INTRODUCTION

LIGHTWEIGHT FILL TO REDUCE SETTLEMENT ON BRIDGE APPROACH EMBANKMENTS TREATED WITH VIBRO STONE COLUMNS

by

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INTRODUCTION

- ORIGINAL GROUND TREATMENT
- OBSERVED DISTRESS
- SETTLEMENT BACK ANALYSIS
- PROPOSED RECTIFICATION
- SUMMARY & LESSONS LEARNED

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BACKGROUND

The project site of the road overbridge (ROB) is locate at the Northern part of Malaysia.

- Consist of 3 road embankments which form a T-junction of the ROB.
- The bridge approach embankment is max 10m high and treated with vibro stone column with reinforced soil wall on both sides.
- Pile embankment was used as transition between the embankment on treated ground using stone column and the bridge.

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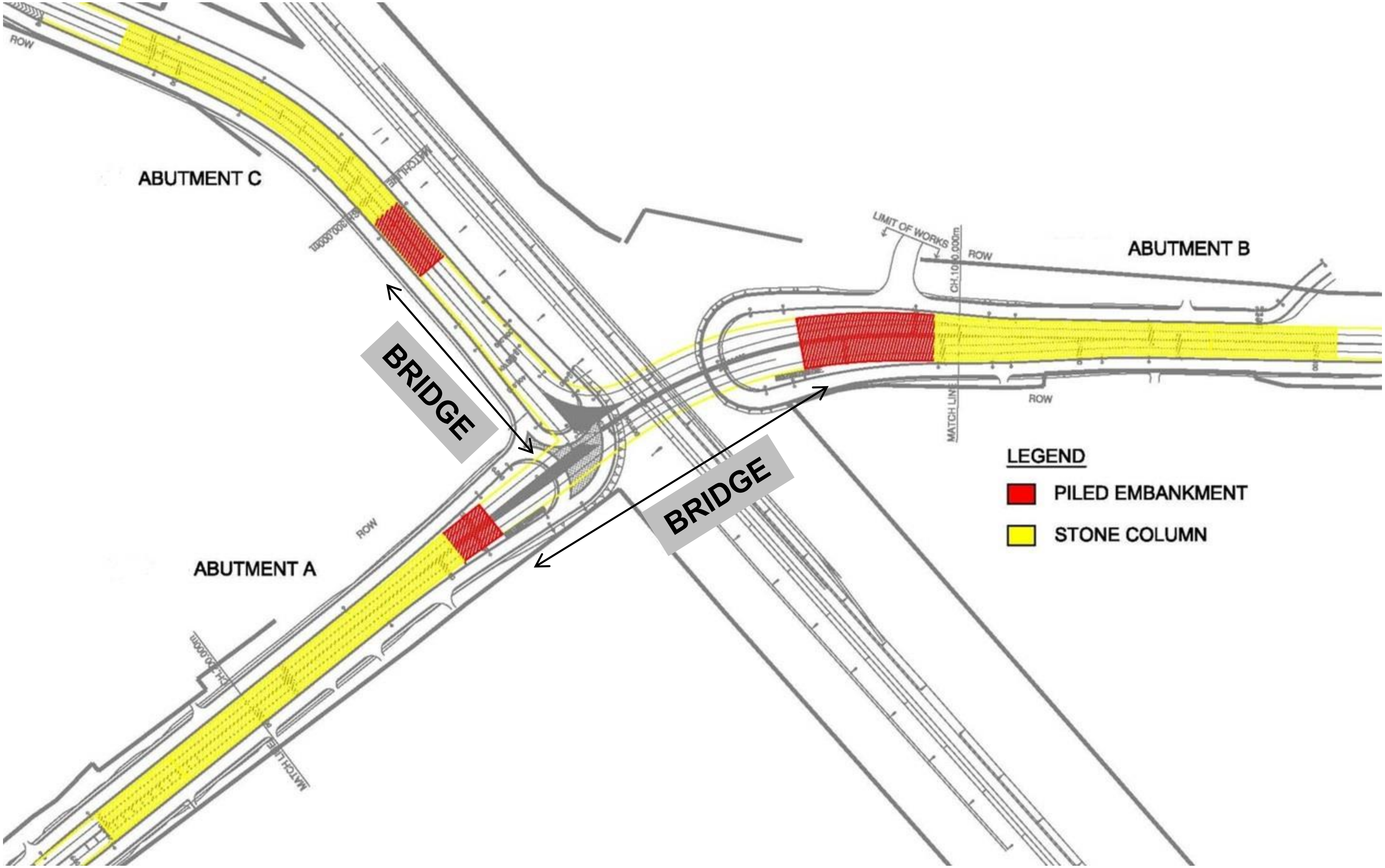
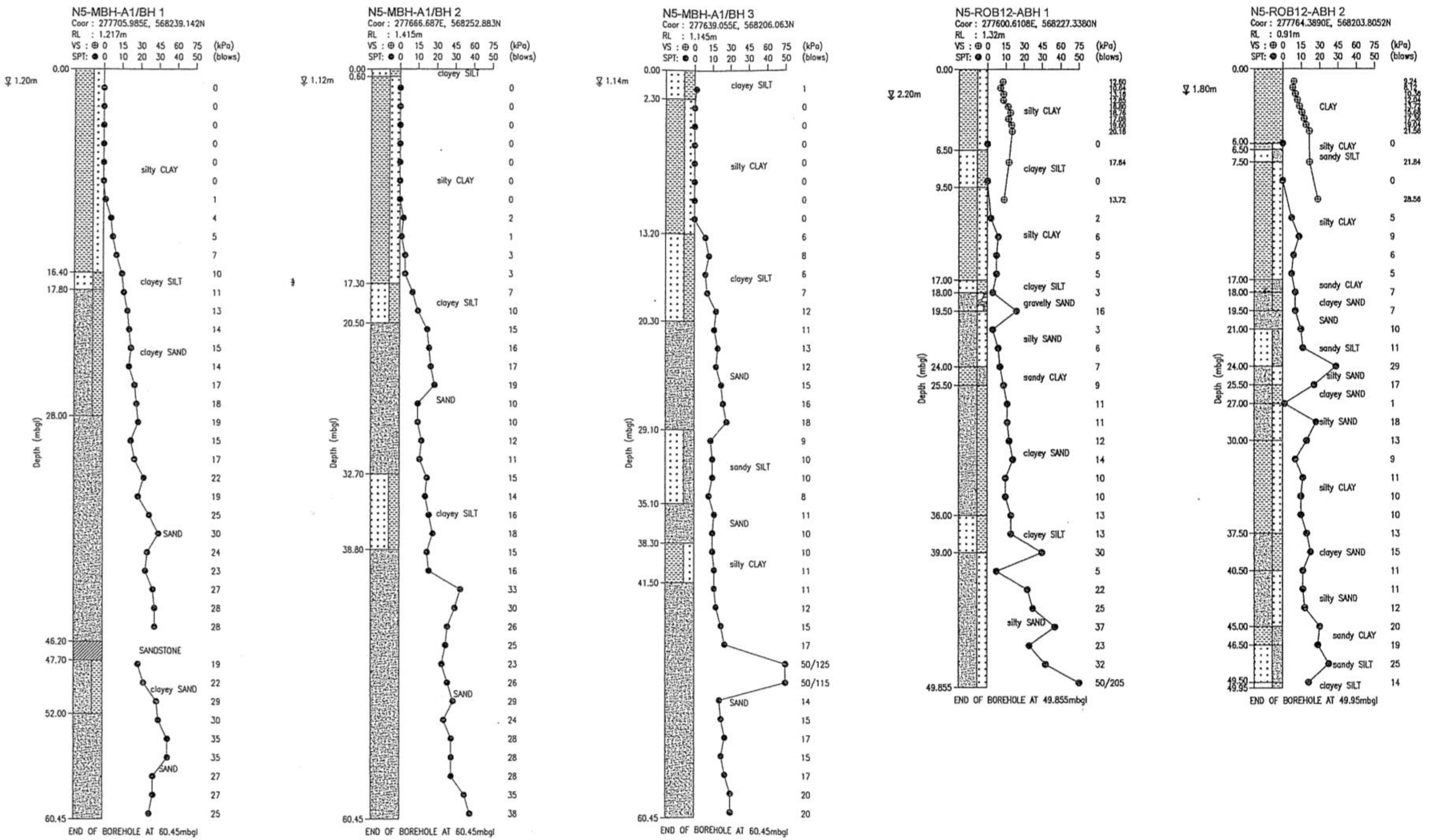
ORIGINAL GROUND TREATMENT



