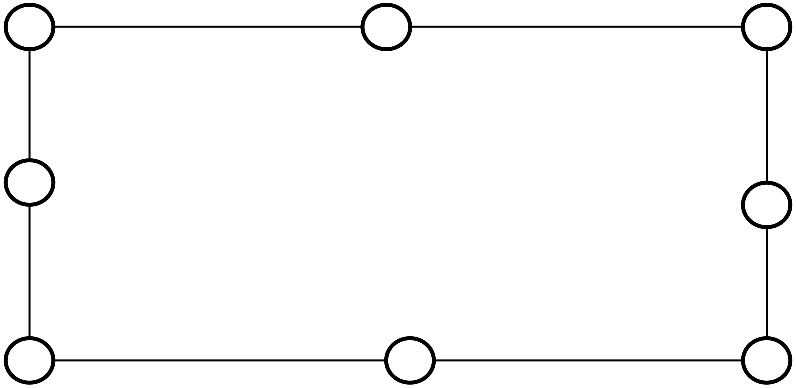
Outline of the Lecture

- Introduction
- Geosynthetic Pile Platforms
- Arching Theory
- Centrifuge modelling of embankments
- **Numerical analyses**
- Case Histories
- Design Methods
- Summary

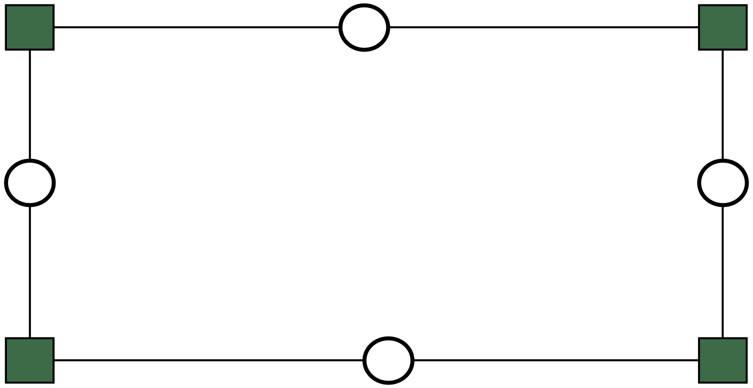
FE analysis considering soil consolidation

- All analyses performed using ABAQUS program
- Stress-pore pressure coupled analyses, Biot (1941)

$$\begin{pmatrix} K & C \\ C & -(E+\delta\times\Delta tH) \end{pmatrix} \begin{Bmatrix} u_{t+\Delta t} \\ \sigma_{t+\Delta t} \end{Bmatrix} = \begin{Bmatrix} F_{t+\Delta t} \\ Q_{t+\Delta t} \end{Bmatrix}$$



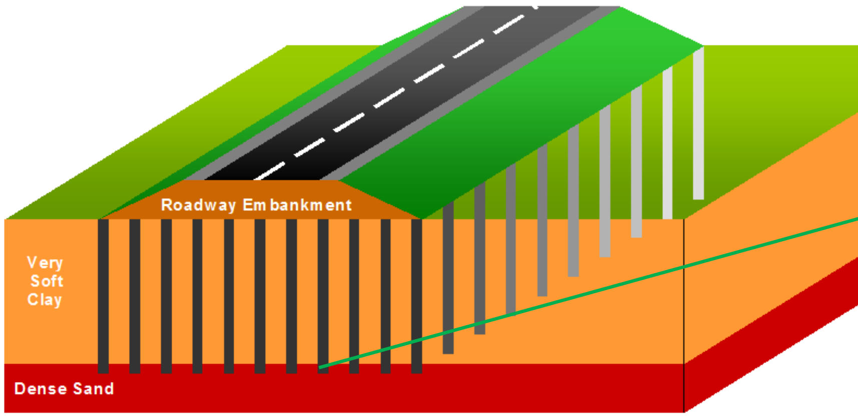
Regular elements (CAX8)

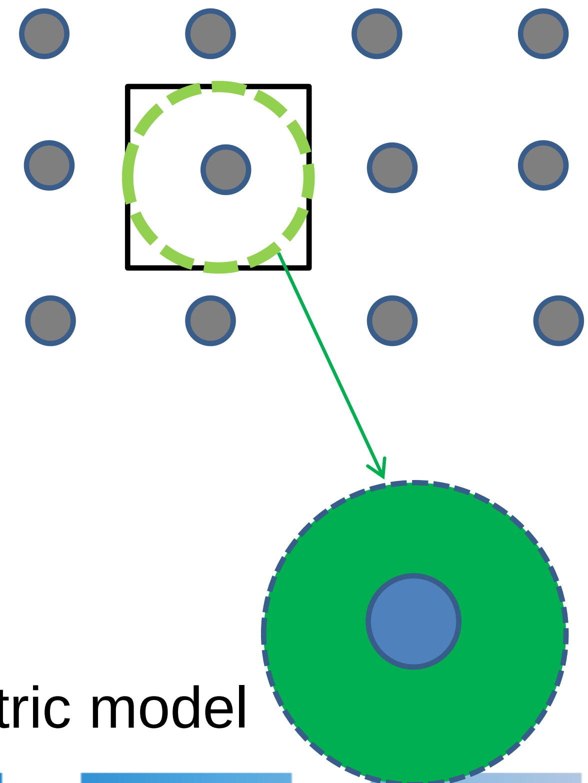


Coupled elements (CAX8P)

Finite Element Models

plan arrangement of piles



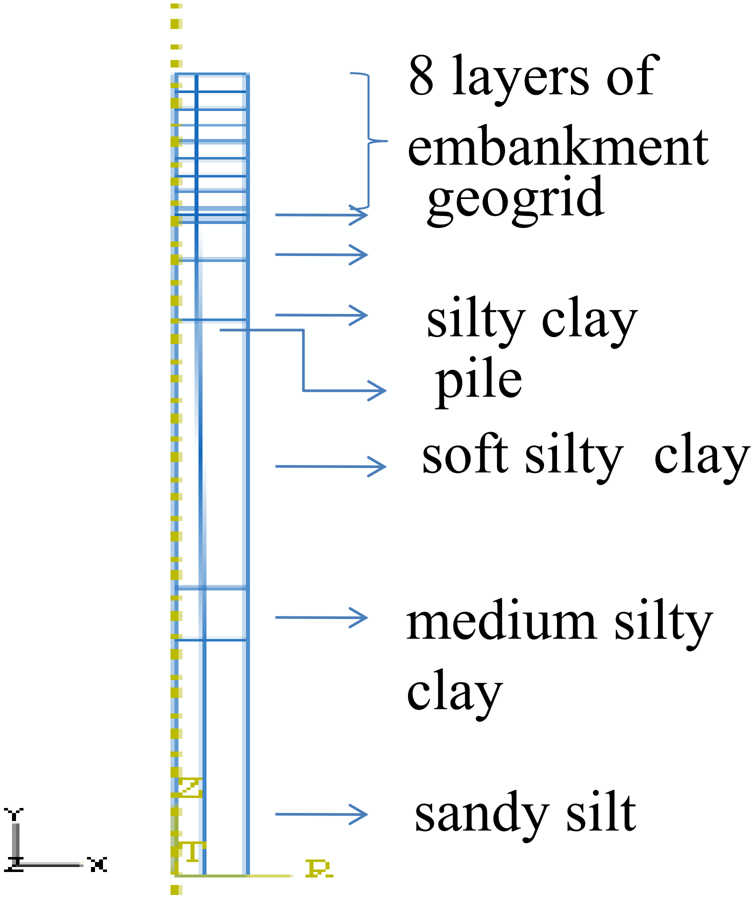


(a) Full Embankment

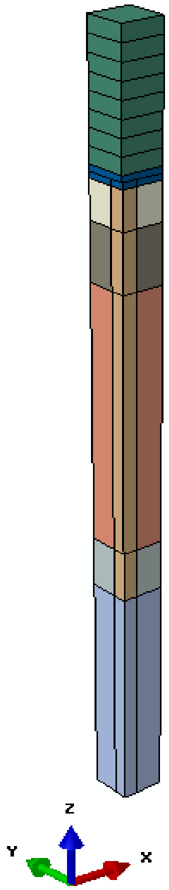
Axisymmetric model

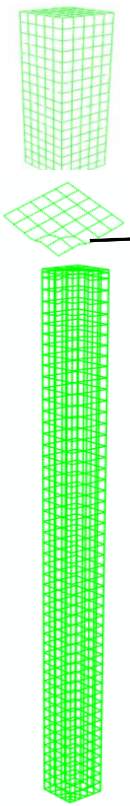
Axi-symmetric model

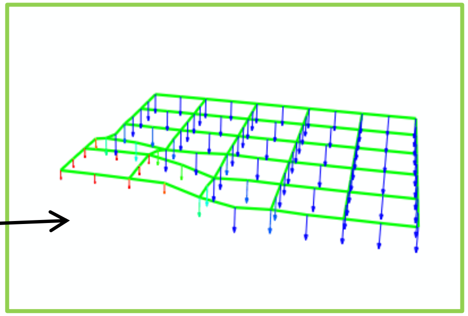
Embankment and Su
fill-CAX8R
Geosynthetic-MAXR
Subsoil-CAX8RP



3-D column model





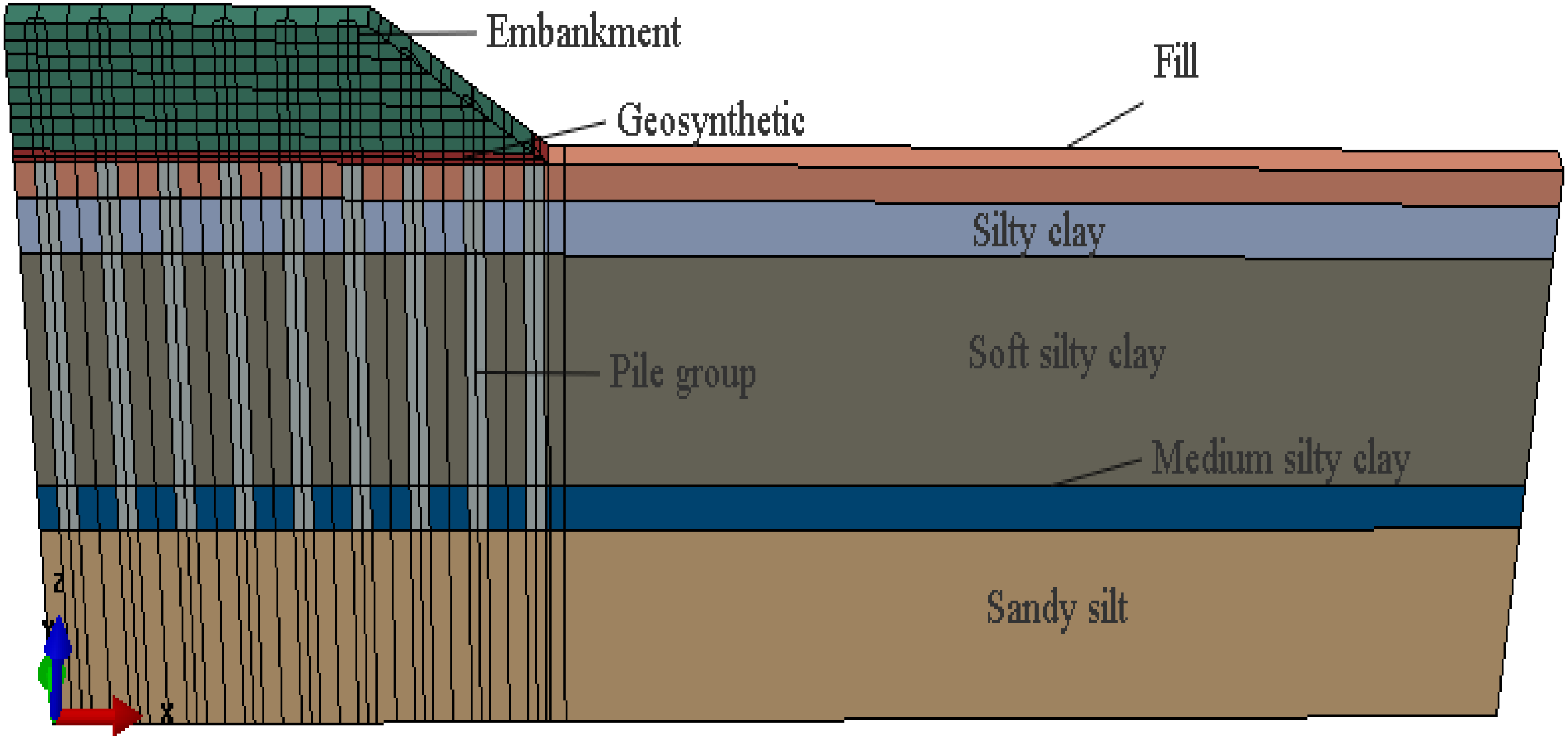


Different elements in the model

- **Embankment and Surface fill-C3D20R**
- **Reinforcement-M3D8R**
- **Subsoil-C3D20RP**

Deflection of reinforcement

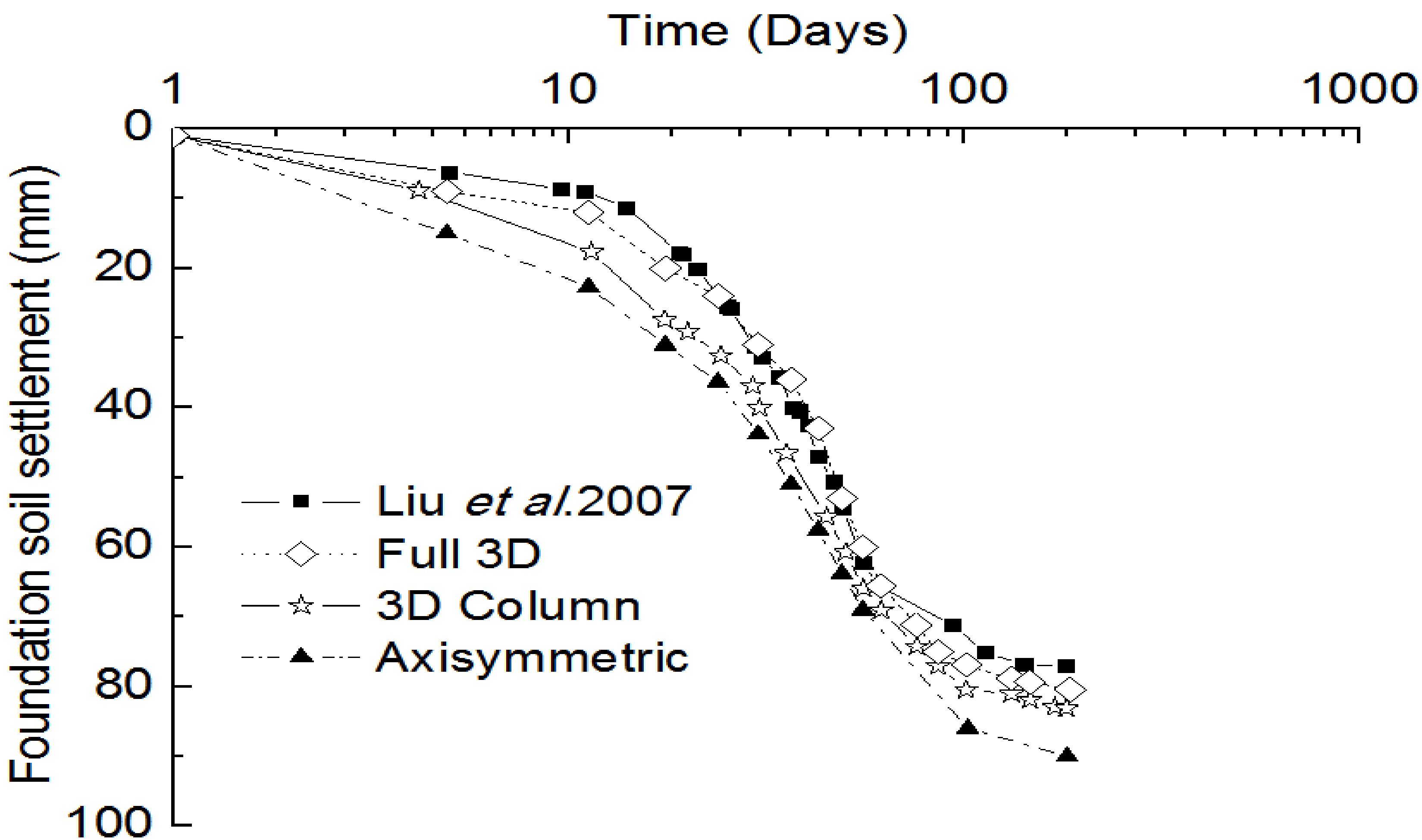
Full-3 dimensional model for the data by Liu et al. (2007)



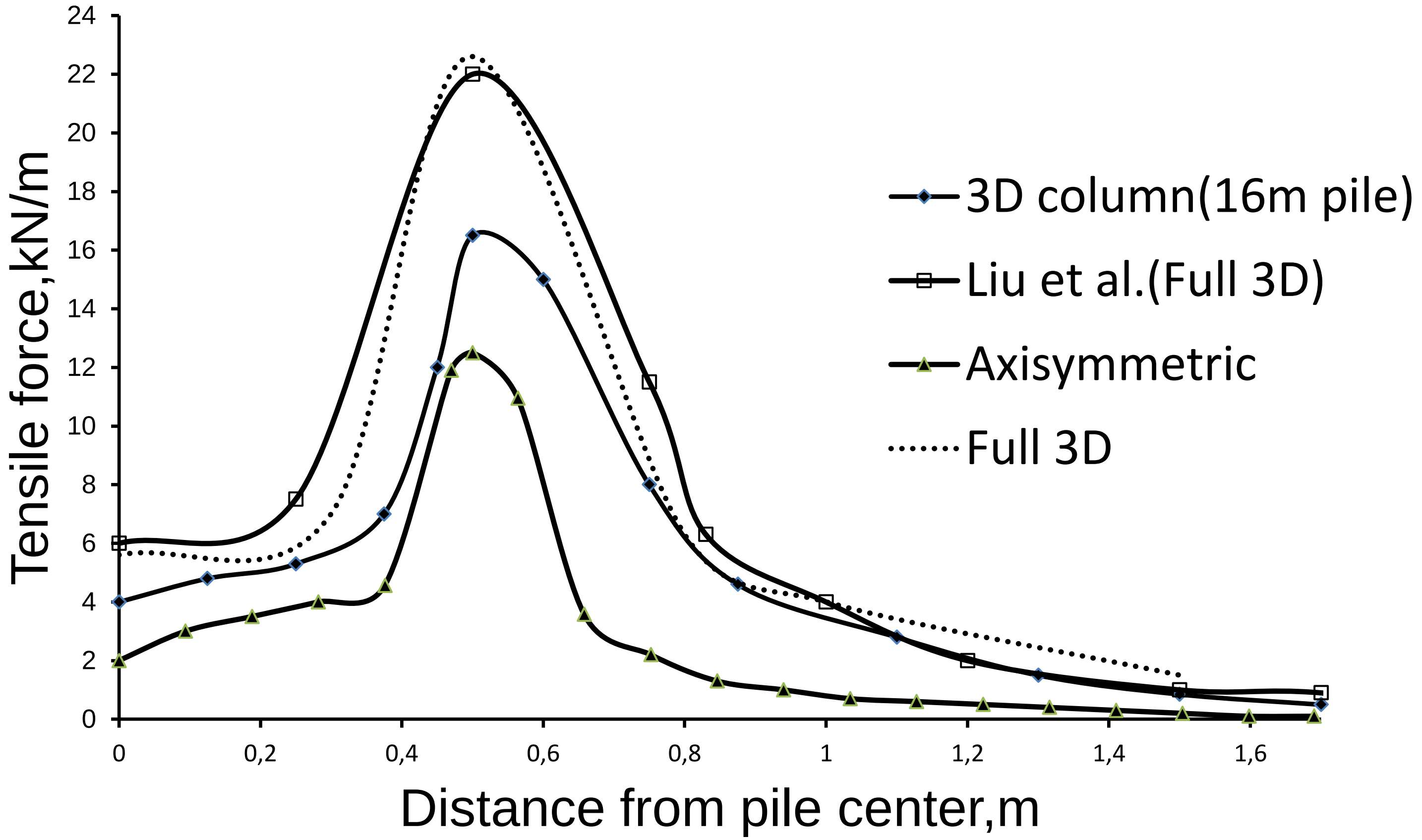
Properties of different soil layers, Liu et al. (2007)

Material	c (kPa)	Φ (deg)	Ψ (deg)	E (MPa)	ν	λ	κ	M	e_1	k ×10^{-4} (m/day)
Embankment	10	30	0	20	0.30					
Gravel	10	40	0	20	0.30					
Coarse-grained fill	15	28	0	7	0.30					
Silty Clay					0.35	0.06	0.012	1.20	0.87	8.64
Soft silty clay					0.40	0.15	0.030	0.95	1.79	4.32
Medium Silty Clay					0.35	0.05	0.010	1.10	0.88	4.32
Sandy silt					0.35	0.03	0.005	1.28	0.97	43.2

Comparison of settlements



Comparison of reinforcement forces at end of construction



Comparison of pile load at end of construction

Height (m)	EBGEO (2010)	BS8006 (Partial & Full)	Hewlett & Randolph	Measured (Liu et al.2007)	Axi-symmetric	3D Column	Full 3D
1.5	73.8	346	66	50	69	72	80
2.5	181	574	127	119	137	141	146
3.0	236	682	175	196	180	187	207
4.0	346	733	217	306	293	302	320
5.6	519	818	308	458	413	425	469

Comparison of foundation soil pressures at end of construction

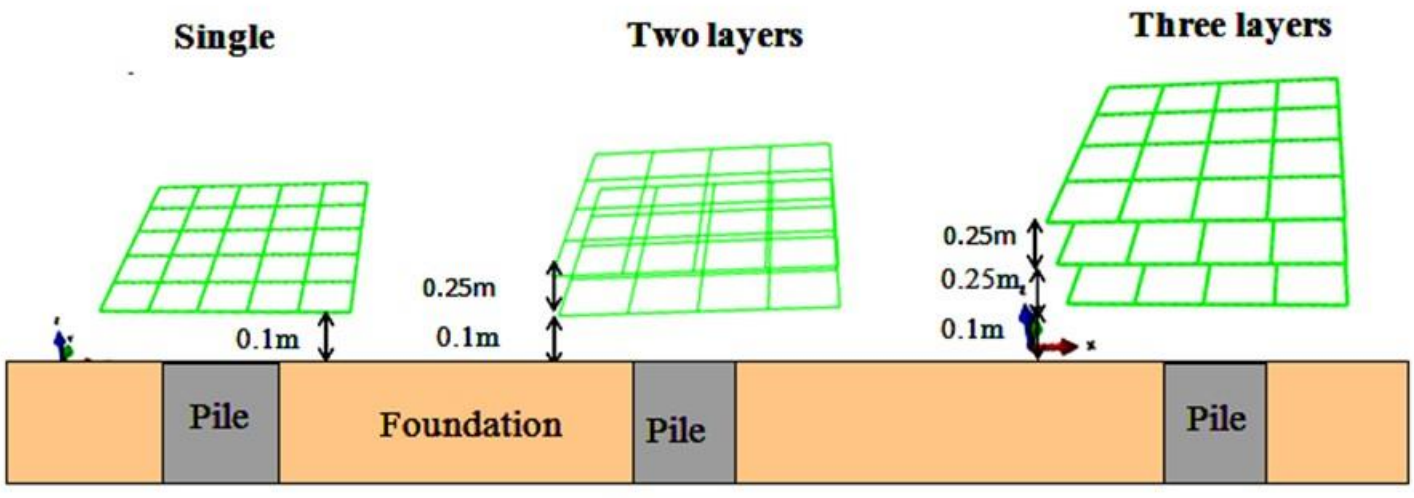
Height (m)	EBGEO (2010)	BS 8006	Measured Liu et al.2007	Axi-symmetric	3D Column	Full 3D
1.5	24.3	0	16.2	13.0	13.5	15.9
2.5	32.4	0	21.6	20.6	21.5	25.3
3.0	37.7	0	26.8	26.2	27.3	32.1
4.0	49.3	0	29.5	34.8	36.2	42.6
5.6	63.8	0	36.7	45.6	47.6	54.2

Comparison of computational efforts

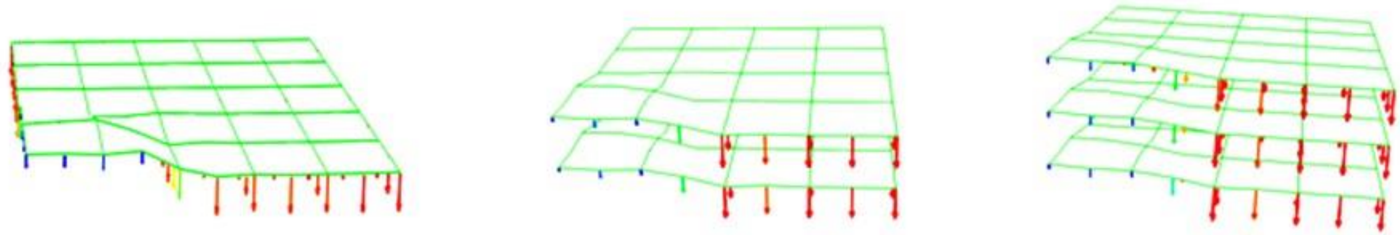
Model	No. of elements in Embankment	No of Geosynthetic Elements	No. of Pile Elements	No. of elements in Foundation	CPU Time
Axi-symmetric	234	9	184	891	15 Min.
3D Column	1860	25	452	9439	45 min.
Full 3D	22,814	274	6074	47,906	96 hours

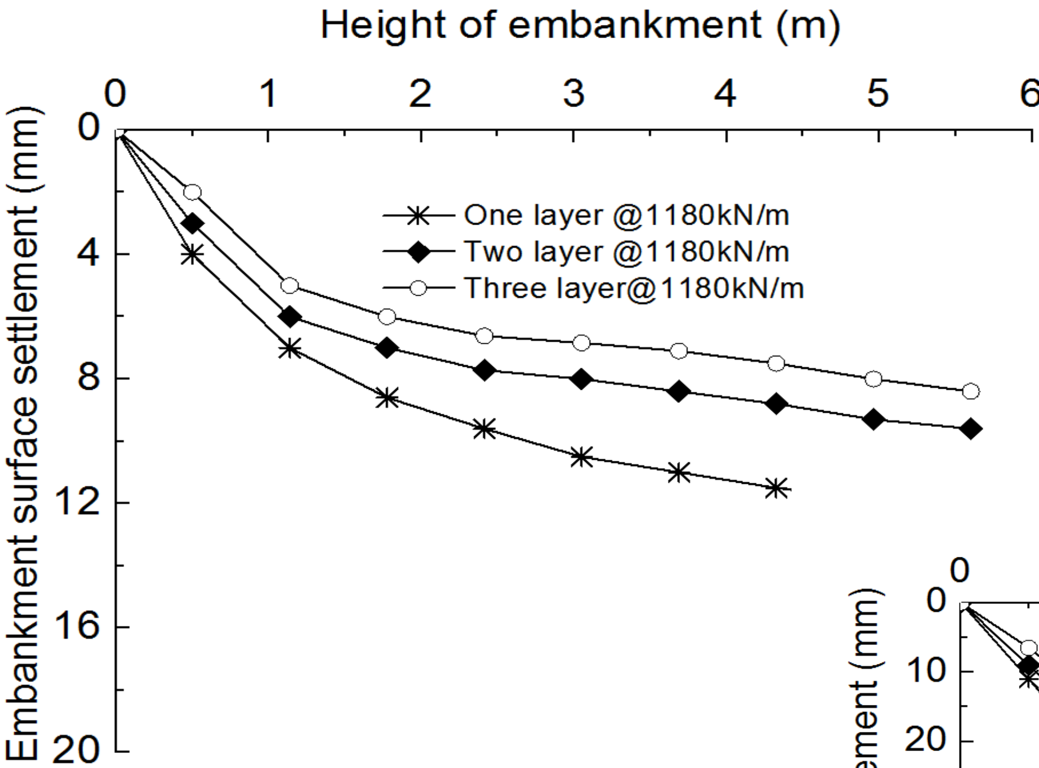
Processor: Intel Xenon E5472, 3GHz, 1600 MHz FSB with 16 GB RAM