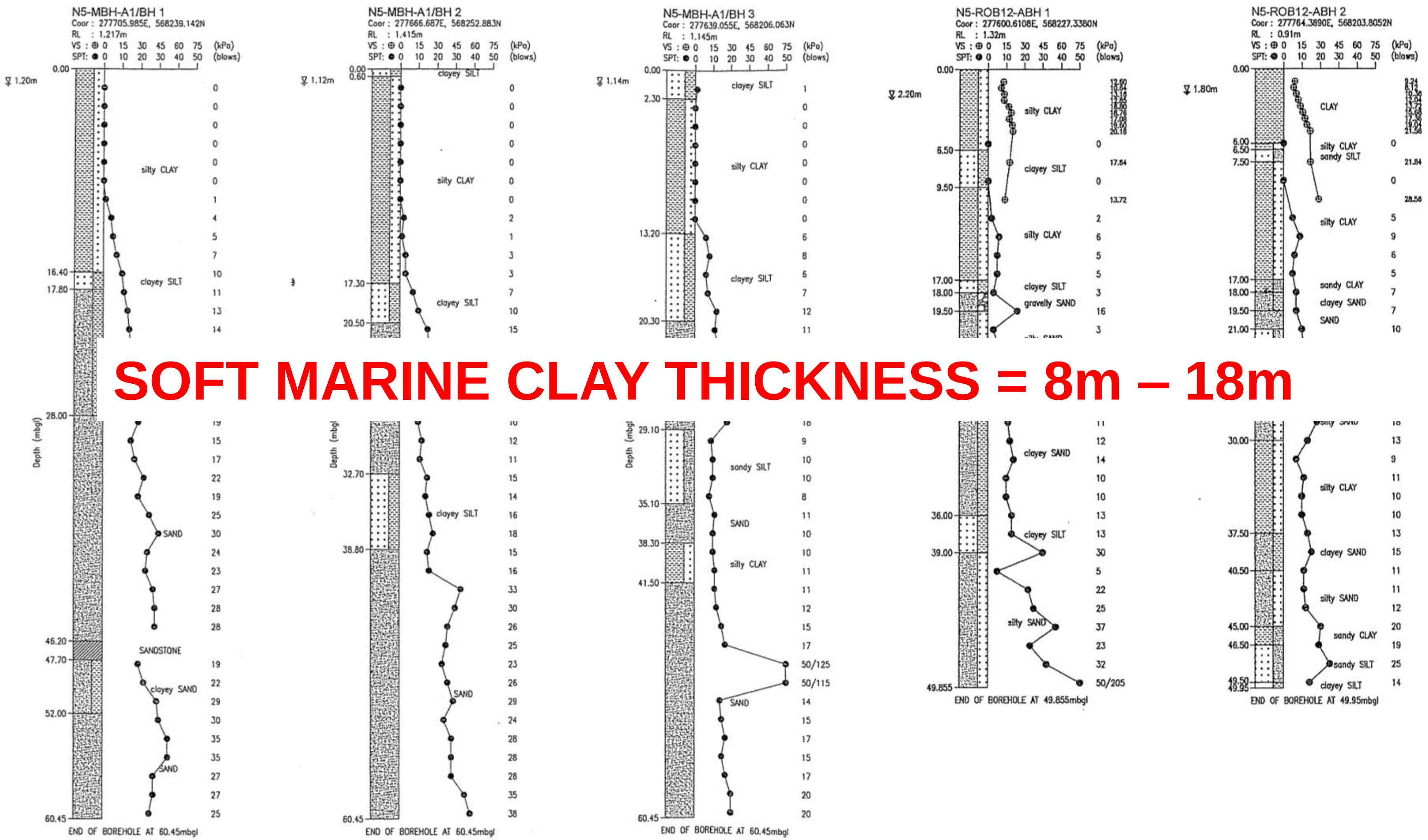
SOIL PROFILE



SIMPLIFIED BORELOGS PROFILE

SCALE 1:300

SOIL PROFILE



SOFT MARINE CLAY THICKNESS = 8m – 18m

SIMPLIFIED BORELOGS PROFILE

SCALE 1:300

SOIL PROPERTIES

Unit Weight (kN/m^3)A	Shear Strength (kPa)	Void Ratio, e_o	Compression Index, Cc	Plasticity Index (%)	Liquid Limit (%)
15 - 17	10 - 30	2 – 3	0.63 – 0.78	30 - 90	60 - 140

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DESIGN CRITERIA

- Total residual post construction settlement within the first seven (7) years of service shall not exceed 400mm.

- The differential settlement within the first seven (7) years of service shall not exceed 100mm within a length of 50m

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ORIGINAL GROUND TREATMENT

- Stone column of minimum diameter of 1m
- Spacing of stone column varies with fill embankment heights

Design Fill Height (m)	Stone Column Spacing (m)	Stone column Length (m)
2-4	2.2 x 2.4	12-16
4-6	2.2 x 2.2	12-16
6-7.5	1.9 x 1.9	12-16
7.5-9	1.65 x 1.75	12-16
9-10	1.5 x 1.5	12-16

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OBSERVED DISTRESS

Distress on RS wall panels was observed while the embankment filling was still in progress along the embankment on treated ground using stone column.

The observed distress were:

- Opening of panels
- High differential settlement

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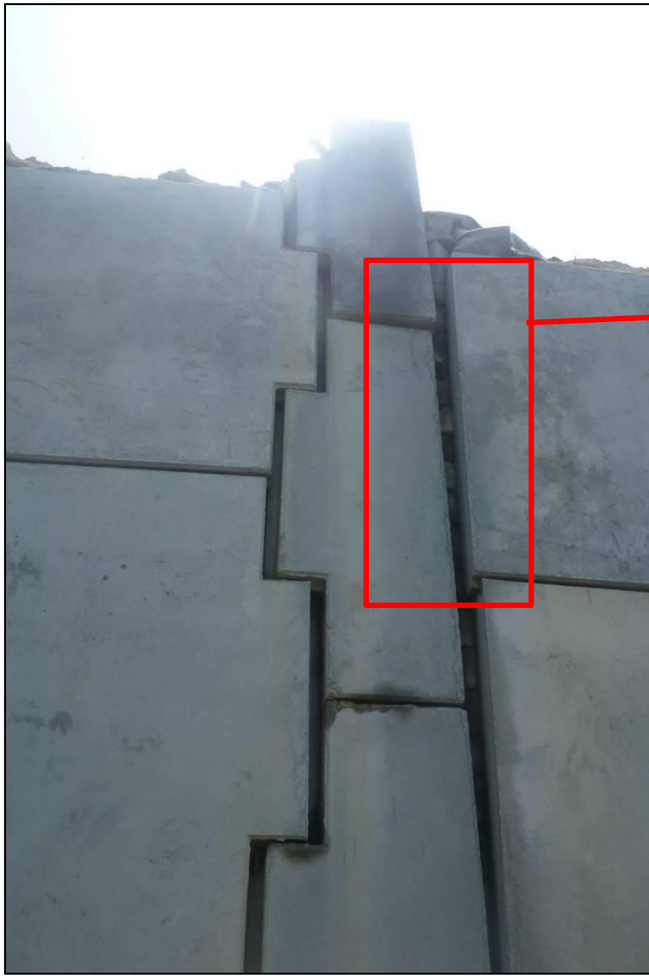
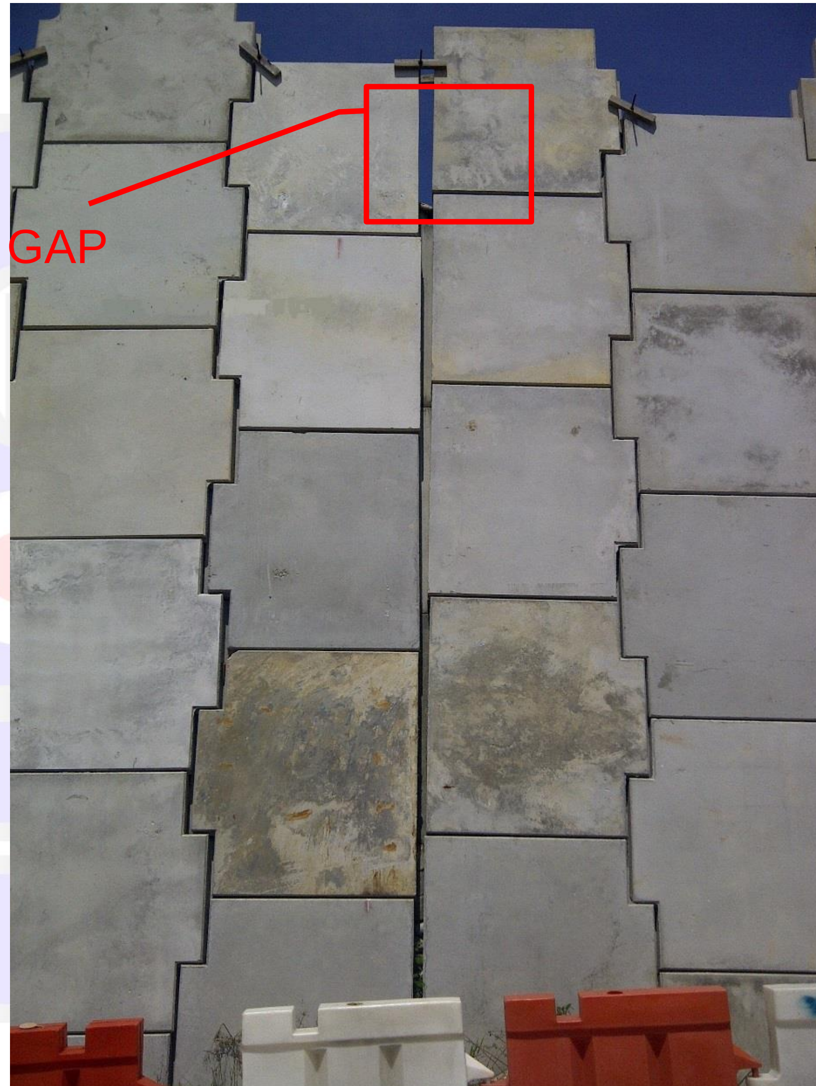
DISTRESS ON RS PANELS