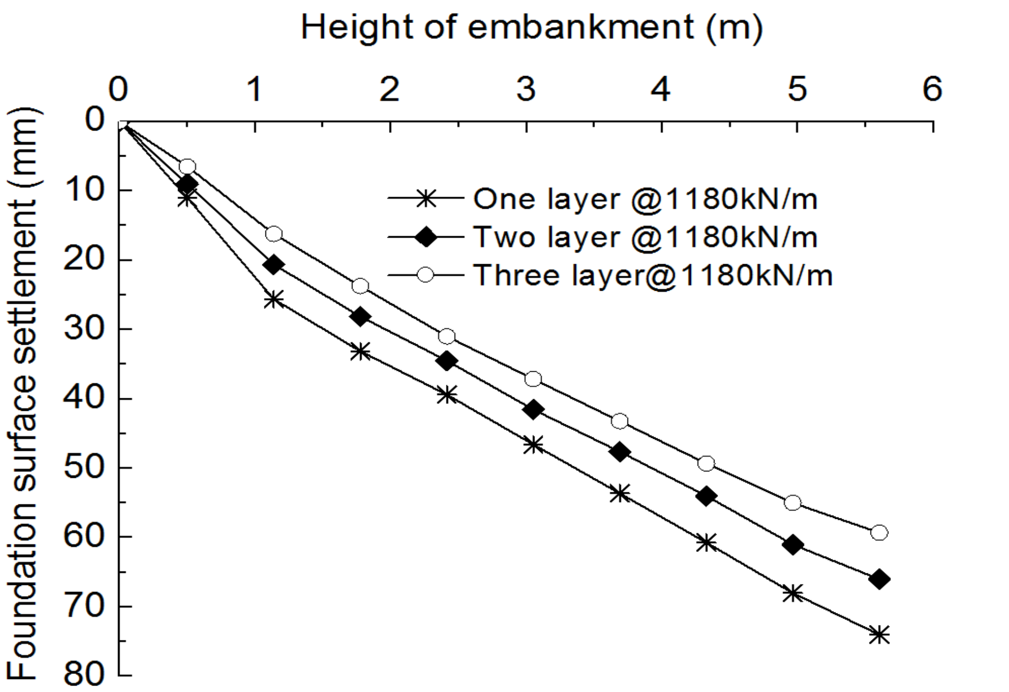
Influence of Number of reinforcement layers



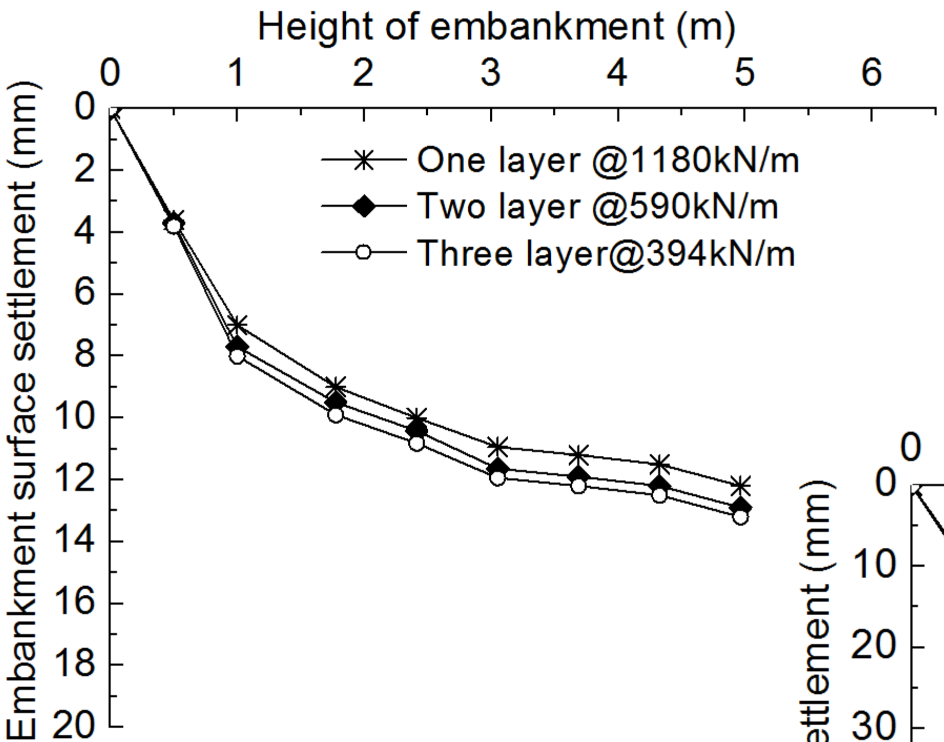
(a)

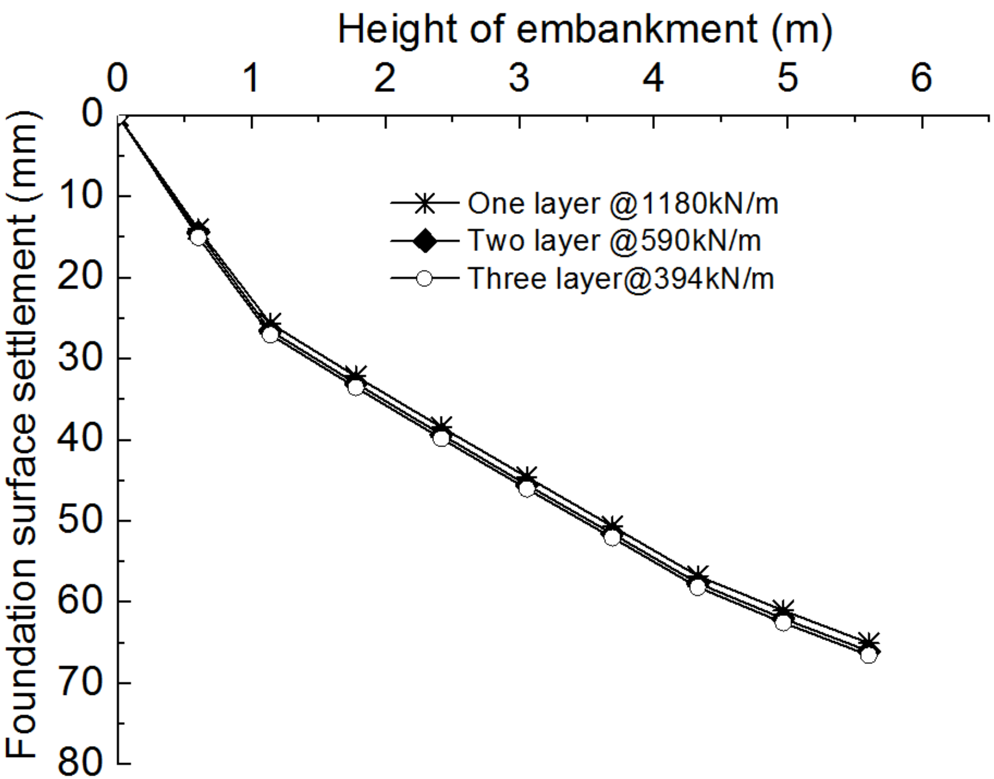






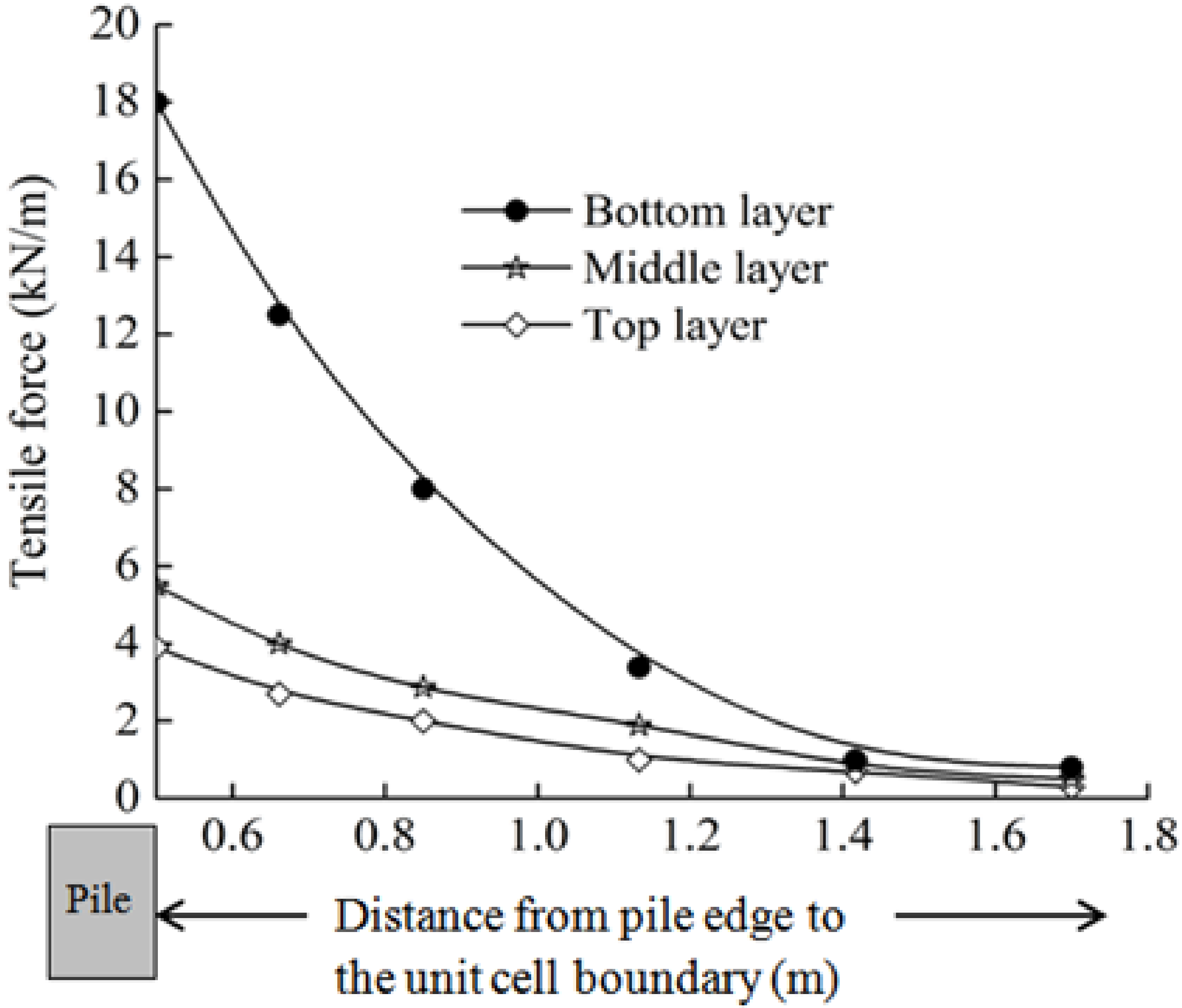
Influence of number of reinforcement layers