LARGE GAP



DISTRESS ON RS PANELS

LARGE GAP

LARGE GAP MAY DUE TO EXCESSIVE SETTLEMENT

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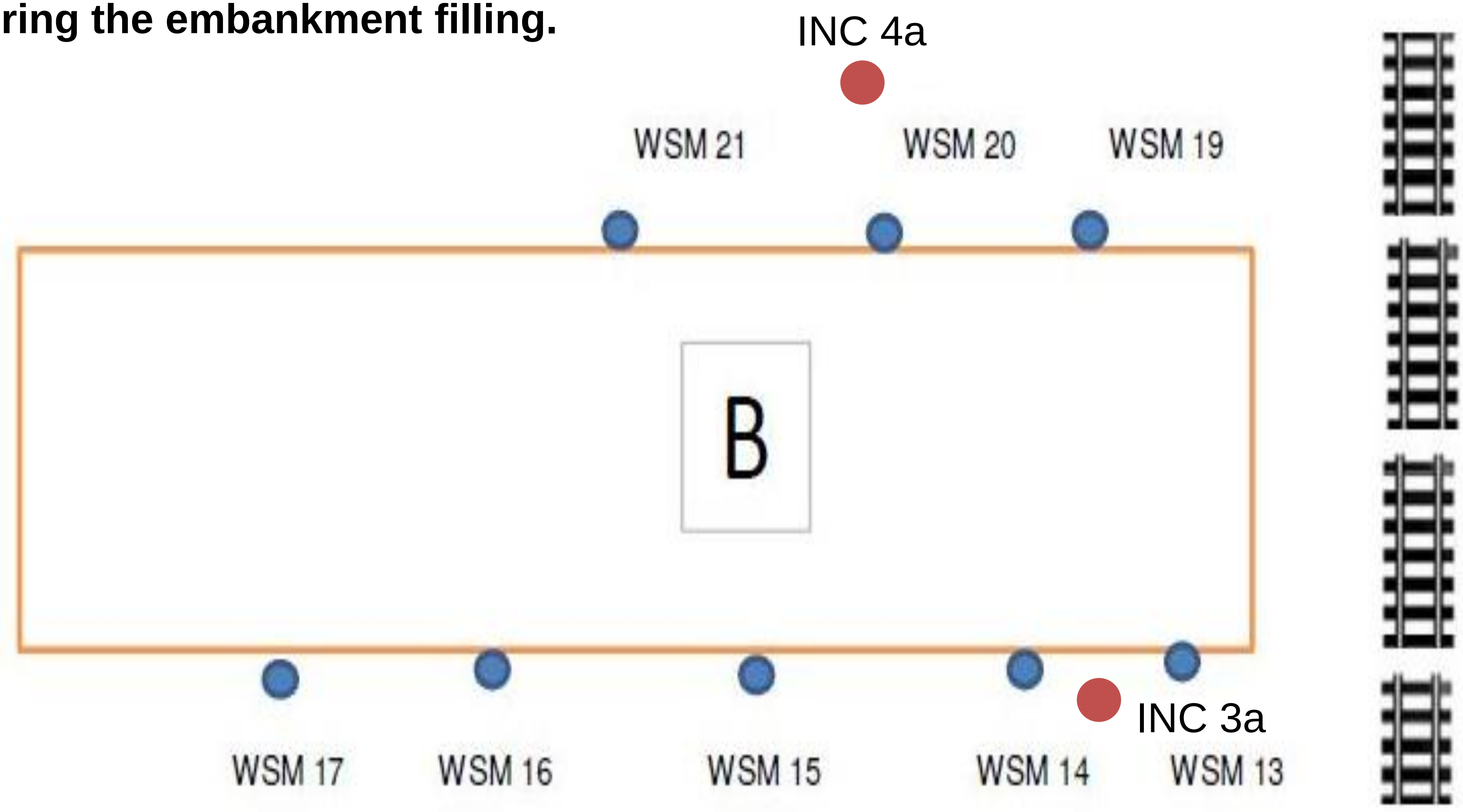
INSTRUMENTATION

Instrumentation only came in half way during the embankment filling.



WSM: Wall Settlement Markers
INC: Inclinometer

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BACK ANALYSIS

- 1-Dimensional Consolidation theory was used to back calculate the amount of settlement that has occurred before the installation of instrumentation.
- Analysis was carried out for embankment at three (3) locations of the settlement markers.

Status of Filling

WSM (SI)	ACTUAL FILL HEIGHT (m)	DESIGN FILL HEIGHT (m)	STONE COLUMN SPACING (m)	STONE COLUMN LENGTH (m)
13	6.596	9.942	1.5 x 1.5	15
20	5.570	8.421	1.65 x 1.75	15
21	3.569	7.141	1.9 x 1.9	15

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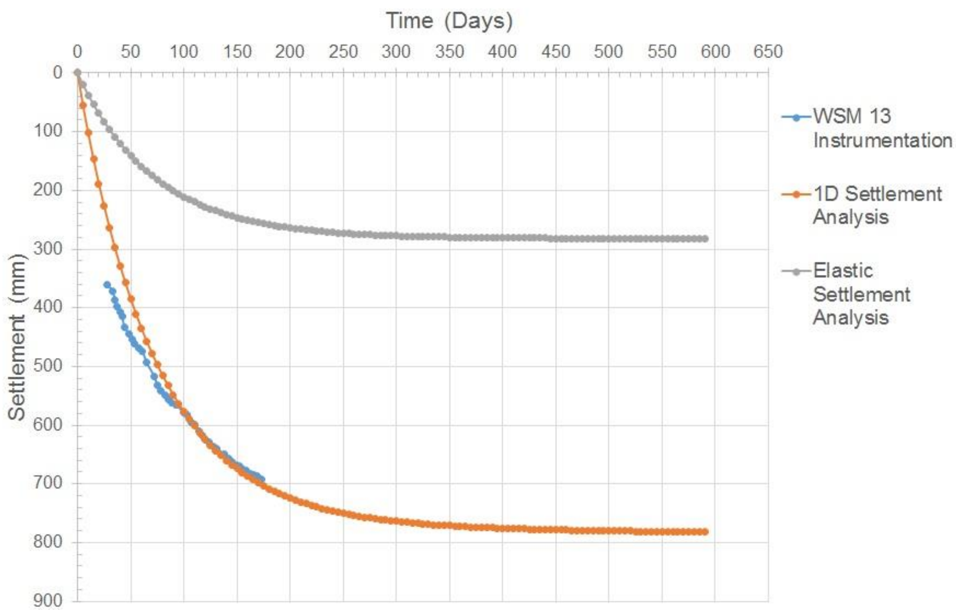
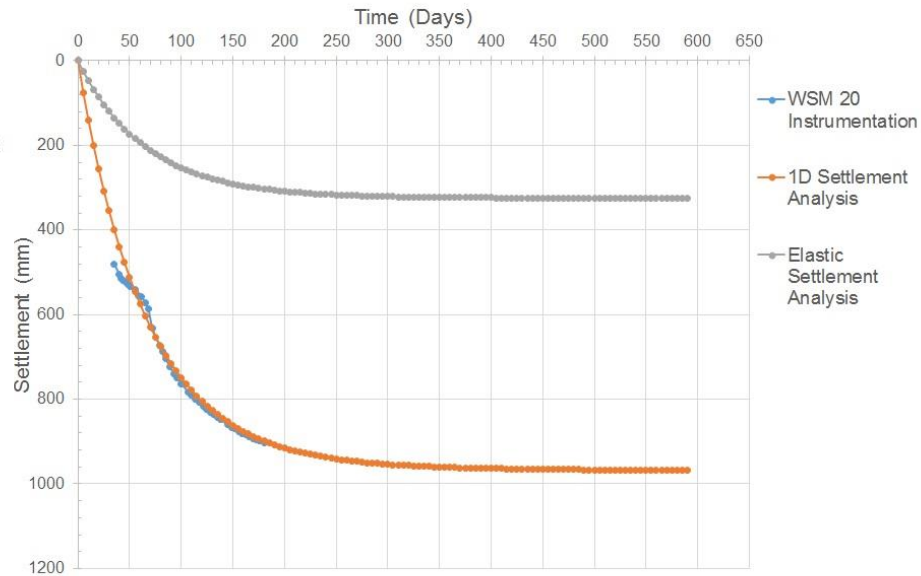
BACK ANALYSIS (CONT'D)

- The measured settlement profile from WSM is matched with the predicted settlement profile generated using 1D-conso theory.
- Comparison is also carried out with settlement curve generated using elastic theory.
- Asaoka's graphical prediction method was adopted to predict the final primary settlement excluding the unmeasured settlement to counter check the back analysis results.