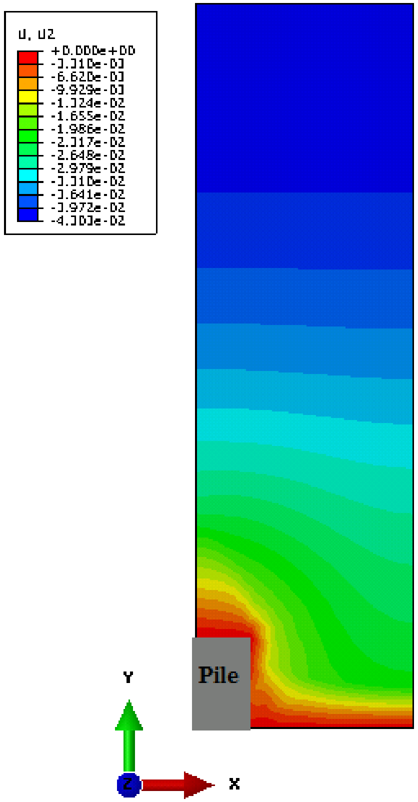


Influence of number of reinforcement layers

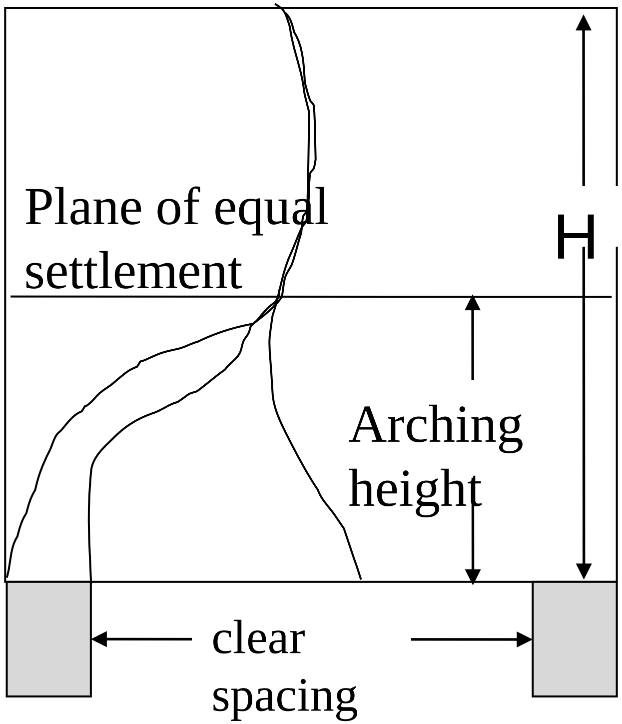
Variation of force in different layers



Determination of arching height from finite element results



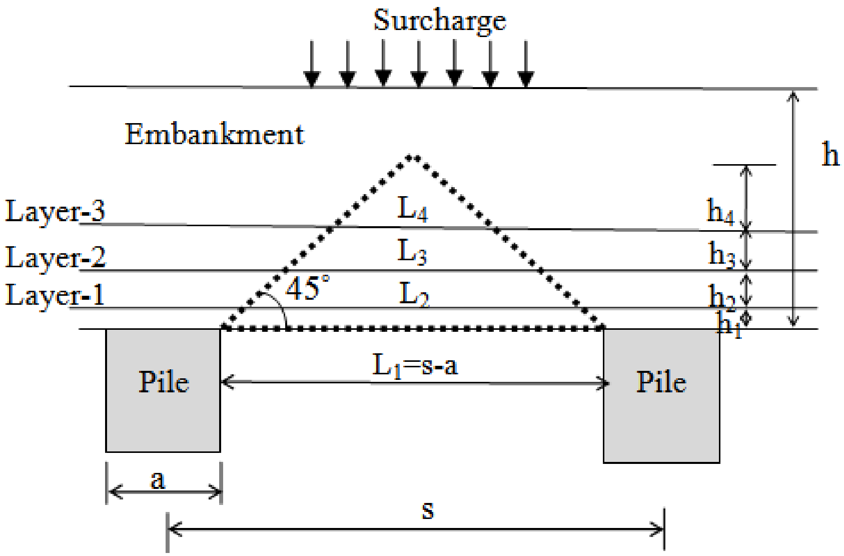
Contours of vertical settlements



Plane of equal settlement

Modulus of reinforcement J (kN/m)	No of reinforcement layers	Bottom Layer	Middle layer	Top layer
	End of construction			
1180	**1**	**18.7**	**-**	**-**
	2	**18.5**	**-**	**6.8**
	3	**18.1**	**5.50**	**3.9**
590	**2**	**13.2**	**-**	**3.5**
394	**3**	**11.4**	**2.8**	**1.7**
	End of consolidation			
1180	**1**	**25.2**	**-**	**-**
	2	**24.9**	**-**	**10.0**
	3	**24.7**	**7.5**	**4.5**
590	**2**	**18.5**	**-**	**6.4**
394	**3**	**16.3**	**5.2**	**3.1**

Collin (2005)



$$W_{Tn} = \frac{(A_n + A_{n+1})h_n\gamma}{2A_n}$$

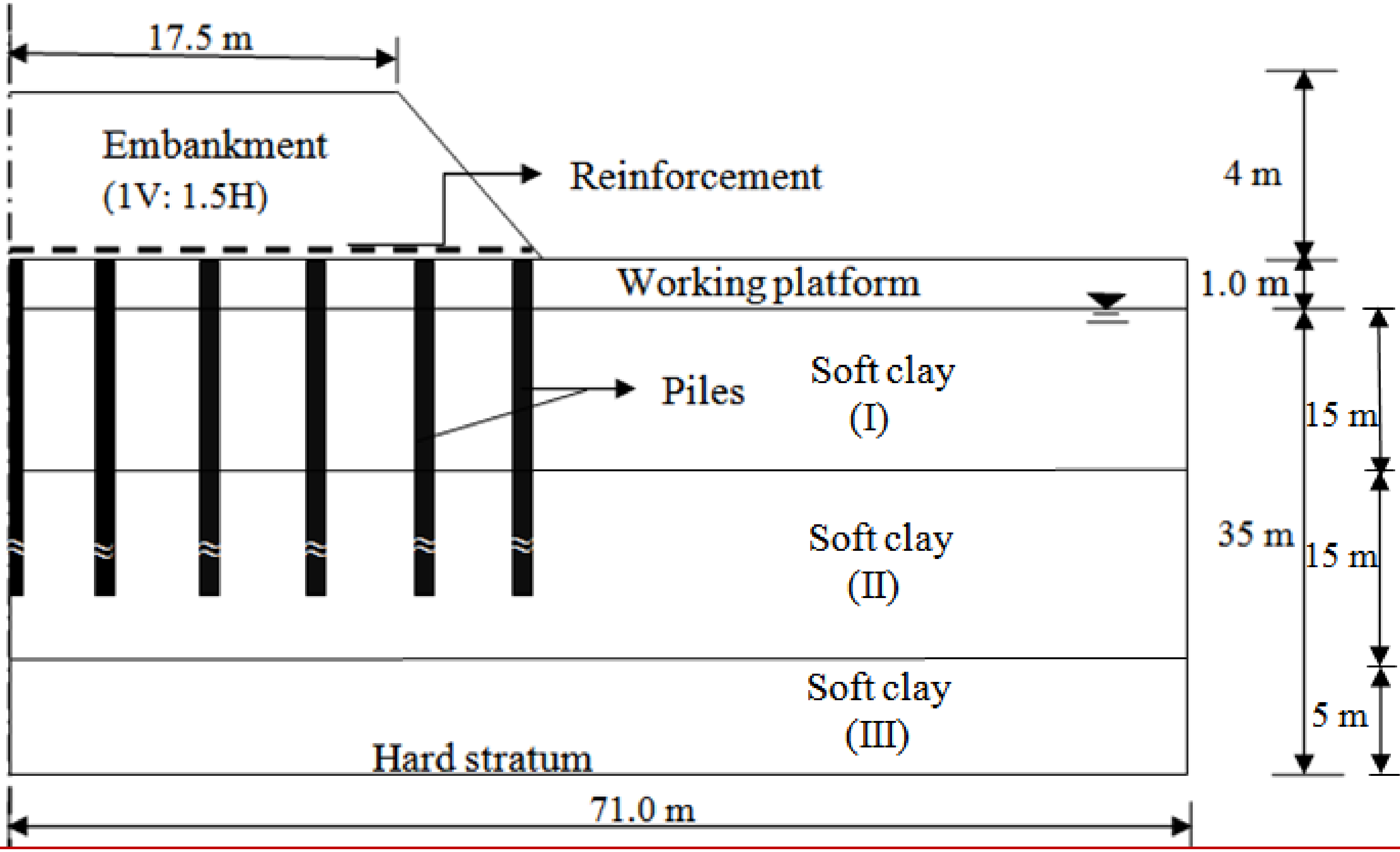
$$T_n = (W_{Tn}\Omega D/2)$$

Forces	Collin's Method (kN/m)	FEA (kN/m)
Layer-1	4.7	11.4
Layer-2	2.6	2.8
Layer-3	1.4	1.7

Observations for multiple layers of reinforcement cases

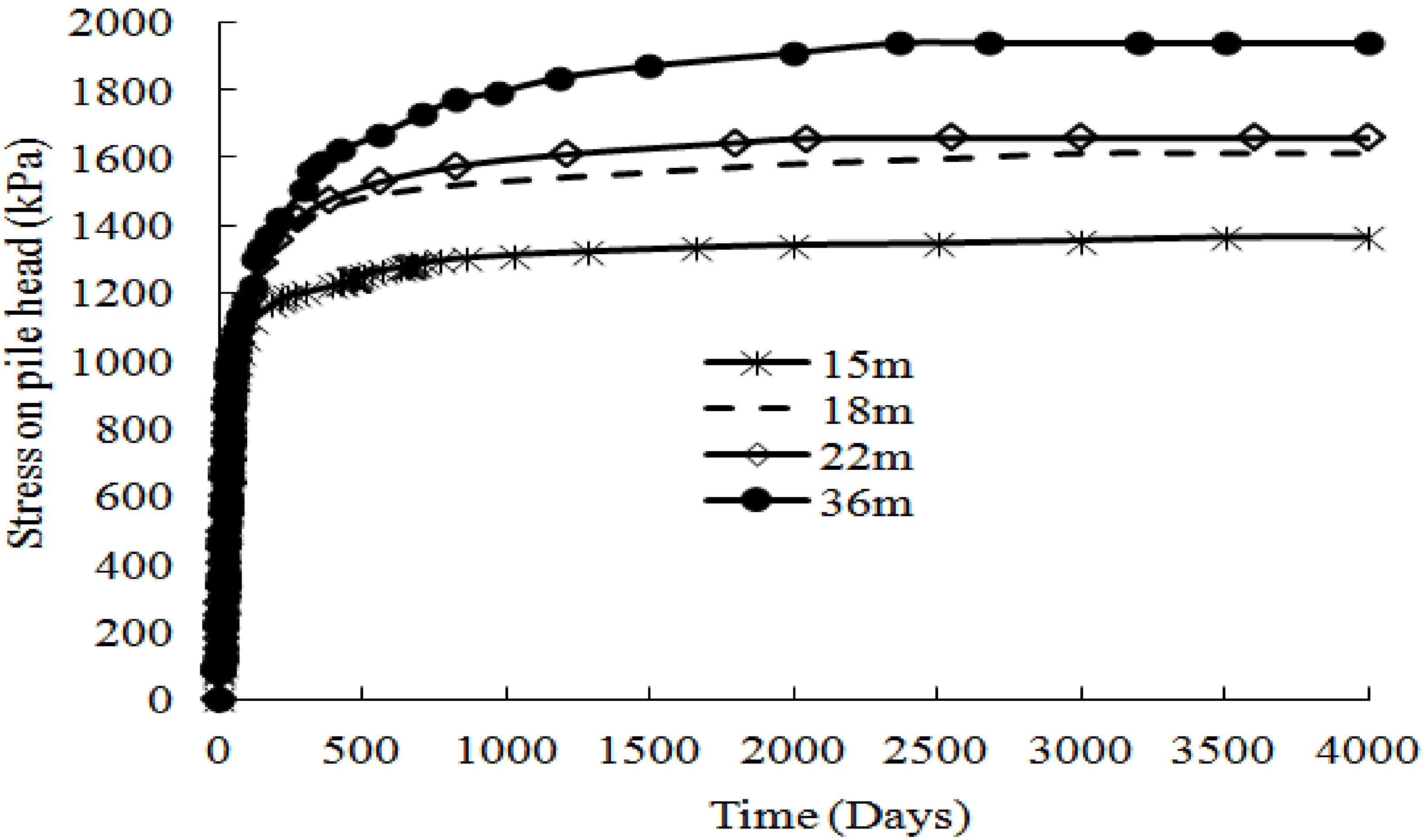
- The maximum force developed in the bottom layer is almost the same irrespective of the no. of reinforcement layers (for the same J).
- Maximum force occurred around the edge of the pile for each reinforcement layer.
- Upper layers mobilised lower forces due to lesser differential settlements at higher elevations.

Embankment Supported on Floating piles

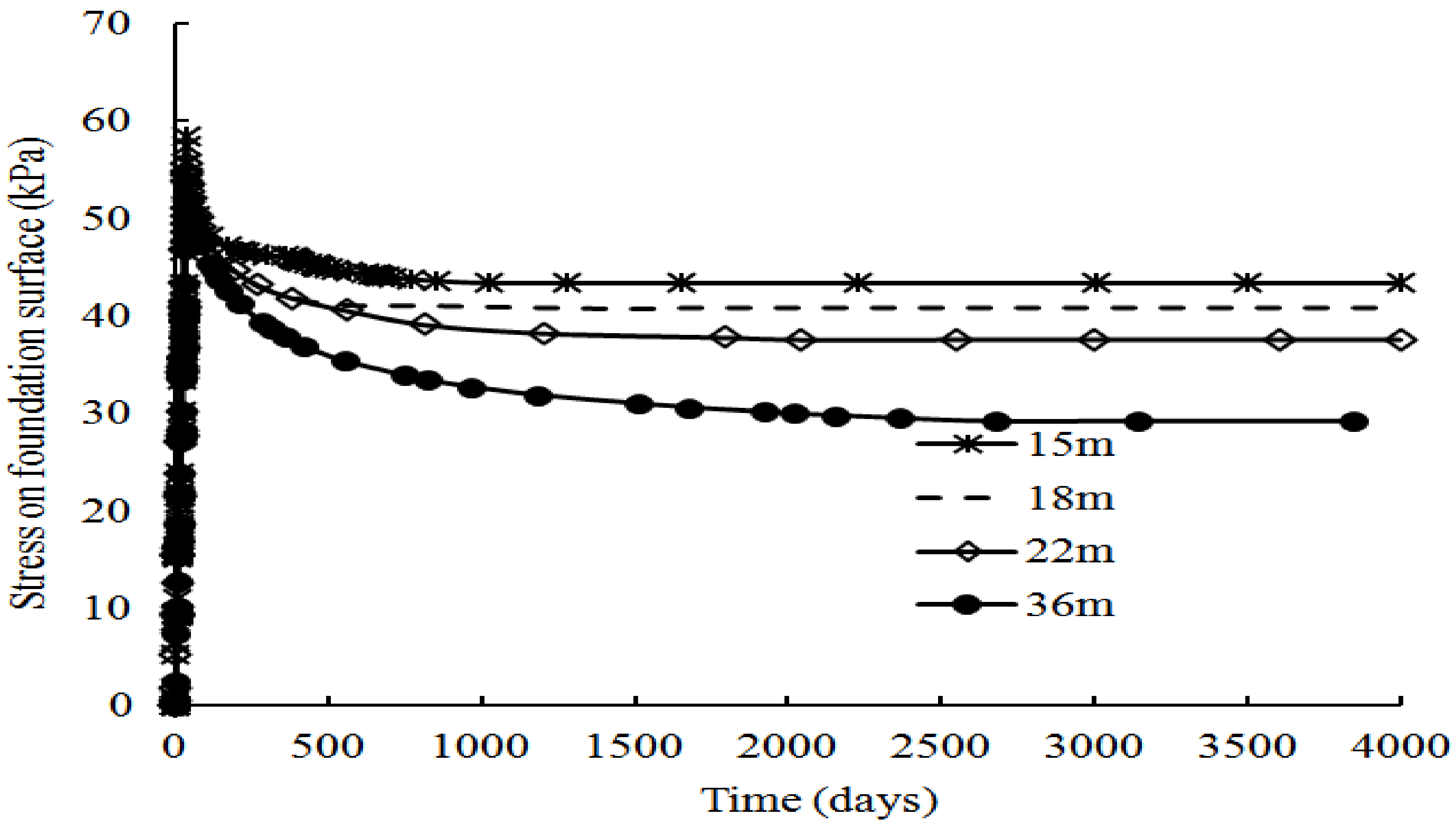


800 mm diameter piles at 2.2 m c/c spacing in square pattern. Area replacement ratio is about 7%.

Stress on the pile head for different lengths of the pile



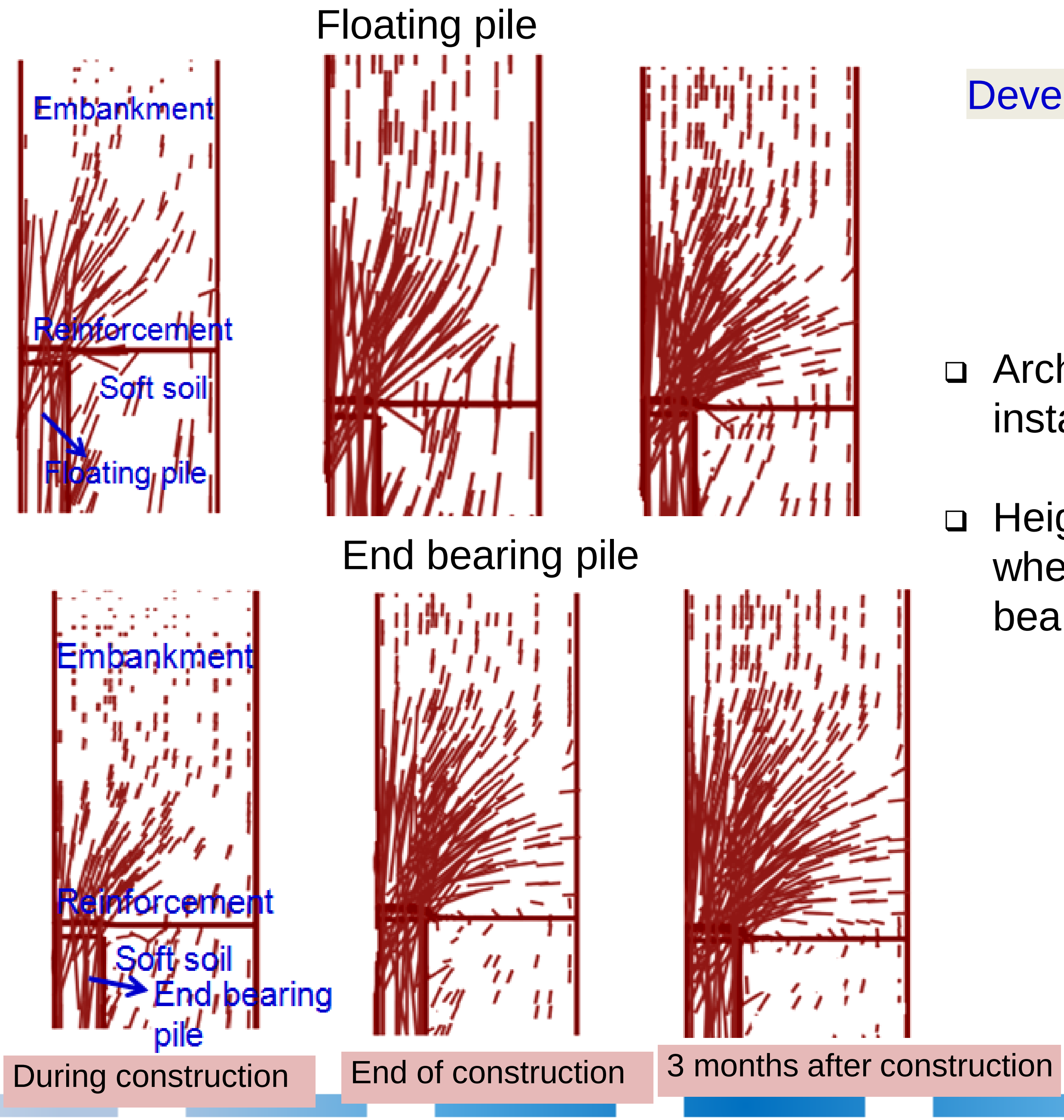
Stress on the soil surface for different lengths of piles



Load sharing for different lengths of the piles

Length of pile, ℓ (m)	End of construction		End of consolidation	
	Pile head (kPa)	Foundation soil surface (kPa)	Pile head (kPa)	Foundation soil surface (kPa)
15 m	841	59	1201	44
18 m	863	57	1363	40
22 m	873	56	1441	38
36 m	873	54	1676	29

Development of soil arching



- Arching action is not an instantaneous phenomenon
- Height of arching is more when the pile is an end bearing one

Numerically predicted vertical stress on foundation soil at the end of construction and end of consolidation (**H= 10 m**)

Embankment height, H (m)	Vertical stress on the soil (kPa)			
	End of construction (kPa)		End of consolidation (kPa)	
	Floating pile (l=22 m)	End bearing pile	Floating pile (l=22 m)	End bearing pile
1.0	9.5	8.7	8.3	7.2
2.0	18.8	17.8	14.6	11.2
3.0	32.8	31.4	24	20.0
4.0	44.9	43.0	37.5	29.2
5.0	56.3	54.3	45.3	41.3
6.0	66.4	66.5	55.2	53
7.0	78.5	76.3	67.3	62.3
8.0	91.2	86.3	80.1	74.4
9.0	102.1	95.4	91.1	80.9
10	113.8	105.6	104.2	90.6

- End of construction - 59% of the overburden stress (Floating piles)
  55% of overburden stress (End bearing piles)
- End of consolidation - Vertical stress acting on the subsoil was found to decrease.

KEY OBSERVATIONS FROM NUMERICAL RESULTS

- Effect of basal reinforcement is dependent on the total modulus provided and not on the number of reinforcement layers.
- Maximum reinforcement force develops in the bottom most layer in the case of multiple basal layers.
- Full arching takes place only after consolidation.
- Arching height is higher for end bearing piles than for floating piles.
- FE predicted arching coefficients are higher than those given in BS8006
- FE results support the Partial and full arching concept of BS 8006 for different height of embankments

Outline of the Lecture

- Introduction
- Geosynthetic Pile Platforms
- Arching Theory
- Centrifuge modelling of embankments
- Numerical analyses
- **Case Histories**
- Design Methods
- Summary

Constructions using GRPES

Most of the constructions took place in the following countries

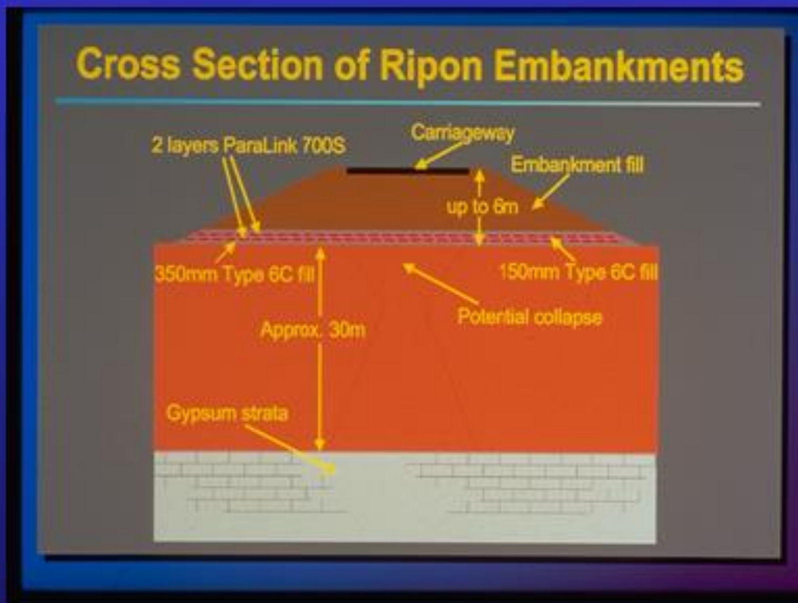
- The Netherlands
- United Kingdom
- Germany
- France
- Scandinavian countries
- China

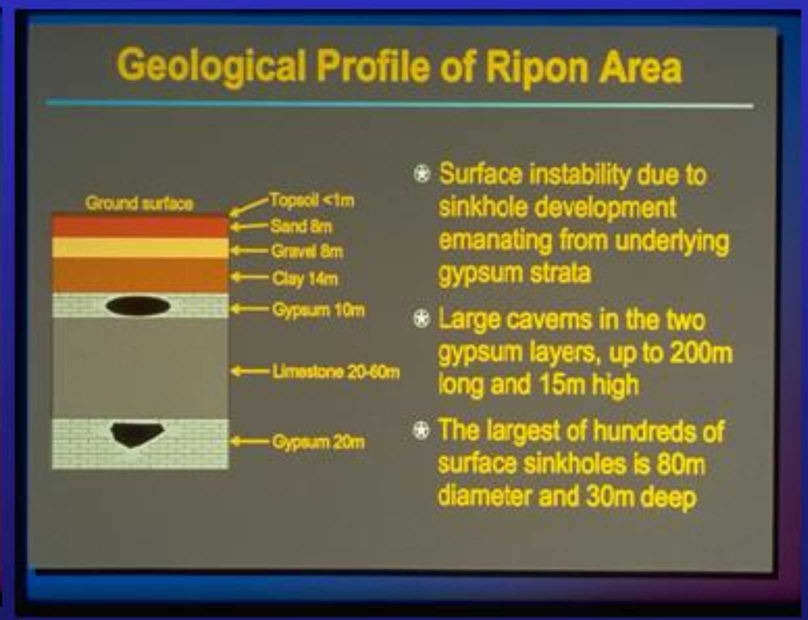
Types of piles

- Timber piles
- Pre-cast concrete piles
- PVC tubes filled with fresh concrete
- Bored cast in situ piles (auger piles)