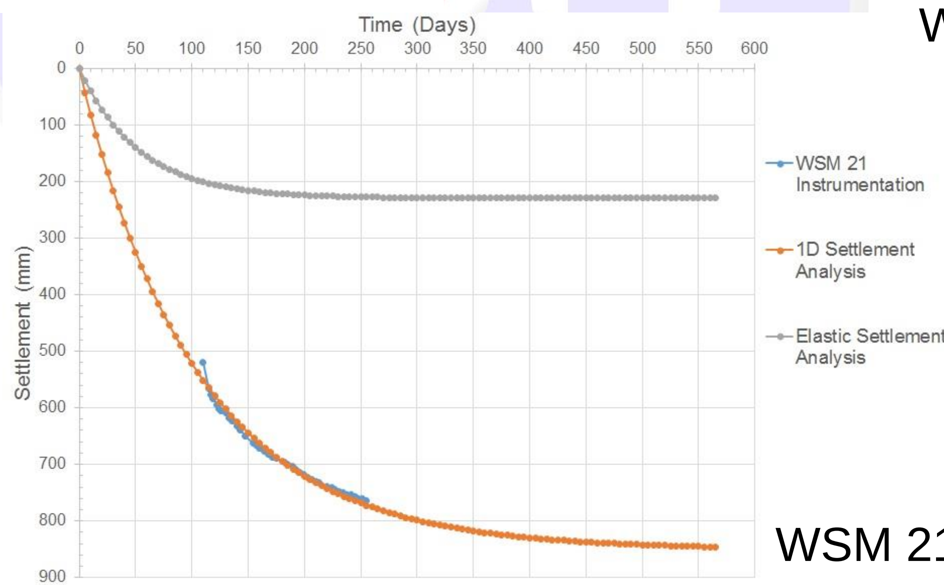
SETTLEMENT-TIME CURVES



WSM 13



WSM 20



WSM 21

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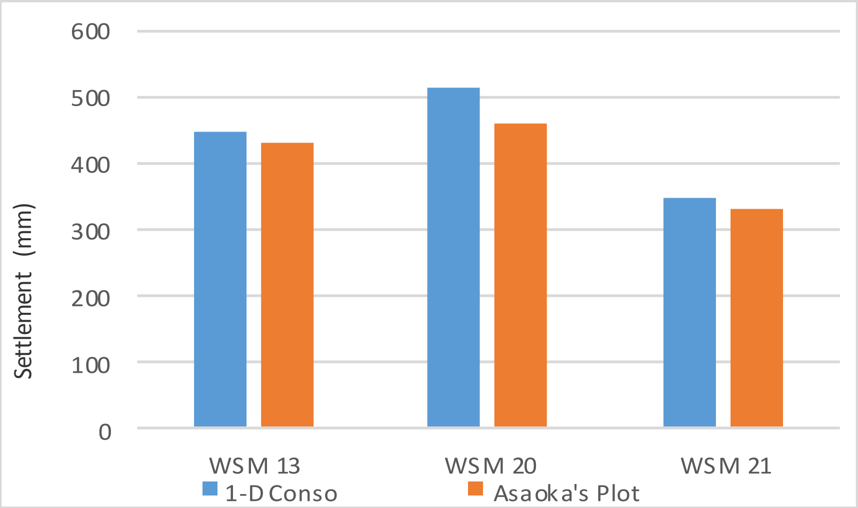
ANALYSIS OUTPUT

Location	Fill height used for analysis (m)	Type of Analysis/Measurements				
		1-D Consolidation Theory			Elastic Theory	Asaoka's Method
		Predicted Unmeasured Settlement (mm)	Predicted Settlement (mm) (after inst)	Total Settlement (mm)	Total Settlement (mm)	Predicted Settlement (mm) (after inst)
WSM 13	6.596	360	449	809	282	430
WSM 20	5.570	480	515	995	325	460
WSM 21	3.569	520	347	867	229	330

Comparison between predicted and measured settlement after the instrumentation installation



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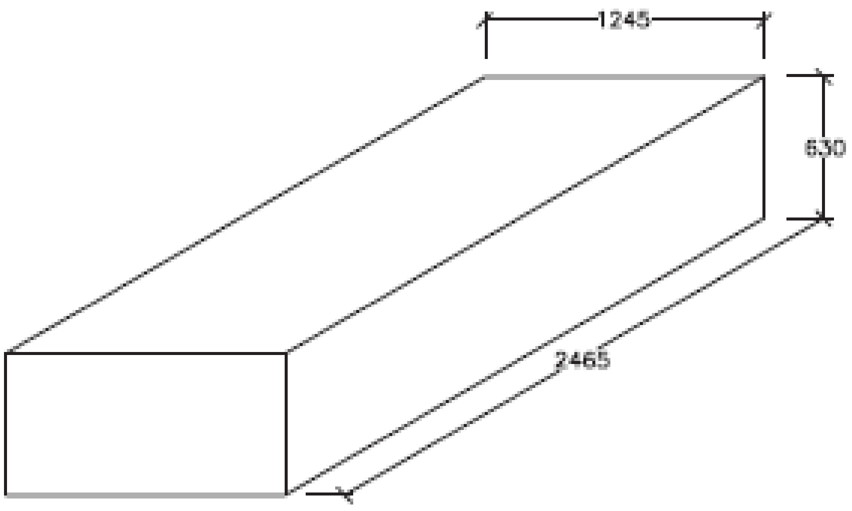
RECTIFICATION PROPOSAL

- Reduce embankment loading using Expanded Polystyrene (EPS).
- EPS is an innovative building material that consist of 98% air and 2% plastic which offer an exceptionally lightweight solution to many applications in construction.



STANDARD SIZE OF EPS BLOCK

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CONCEPT OF EPS PROPOSAL

1. LIGHT WEIGHT SOLUTION

2. REDUCTION OF STRESS AT FOUNDATION TO DESIRABLE LEVEL

3. MINIMISED RESIDUAL SETTLEMENT TO ACCEPTABLE LEVELS

THE ABOVE IS ACHIEVED BY REDUCING THE ORIGINAL FILL HEIGHTS TO PROVIDE REDUCTION IN STRESS LEVEL AND MINIMISE RESIDUAL SETTLEMENTS