HUESKER

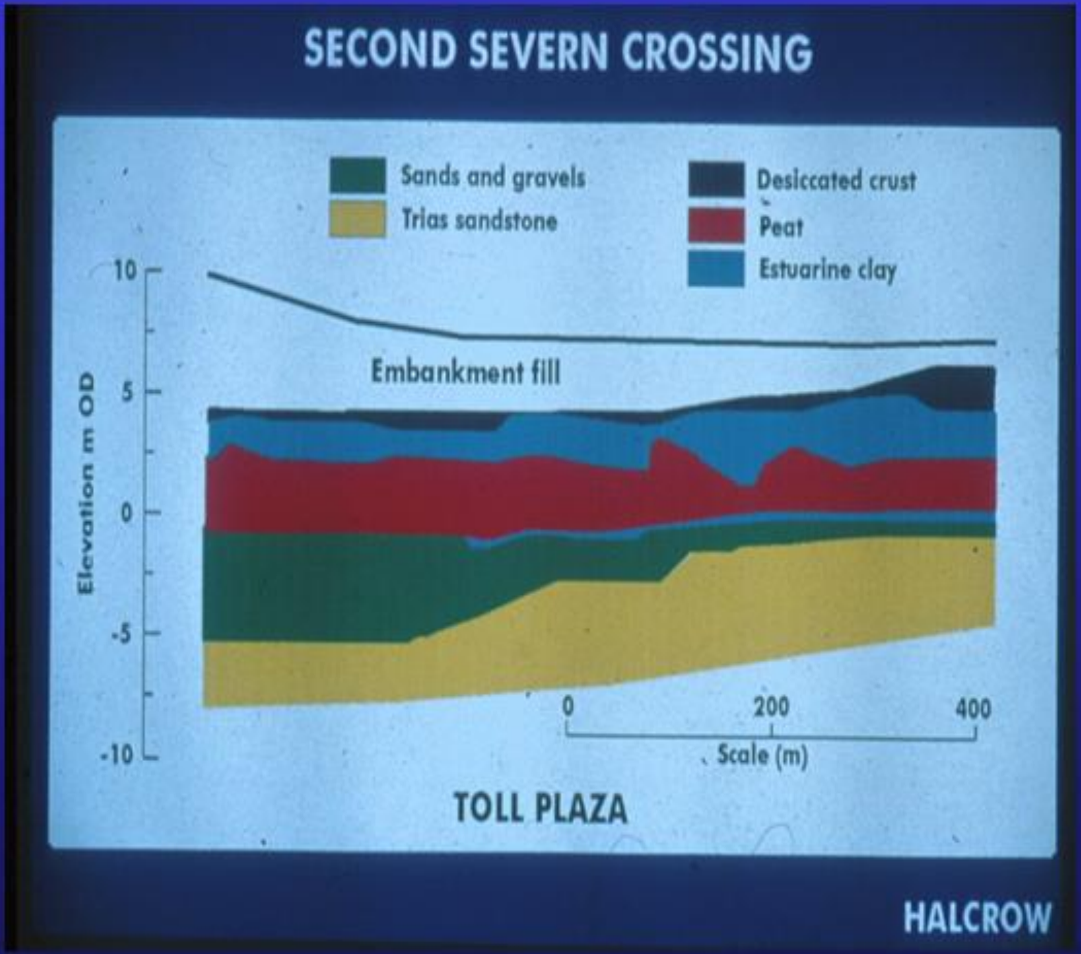
Case History: Ripon By-Pass





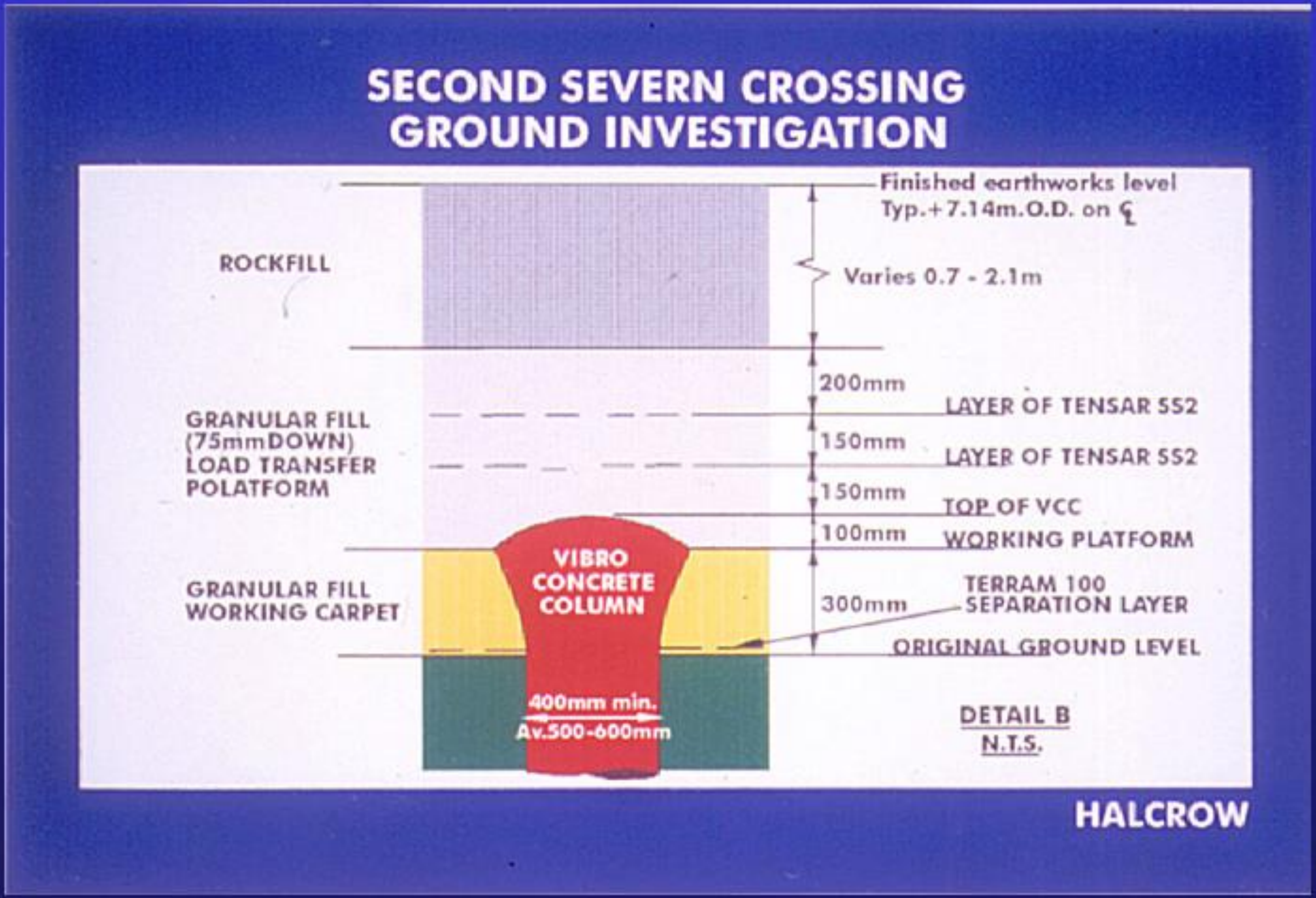
View of Ripon By-Pass

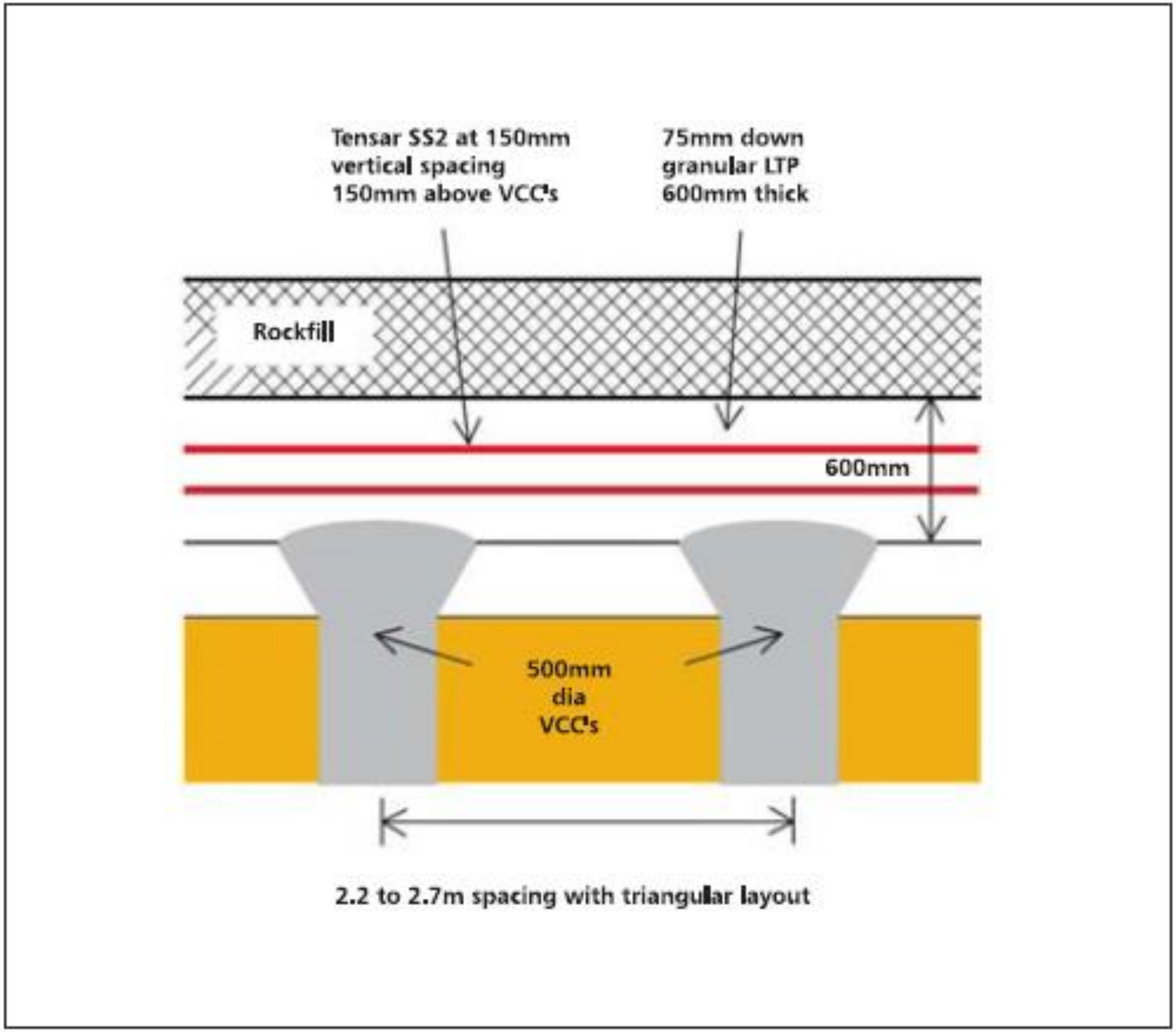
Second Severn Crossing



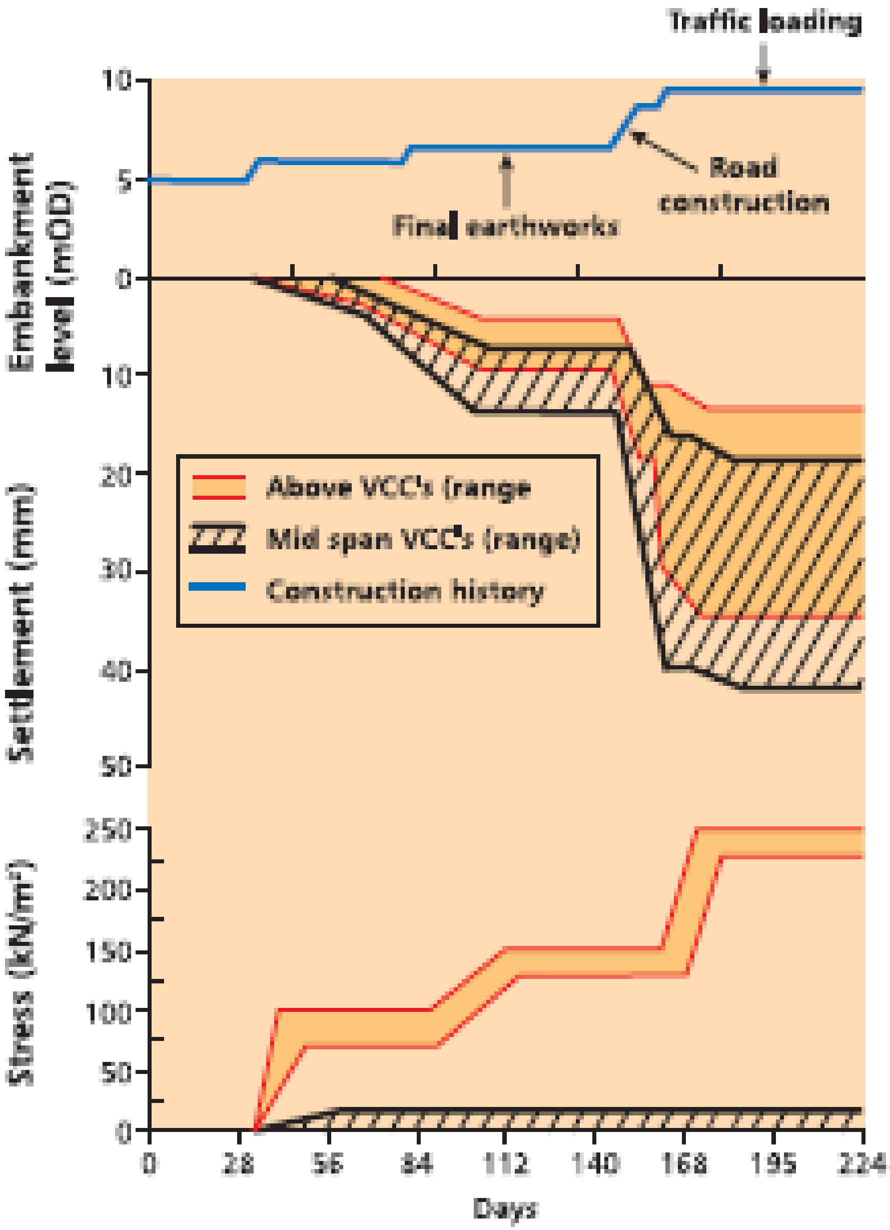
Soils cross section showing extensive deposits of estuarine mud underlain by peat.

Load Transfer Platforms



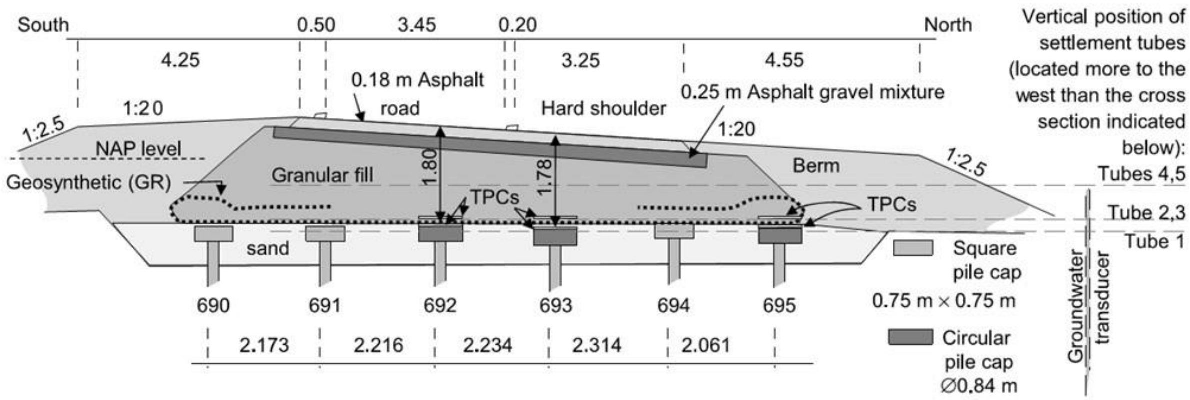


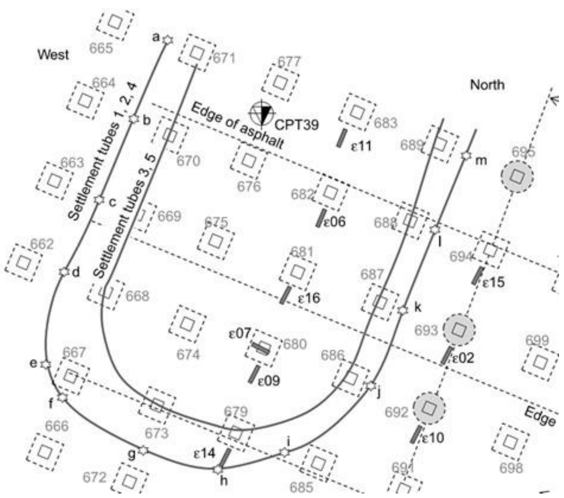
Tensar SS2 at 150mm
vertical spacing
150mm above VCC's
75mm down
granular LTP
600mm thick
Rockfill
600mm
500mm
dia
VCC's
2.2 to 2.7m spacing with triangular layout



Measured data from Second Severn, UK (Tensar, UK)

- Van Eekelen and Han (2020)
- Construction of a new motorway exit near Woerden, the Netherlands





- Project monitored over a period of 4 years.
- Height of fill = 1.8 m;
- 900 square precast concrete piles with side width = 0.29 m and centre to centre spacing = 2.24 m x 2.26 m; Width of square pile cap = 0.75 m.
- Subsoil consists of very soft clay to peat of depth 17 m.
- 2 layers of Geogrid reinforcement having tensile stiffness = 4611 kN/m for 2.5% strain for 10 years is placed between 0.16 m of uncompacted sand and compacted broken recycled construction material (ϕ_{peak} = 49 - 53°).
- The thickness of working platform = 0.75 m.
- Measured ground water level varied between 1.69 – 1.27 m

- Subsoil support was found to be zero. So GR deflection and subsoil deformation patterns were different.
- Total load per pile was found to be equal to the sum of load on pile due to arching and tensioned membrane effect indicating absence of subsoil support.
- Inverse triangular load distribution observed on the load between the GR strips.
- Less arching was observed during rainy season when compared to summer season. This was attributed to the moisture in the fill or return of subsoil support during rainy season due to swelling of the subsoil.

Briancon and Simon (2012)

	Section 1 PE (1.6 × 1.6 m²)	Section 3 GRPE (1.6 × 1.6 m²)	Section 4 GRPE (1.8 × 1.8 m²)	Design criterion
Surface settlement	13.6 cm[a]	8.8 cm[a]	11.2 cm[a]	< 10 cm[b]
Stress applied on pile (kPa)	2769	3062	3914	
Stress applied on soil (kPa)	14.4	1	2	
Load applied on pile (kN)	208	230	294	
Maximum stress inside pile (kPa)	3741	3821	4740	< 7.8
Maximum load inside pile (kN)	281	287	356	< 585
Maximum tensile strength of geogrid (kN/ml)	—	146	189	< 284
Maximum tensile strain of geogrid (%)		1.1	1.45	< 2.2

[a]Total counted from beginning of earthworks; [b]After installation of the ballast layer.

PE – embankment on pile suppot
GRPE – geogrid reinforced piled embankment

Outline of the Lecture

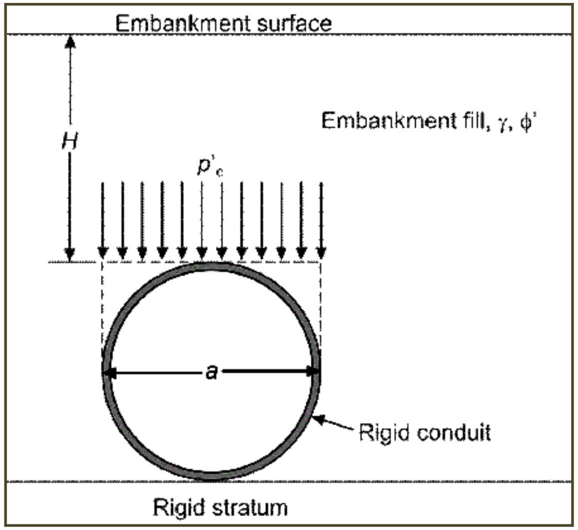
- Introduction
- Geosynthetic Pile Platforms
- Arching Theory
- Laboratory investigations
- Numerical analyses
- Case Histories
- **Design Methods**
- Summary

Design Methods

BS 8006: 2010

- Most widely used method and is highly conservative.

- Positive projecting conduits (Marston's,1913)-Empirical relationship for the ratio of vertical stress acting on the conduit to the average vertical stress at the same depth.

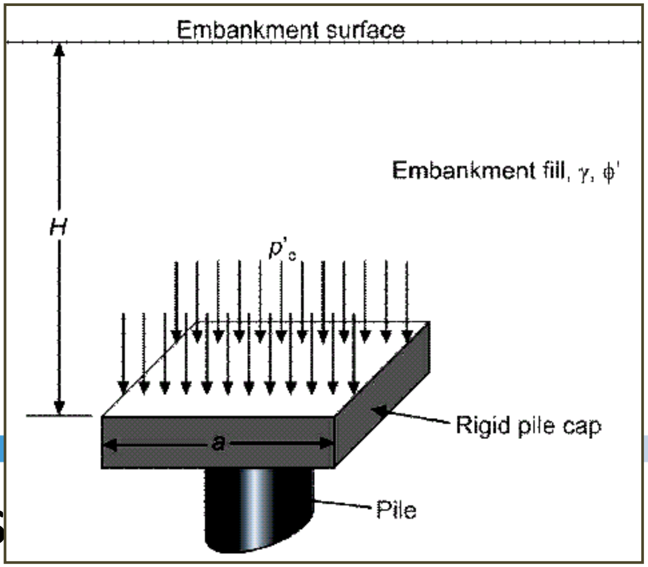


$$\frac{p'_c}{\sigma'_v} = \frac{C_c a}{H} \quad \text{where } \sigma'_v = \gamma H$$

- Jones *et al.*(1990)-Modified Marston's 2D arching approach to 3D

$$\frac{p'_c}{\sigma'_v} = \left(\frac{C_c a}{H}\right)^2$$

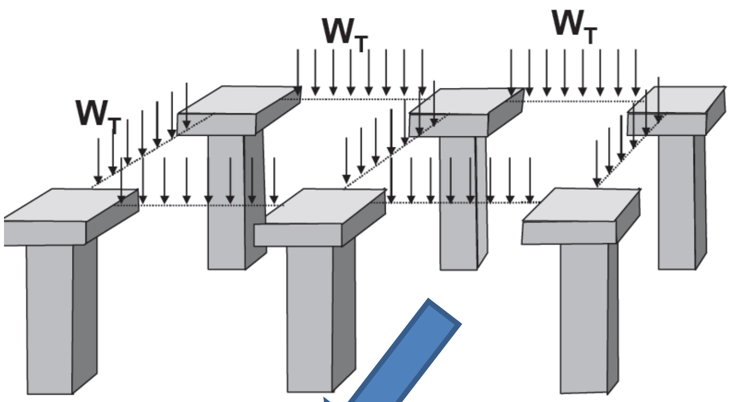
Where C_c is the arching coefficient



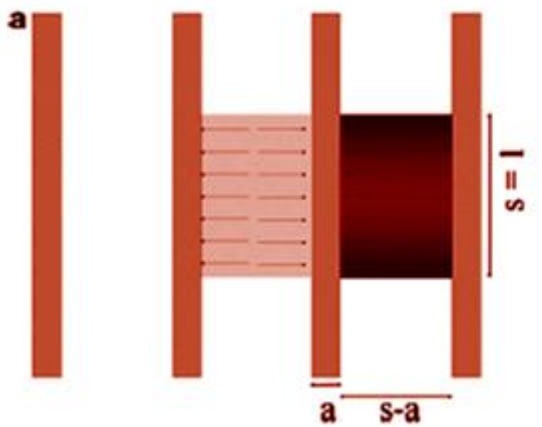
(a) British Standard-BS8006-1:2010 continued…..

$$\text{End bearing piles,} C_c = 1.95\frac{H}{a} - 0.18$$

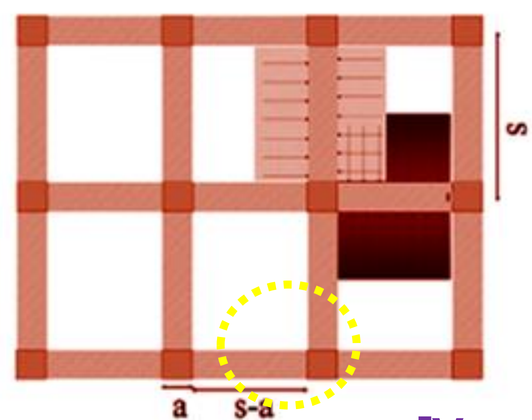
$$\text{Friction piles,} C_c = 1.5\frac{H}{a} - 0.07$$



Tensile force in the reinforcement



2D plane strain configuration

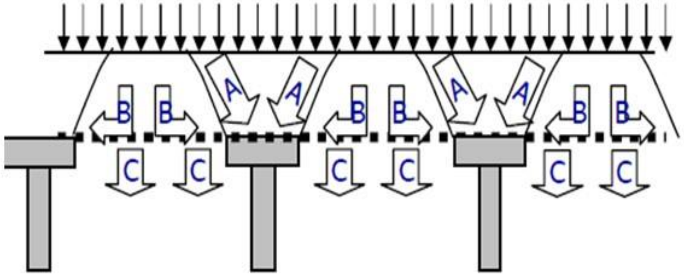


[Van Eekelen et al. 2011]

3D configuration

1. Embankment height is below the critical height of 1.4(s-a):

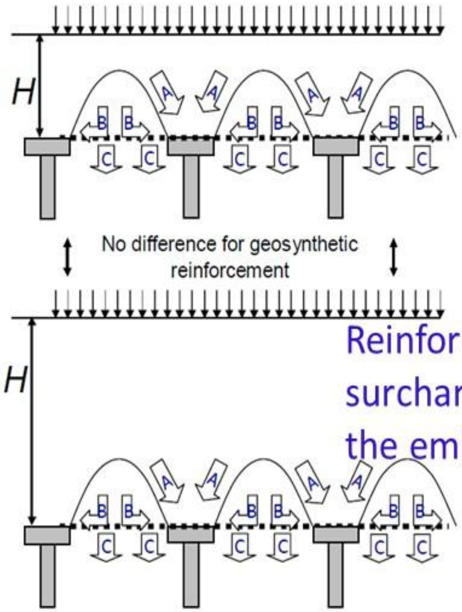
Arching is not fully developed



A= Load acting on the piles due to arching
B= Load taken by the geosynthetic and
C= Load acting on the soft subsoil

2. Embankment height is above the critical height of 1.4(s-a):

Full arching is developed



Reinforcement does not feel the surcharge load or the extra height of the embankment beyond H_{cr}

Design Guidelines British Standard : BS 8006-1(2010)

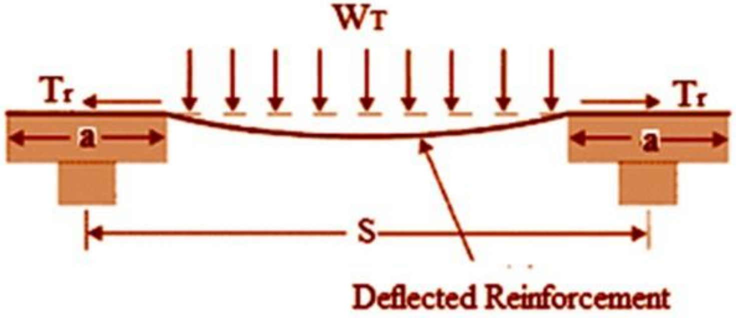
$$W_T = \frac{s(\gamma H + p)}{s^2 - a^2}\left[s^2 - a^2\left(\frac{p'_c}{\sigma'_v}\right)\right]$$

Full arching, H=s-a

P = 0

From partial to full arching →

$$W_T = \frac{1.4 s\gamma(s-a)}{s^2 - a^2}\left[s^2 - a^2\left(\frac{p'_c}{\sigma'_v}\right)\right]$$



Equilibrium of tension membrane

- Differential equation for the tension membrane is derived and solved resulting in the tensile force in the reinforcement

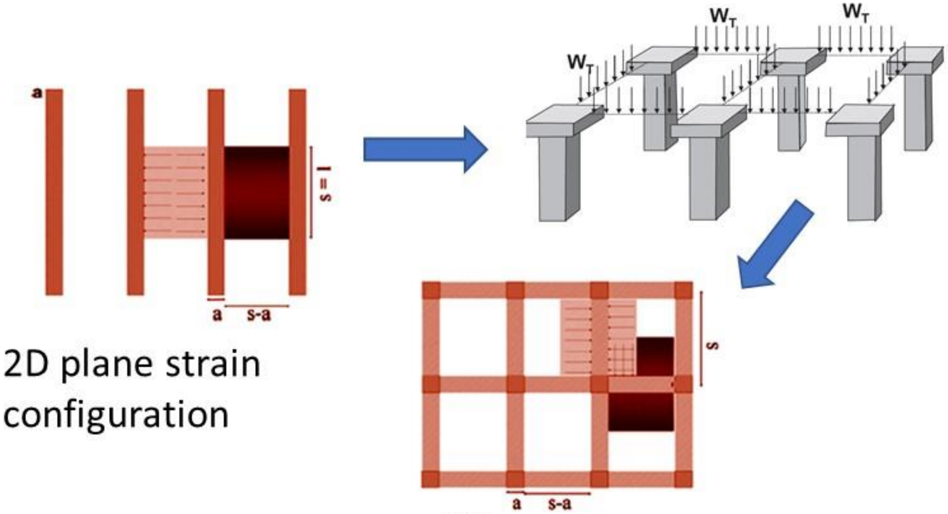
$$\text{Geosynthetic Tension, } T_r = \frac{W_T(s-a)}{2a}\sqrt{1 + \frac{1}{6\varepsilon}}$$

where ε is the geosynthetic strain

Modified BS 8006 (van Eekelen et al. (2011))

- The difference between BS 8006 (2010) and Modified BS 8006 proposed by van Eekelen et al. (2011) is in the calculation of load on GR strip.
- In BS 8006(2010) area that transfers tensile load into the reinforcement is taken as a strip width of ‘s’.
- 2D situation is considered.
- In modified BS 8006, 3D configuration is considered and GR strip carries half of the load on one square. So double load calculation error eliminated.