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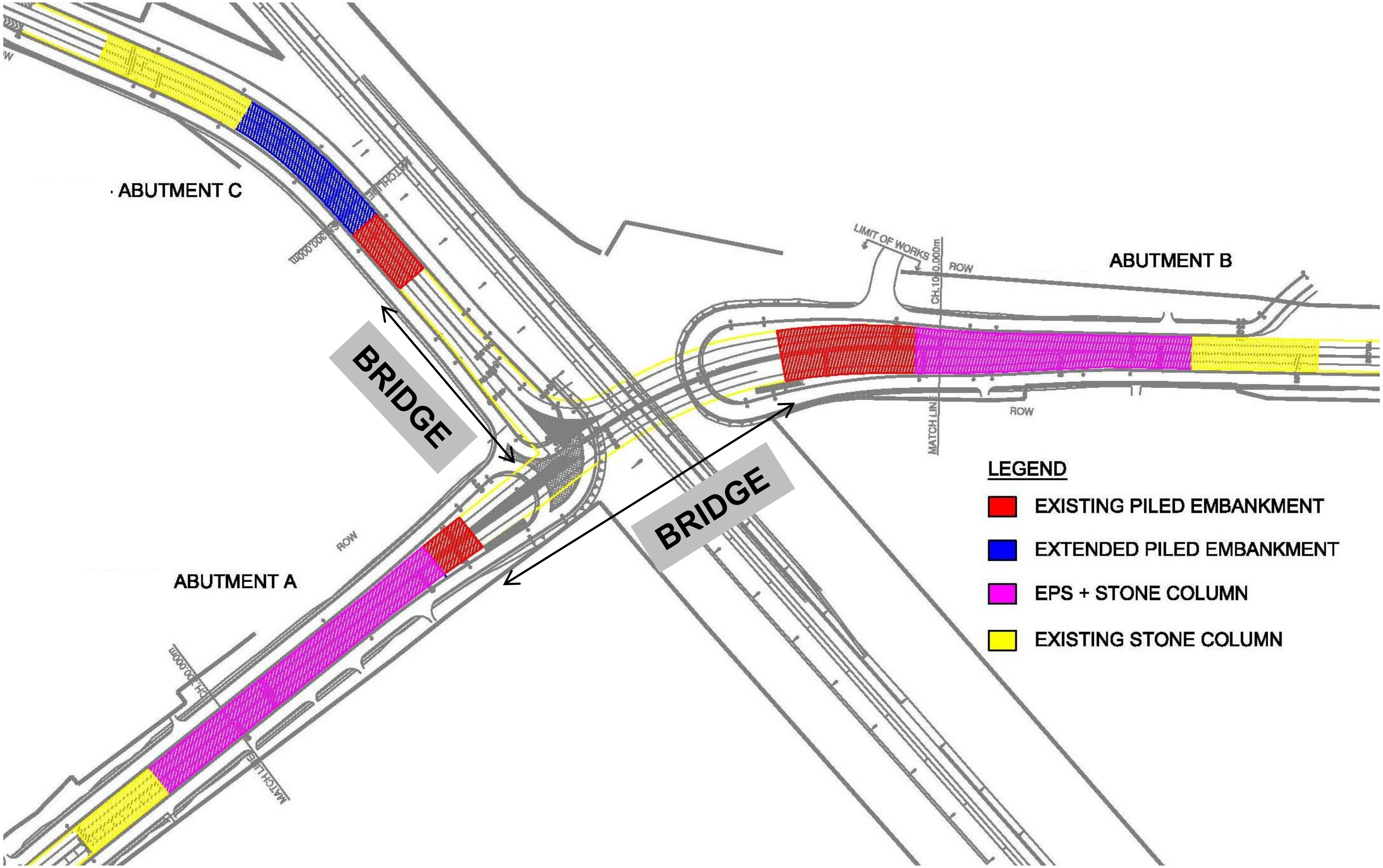
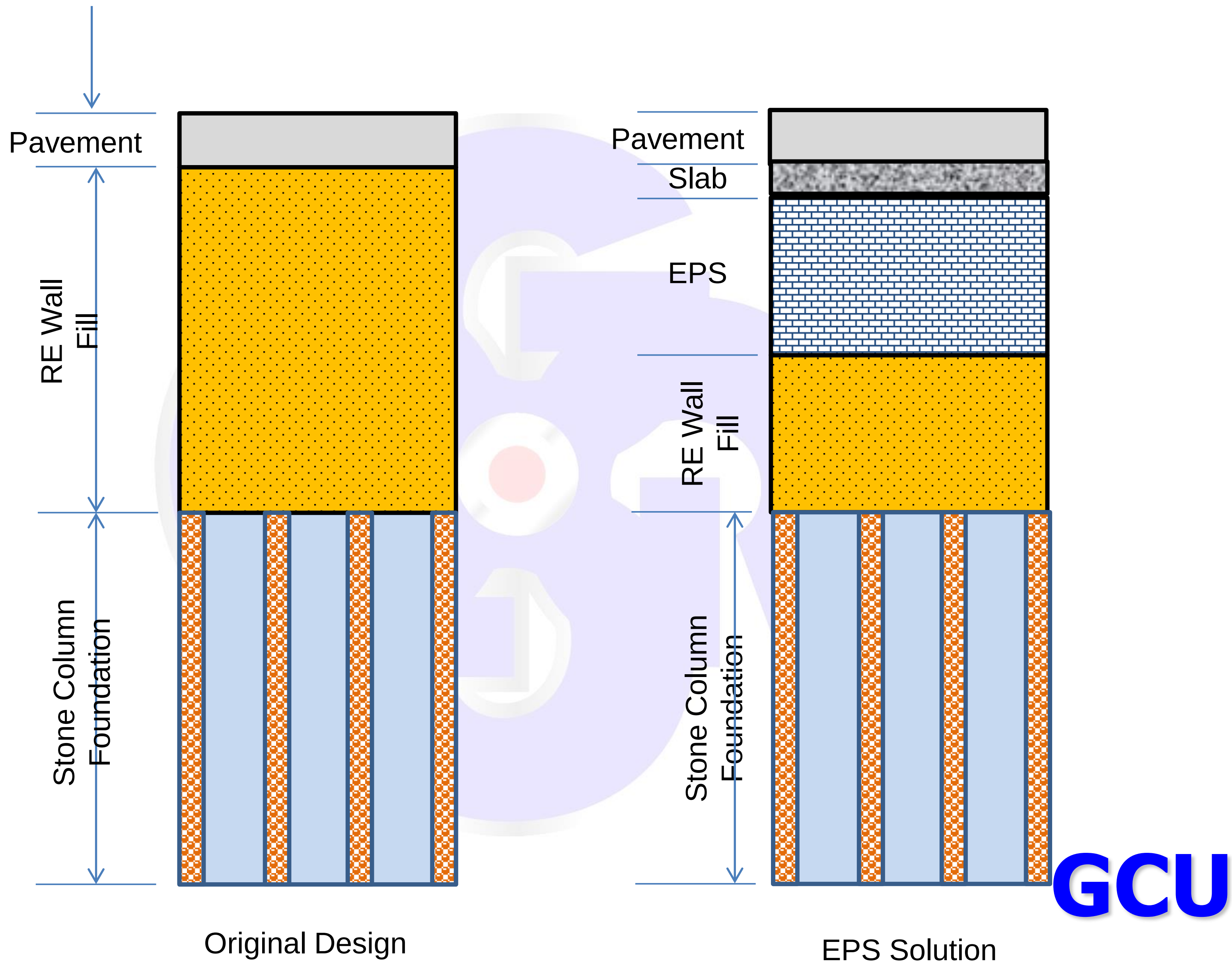
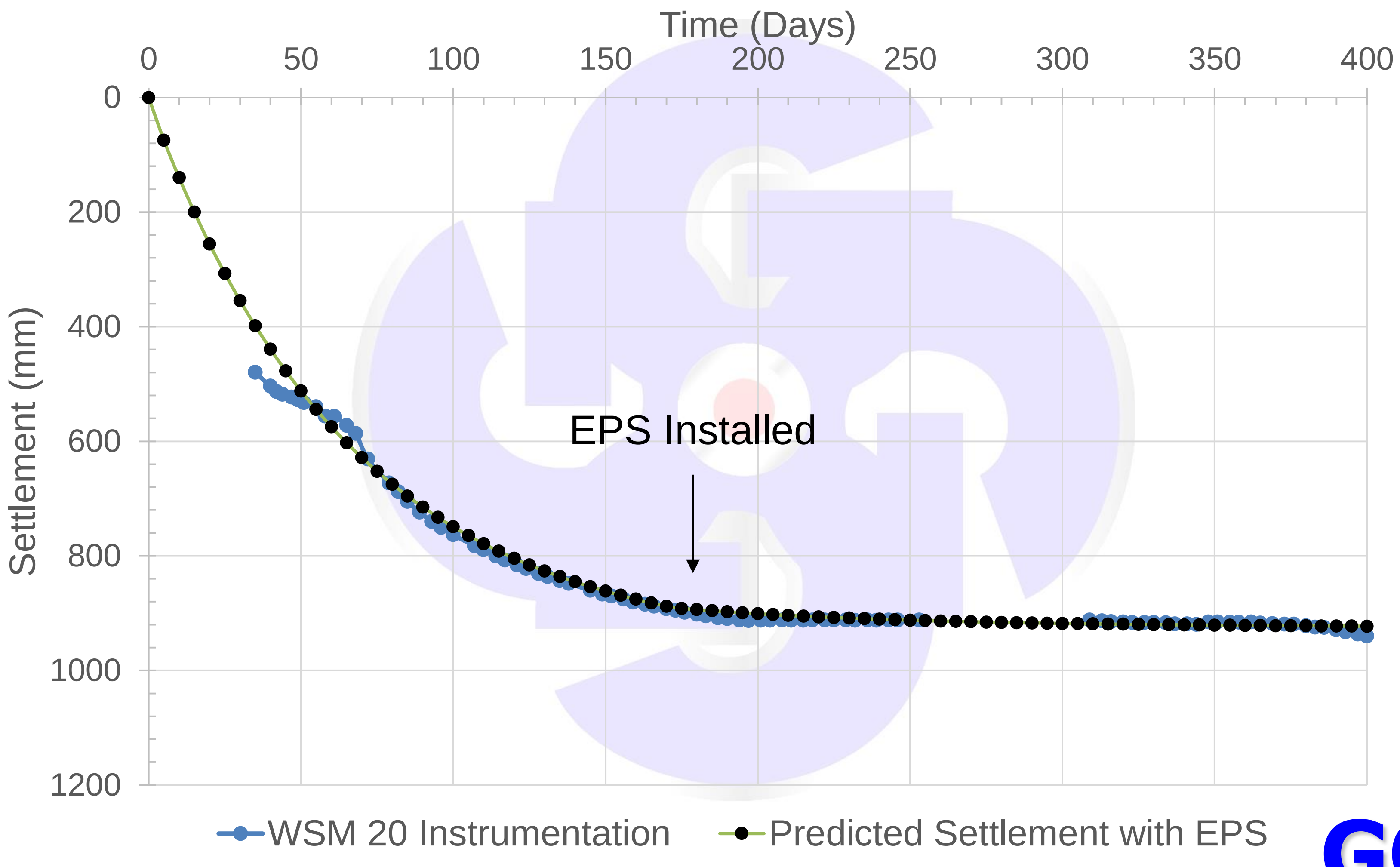
RECTIFICATION PROPOSAL