Tensile force in the reinforcement



2D plane strain configuration

3D configuration

[Van Eekelen et al. 2011]

Design Guidelines British Standard : BS 8006-1(2010)

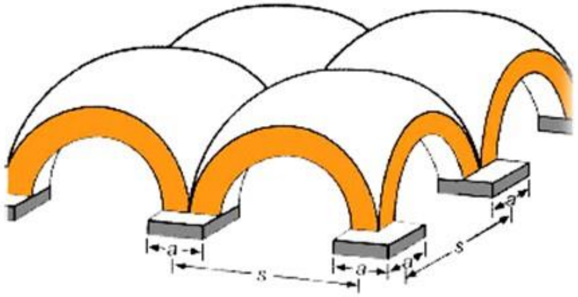
Hewlett and Randolph Method (1988)

- This theory is based on limit state of soil in hemispherical domed region
- The stresses are calculated considering the vertical equilibrium

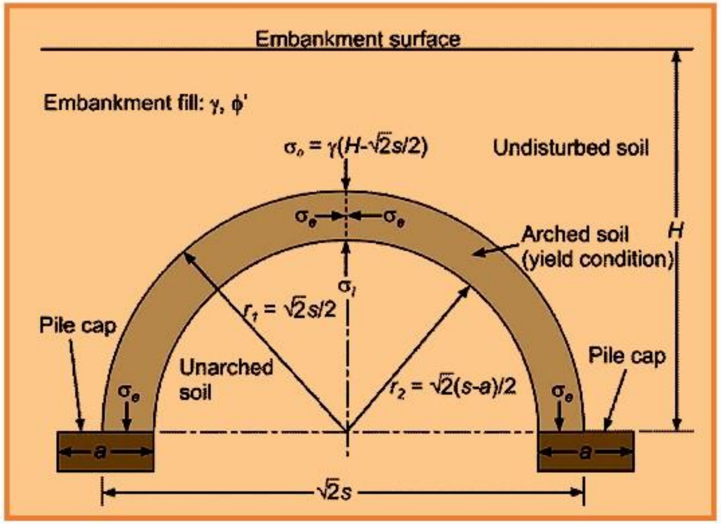
$$\frac{d\sigma_R}{dR}+\frac{2(1-K_p)\sigma_R}{R}=-\gamma$$

$$\text{where } R=\frac{(s-a)}{\sqrt{2}} \text{ inner radius}$$

$$=\frac{s}{\sqrt{2}} \text{ outer radius}$$



Hemispherical domes



(Hewlett & Randolph, 1988)

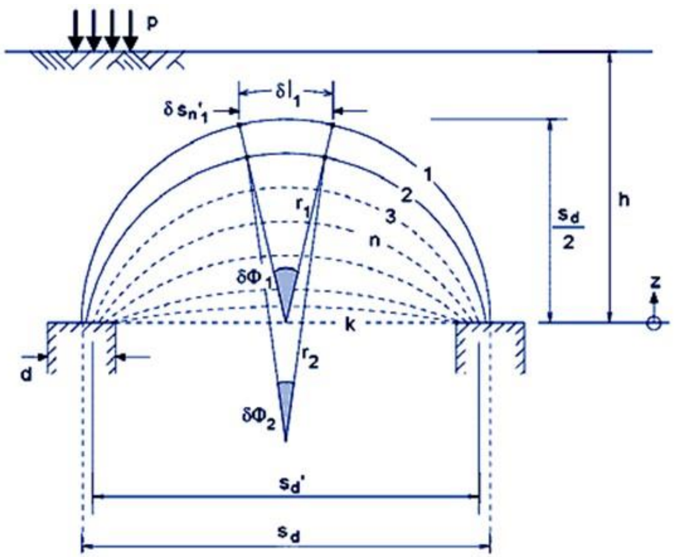
Design Guidelines British Standard : BS 8006-1(2010)

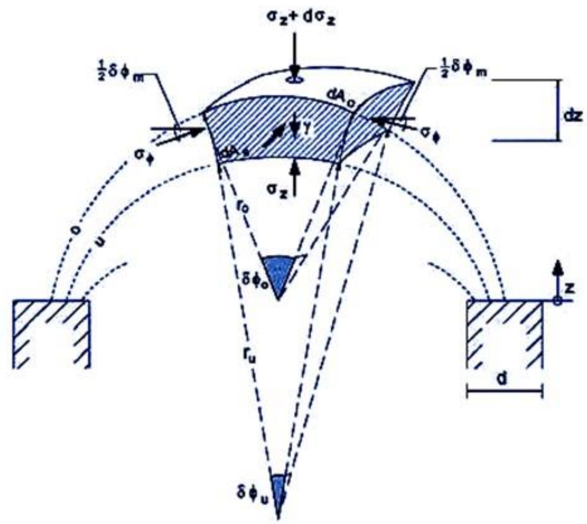
- Method considers equilibrium at two critical positions 1) crown of the arch 2) pile cap surface.
- Efficacy at two critical locations are determined and the minimum value is considered for the load determination calculations.
- $E_{crown} = 1 - (1 - \delta^2)(A - AB + C)\ where\ \delta = \frac{a}{s} and\ K_p = \frac{1+sin\phi}{1-sin\phi}$
- $A = (1 - \delta^2)^{2(K_p-1)}; B = \frac{\frac{s}{\sqrt{2H}}[2K_p-2]}{[2K_p-3]}; C = \frac{\frac{s-a}{\sqrt{2H}}[2K_p-2]}{[2K_p-3]}$
- $E_{cap} = \frac{\beta}{1+\beta}\ where\ \beta = \frac{2K_p}{K_p+1} . \frac{1}{1+\delta} . ((1 - \delta)^{-K_p} - (1 + \delta K_p))$
- Load on the piles $= E_{min} \times s^2 \times \gamma H$
- Equally distributed load on the GR, $W_t = s\,\gamma H\ (1 - E_{min})s^2$

- Design methods are also available in German guidelines EBGEO (2010)
- Dutch guidelines CUR 226 (2016)
- Brief description of the piled embankments & Link for the Excel spread sheet program with basic equations of the concentric arches model are available at: www.piledembankments.com

German guidelines EBGEO (2010)

Based on multi-shell arching theory of Zaeske (2001).



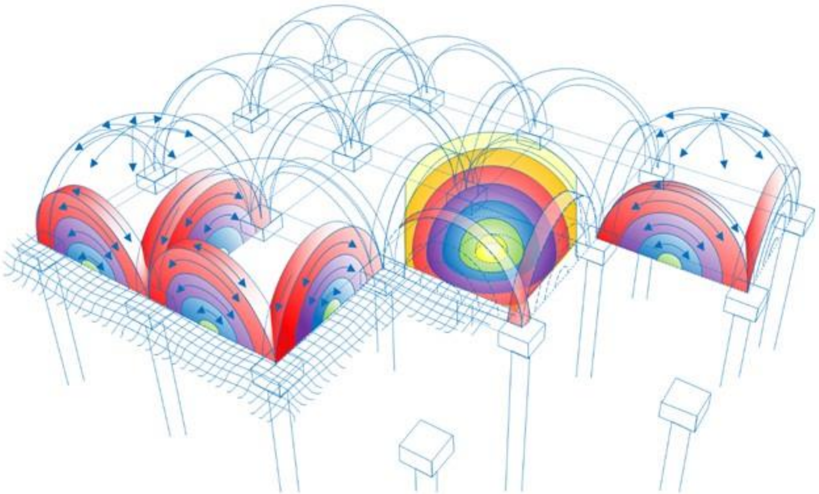


Multi shell arching theory adopted in New German Method (Kempfert,2004)

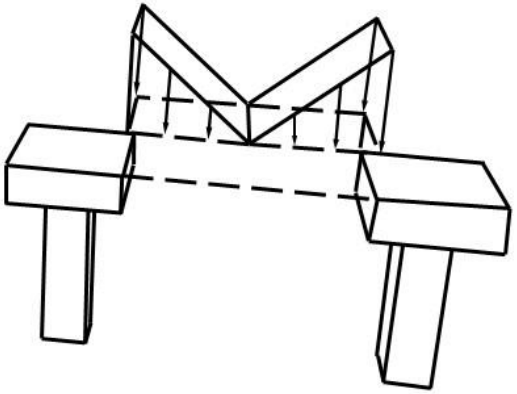
$$-\sigma_z dA_u + (\sigma_z + d\sigma_z) dA_o - 4\sigma_\varphi dA_s \times Sin\left(\frac{\delta\Phi_m}{2}\right) + \gamma dV = 0$$

Design Guidelines Dutch Standard : CUR 226(2016)

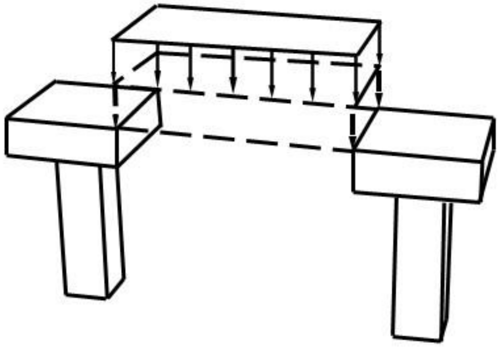
- Adopts major parts of the German EBGEO 2010.



Concentric Arches Model



Inverse triangular load distribution



Uniform load distribution

SUMMARY

- Geosynthetic Reinforced Piled Embankment systems (GRPES) require less construction times compared to other methods
- Post-construction settlements can be reduced to a large extent
- Require low maintenance during the service life
- Design methods available in BS8006, EBGEO & Dutch standards

ACKNOWLEDGEMENTS

- The speaker is thankful to several past and current students and colleagues for their input
- Dr. Anjana Bhasi of NIT Calicut for the results from different finite element analyses
- Ms. Reshma for centrifuge model tests with different configurations
- Prof. BVS Viswanadham, IIT Bombay for co-guiding Ms. Reshma & help with centrifuge model tests at IIT Bombay

REFERENCES

1. Biot, M.A. (1941) General theory of three dimensional consolidations. *Journal of Applied Physics*, **12**,155-169.
2. British Standards BS8006: 2010 Code of practice for strengthened/Reinforced soils and other fills. Section 8.3.3 British Standard Institution, *ISBN 978-0-580-53842-1*
3. Britton, E. and P. Naughton (2008) An experimental investigation of arching in piled embankments. Proceedings of EuroGeo 4, Edinburgh, UK, Paper No. 106.
4. CUR 226 (2016) *Dutch design guideline for piled embankments*. ISBN 978 –90–376-0518-1.
5. EBGEO (2010).*German Standrad*-Recommendation for design and analysis of earth structures using geosynthetic reinforcements. German Geotechnical Society ,Ernst &Sohn GmbH &Co.
6. Jones, C.J.F.P., C .R. Lawson and D.J. Ayres Geotextile reinforced piled embankments. pp. 155-160. In Den Hoed (eds.) Geotextiles, Geomembranes and related products, Balkema, Rotterdam, 1990.
7. Kempfert, H. G., D. Zaeske and D. Alexiew (1999) Interactions in reinforced bearing layers over partial supported underground. Proceedings of the 12th European Conference on Soil Mechanics and Geotechnical Engineering, Amsterdam, 31, Balkema, 1527-1532
8. Kempfert, H.G., C.Gobel,D.Alexiew and C. Heitz (2004) German Recommendations for Reinforced Embankments on Pile-Similar Elements. Proceedings of the EuroGeo3,Munich DGGT,279-284
9. McNulty,J.W (1965) An experimental study of arching in sand. Tech Report I-674,U.S.Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi.
10. Russell, D. and N. Pierpoint (1997) An assessment of design methods for piled embankments. *Ground Engineering*, 30(11), 39-44.
11. Han, J. and M.A. Gabr (2002) Numerical analysis of geosynthetic-reinforced and pile-supported earth platforms over soft soil. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE,128(1), 44-53.

REFERENCES

12. Hewlett, W.J. and M.F. Randolph (1988) Analysis of piled embankments. *Ground Engineering*, 21(3), 12-18.
13. Liu, H.L., W. W. Charles, and K. Fei (2007) Performance of a geogrid-reinforced and pile-supported highway embankment over soft clays-Case study. *Journal of Geotechnical and Geoenvironmental Engineering*, *ASCE*, 133(12), 1483-1493.
14. Marston, A. and A.O. Anderson (1913) The theory of loads on pipes in ditches and tests of cement and clay drain tile and sewer pipe. *Engineering experiment station*, Bulletin No.31.
15. Satibi, S. (2009) Numerical analysis and design criteria of embankments on floating piles*. A PhD thesis submitted to the Universität of Stuttgart, Stuttgart, Germany.*
16. van Eekelen, S.J.M., and A. Bezuijen (2011) Analysis and modification of the British Standard BS8006 for the design of piled embankments*. Geotextiles and Geomembranes,* 29,349-351
17. van Eekelen, S.J.M. and Han, J. (2020) Geosynthetic-reinforced pile-supported embankments: state of the art, Geosynthetics International, Vol. 27, N0. 2, 112-141.
18. Zaeske,D.(2001) ZurWirkungsweise von unbewehrten und bewehrtenmineralischen Tragschichtenuerpfahlartigen Grundungsetementen. Schriftenreihe Geotechni, University of Kassel, Germany.

Thank you!

Connect With Us On LinkedIn!
https://www.linkedin.com/company/international-geosynthetics-society

Like Us On Facebook!
www.facebook.com/GeosyntheticsSociety

Follow Us On Twitter!
@IntGeosynthSoc