SCHEMATIC PRESENTATION OF ORIGINAL DESIGN AND EPS SOLUTION



TIME SETTLEMENT CURVE AFTER EPS INSTALLATION



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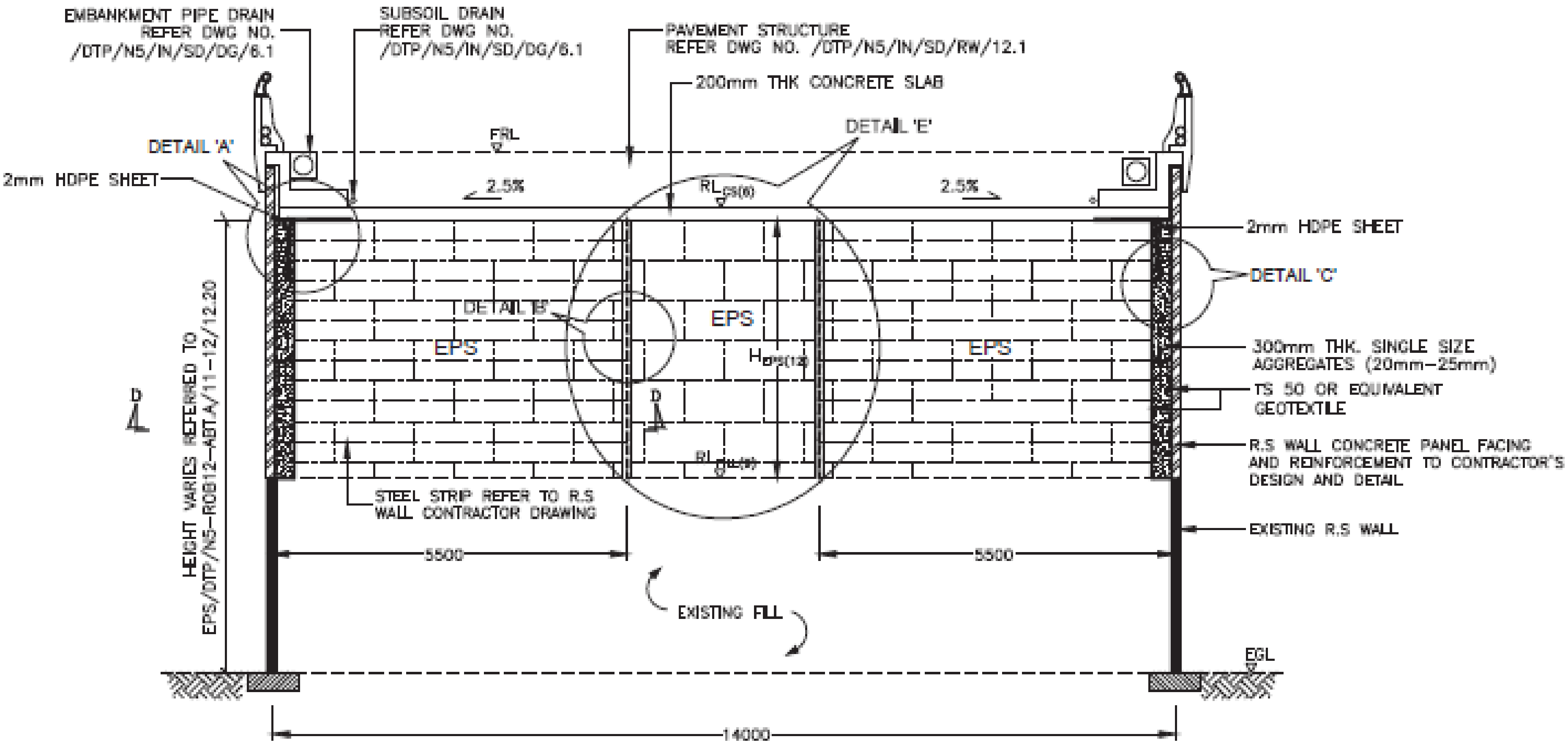
TYPICAL SECTION



TYPICAL SECTION

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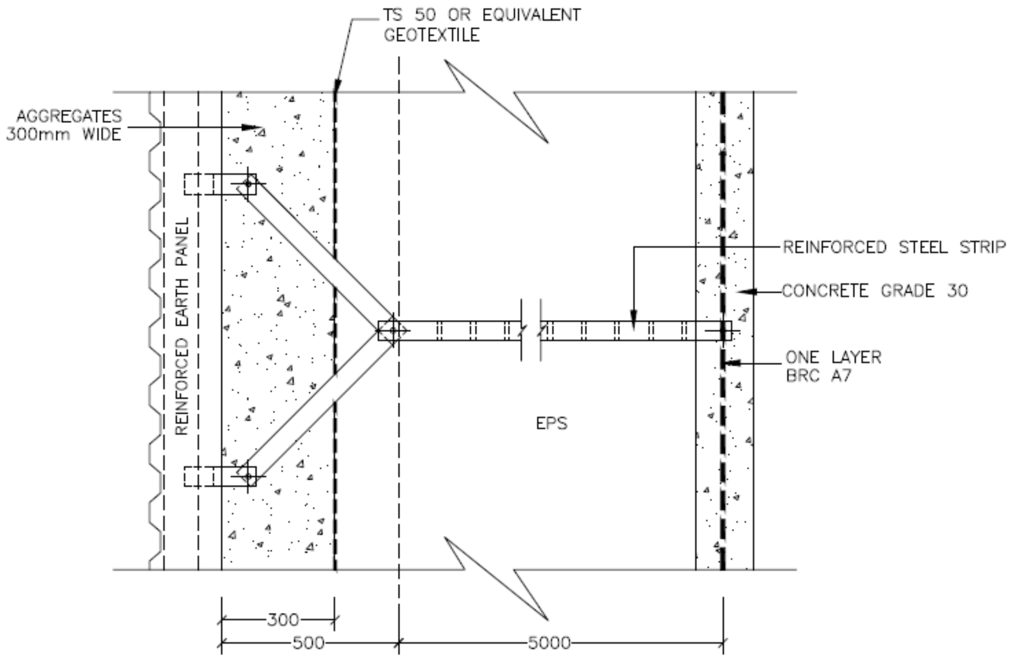
CONNECTION DETAILS



SECTION a-a

PLAN VIEW OF REINFORCED STEEL STRIPS

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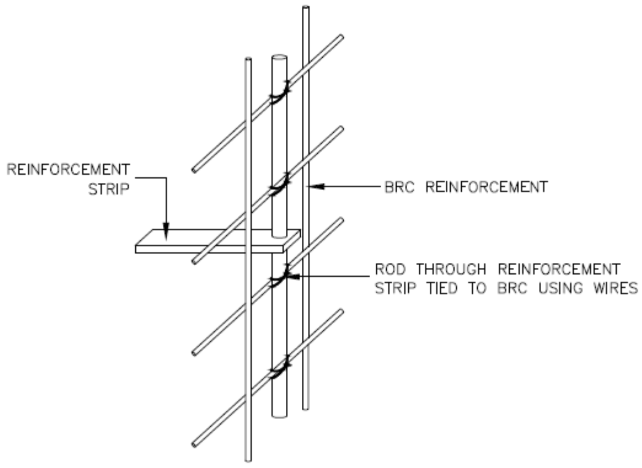

DETAIL 'B'

JOINT BETWEEN REINFORCING STRIP
AT MODIFIED ANCHOR BLOCK

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PICTURE OF EPS INSTALLATION (1)

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PICTURE OF EPS INSTALLATION (2)

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PICTURE OF EPS INSTALLATION (3)

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PICTURE OF EPS INSTALLATION (4)

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SUMMARY & LESSON LEARNED

- Back analysis is used:
  - to estimate past settlement before instrumentation
  - To estimate total settlement due to the embankment fill
  - To estimate settlement due to the replacement of soil with EPS

- Elastic theory has been compared and found to be inappropriate to be used for settlement estimation in soft ground.

- Prediction of settlement in soft ground condition is much more precise and accurate by incorporating 1-D consolidation theory in Priebe method of vibro stone column calculation.

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SUMMARY & LESSON LEARNED (CONT'D)

- EXPANDED POLYSTERENE (EPS – LIGHT WEIGHT SOLUTION) is proposed as light weight solution to overcome the distress in RS wall.
- Settlement of an embankment with RS wall shall be accessed and considered with the tolerable settlement of RS wall.

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THANK YOU

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Pilot tests on methods to form working platform on very soft clay

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Outline

1. Problem
2. Proposed method
3. Results and data analysis
4. Conclusions

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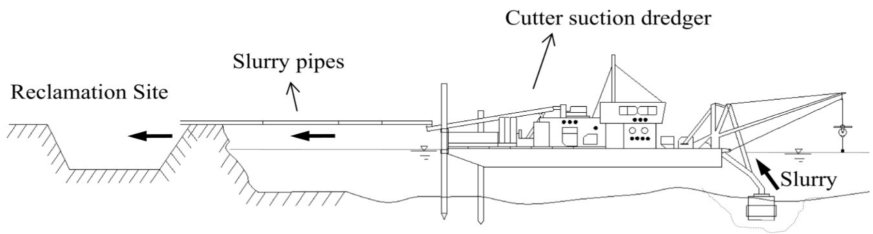
Problem in using dredged slurry for land reclamation



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Difficulties in the use of soft soil for land reclamation

- **Major difficulty:** The top surface is too soft for workers and machines to go on top to carry out soft improvement work.
- **Key technical challenge:** ***how to form a work platform??***
- **Method to use:** ***the one with the lowest unit cost!***

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Methods for creating a working platform

(after Chu et al. 2013)

#	Method	Description / Mechanisms	Advantages	Limitations
1	Sun drying	Reduce the water content of soil and form of a desiccation layer	Simple and economical	Very time consuming; Depth of improvement is not sufficient
2	Capping with sand or good earth	Place sand or good earth in thin layers	Relatively cheap	Slow and difficult to implement
3	Use of geotextile	Place a layer of geotextile to the top of soft soil before soil or fill is placed.	Relatively quick and reliable	Expensive & need special equipment
4	Lime or cement mixing	Use lime or cement mixing to strengthen a layer of soil at the top to form a working platform	Relatively reliable	Expensive & difficult to achieve uniformity
5	Dewatering	Use special drainage methods to dewater or consolidate the a layer of soil at the top to form a working platform	Relatively cheap	The method needs to be further developed or verified

TWO PILOT TESTS USING TWO METHODS

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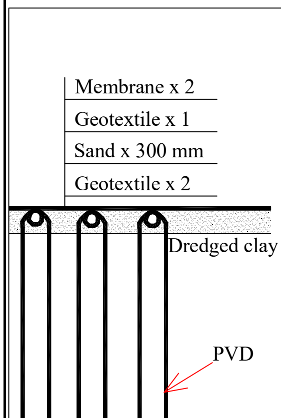
Method-1: Conventional Vacuum Consolidation with membranes (CVC)



(a) sketch of the method (b) installation short PVDs (c) laying sand blanket layer

Method of using vacuum preloading combined with short PVDs to form working platform

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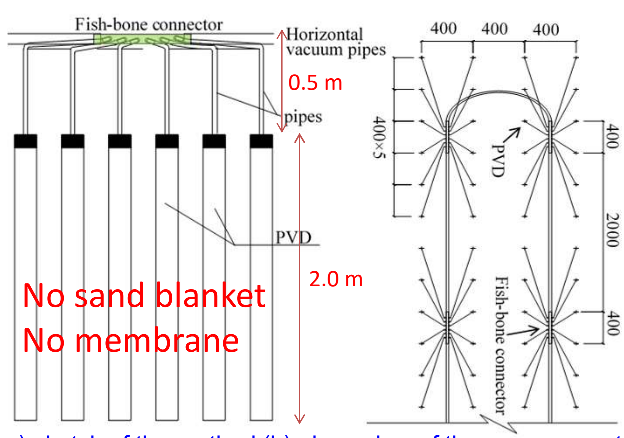
Method-2: Fish-bone Vacuum Consolidation without membrane (FVC)



a) sketch of the method (b) plane view of the arrangement (c) installation of the PVDs

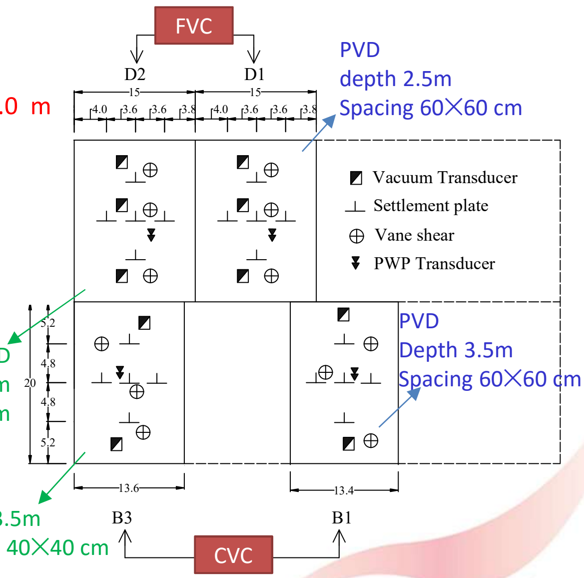
Method of using vacuum preloading combined with fish-bone connectors connected drainage pipe and short PVDs to form working platform

8

Pilot Tests and Results

a. Soil conditions

1. New dredged clay slurry 0.0-4.0 m
   Su: 0.7 – 2.7 kPa avg. 2.0 kPa
   Water content: 85%
   unit weight: 15.2 kN/m^3
   Liquid limit: 44
   Plastic limit: 21
2. Soft clay 4-19 m Su=8-30
3. Stiff silty clay > 19 mm Su=32



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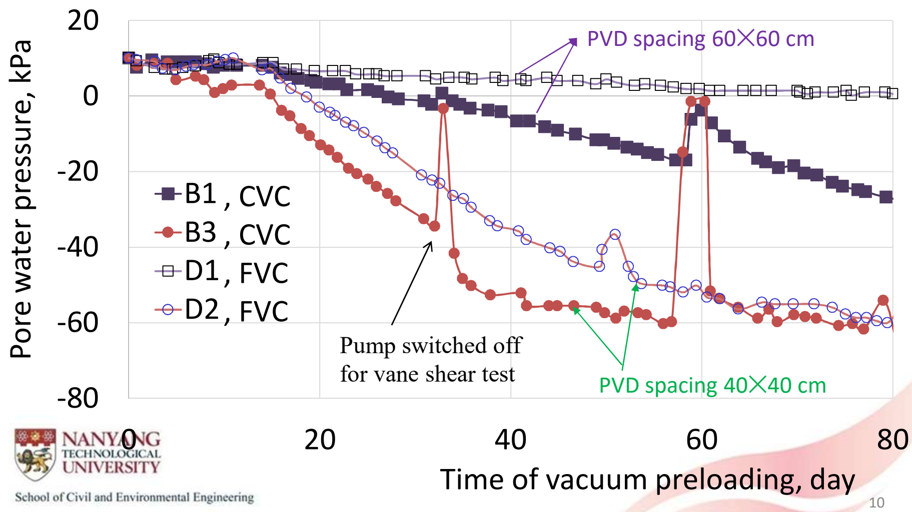
Results – Pore pressures

b. Pore water pressure in soil at 1.0-m-deep



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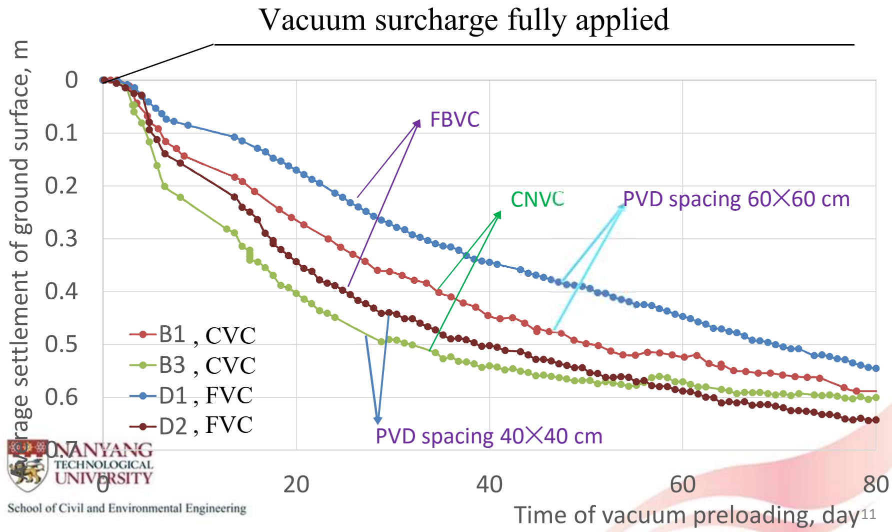
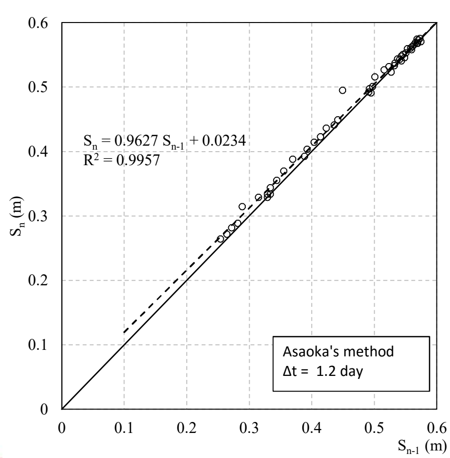
Results - Settlements

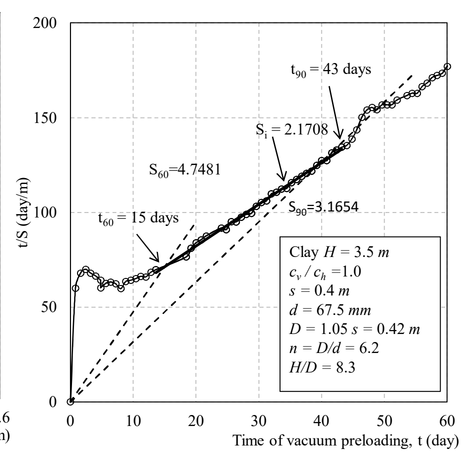
c. Ground surface settlement



Data analysis

d. Estimate of the ultimate settlement





Estimate the ultimate settlement of soil in subzone B3 using (a) Asaoka's method and (b) hyperbolic method

Data analysis

e. Average degree of consolidation

Subzone	Based on settlement data						Based on pwp data (Chu and Yan, 2015)		
	S_{PVD}	$S_{t=60}$	Asaoka, 1978		Hyperbolic Method		$\Delta u_{t=60}$	u_s	$U_{h=1.0\ m}$
	(mm)	(mm)	$S_{t=\infty}$ (mm)	U (%)	$S_{t=\infty}$ (mm)	U (%)	(kPa)	(kPa)	(%)
CVC B1 60cm	134	524.3	737.0	89.3	774.0	85.1	20.1	80	25.1
FVC D1 60cm	123.3	447.2	814.0	70.1	877.7	65.0	8.59	80	10.7
CVC B3 40cm	312	570.7	938.6	94.0	917.2	96.2	73.7	80	92.1
FVC D2 40cm	141.6	588.3	873.7	83.5	816.8	89.4	63.7	80	79.6

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Data analysis

f. Change in soil properties of soil at 1-m-deep

Subzone	Before vacuum preloading		After vacuum preloading					
			S_u (kPa)			w (%)		
	w (%)	S_u (kPa)	10 (days)	30 (days)	60 (days)	10 (days)	30 (days)	60 (days)
CVC B1 60cm	83.0	0	4.8	7.1	13.1	55.0	48.5	46.2
FVC D1 60cm	87.8	0	2.6	3.8	5.4	77.9	64.0	57.1
CVC B3 40cm	85.0	0	13.8	21.4	26.4	46.5	44.2	43.4
FVC D2 40cm	83.8	0	10.1	15.3	20.5	59.5	52.5	48.0

10 Times

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Cost Comparison

	Items	Unit price	Amount	Total (¥)
CNVC	PVD installation	2.5 ¥/m^2	7160 m^2	17900
	Geotextile layer × 2 layers	9.0 ¥/m^2	300 m^2	2700
	Horizontal drains	3.5 ¥/m	375 m	1312.5
	Sand blanket layer	120 ¥/m^3	90 m^3	10800
	Consolidate 60 days	55 ¥/m^2	300 m^2	16500
			Total (¥)	49212.5
			Unit Cost (¥/m^2)	164.0
	Items	Unit price	Amount	Total (¥)
FBVC	PVD installation	2.5 ¥/m	537[illegible] m	13,425.8
	Fish-bone connector	12 ¥/pcs	16[illegible]5 pcs	1980
	Connectors	2 ¥/pcs	1989 pcs	3978
	Drainage pipes	3.0 ¥/m^3	[illegible] m^3	5967
	Sealing pipes	3.5 ¥/m	[illegible]	1050
	Consolidate 90 days	55 ¥/m^2	30[illegible] m^2	16500
			Total (¥)	42,900.8
			Unit Cost (¥/m[illegible])	143.0

-14%

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Conclusions

1. Two methods CVC and FVC to use vacuum preloading to form a working platform were evaluated using pilot tests
2. Both methods are effective and comparable.
3. After 60 days of vacuum preloading, the undrained shear strength of the soil has increased from 0 to are over 20 kPa and the water content reduced from 84% to 48%.
4. The FVC method is cheaper (14%) and faster than CVC method.

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Thank you!

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Ground Improvement for Tanks

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19th Southeast Asian Geotechnical Conference & 2nd AGSSEA Conference (19SEAGC & 2AGSSEA)
Kuala Lumpur 31 May – 3 June 2016

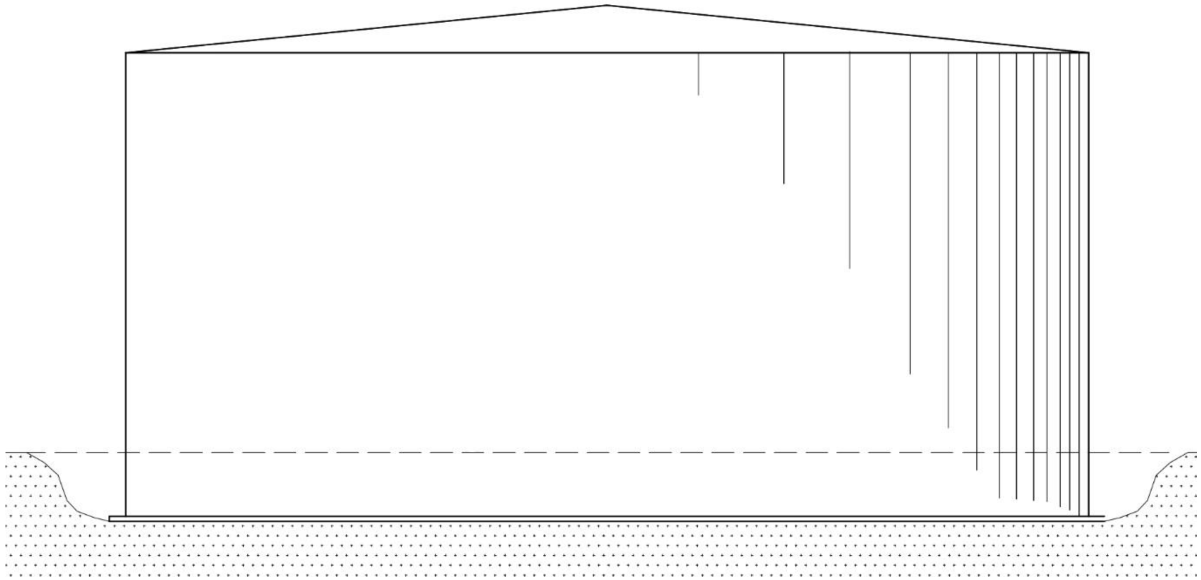
The obvious, which is occasionally forgotten

- **Tank deformations are acceptable as long as**
  - Deformations do not lead to impairment of serviceability
  - Deformations do not create stresses that exceed allowable limits

- **Tanks are not framed structures, and implementation of building codes to tanks is insubstantial**
- **Tanks must be designed based on standards for tanks**

Uniform Settlement

- **Most standards do not appear to be overly sensitive to this type of tank settlement, but draw attention to its effects:**
  - Piping
    - Flexible connections
    - Periodically repositioning the pipe supports

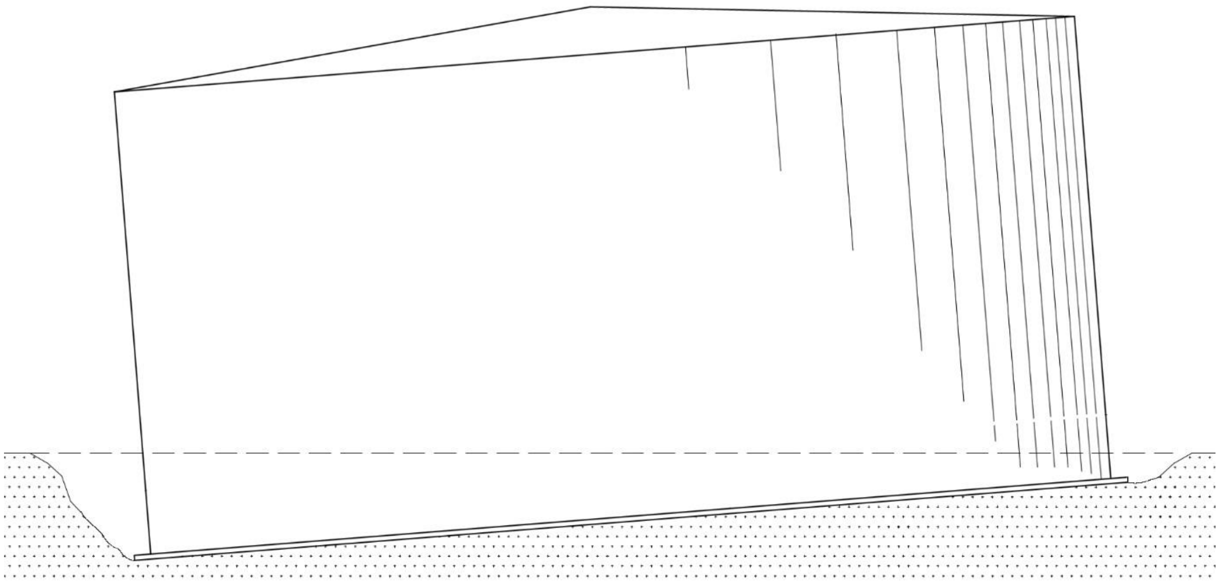


Uniform Settlement

- **ACI 376 (concrete structures for the containment of refrigerated liquefied gases)**
  - Does not specify limits provided that other provisions of the standard are met, and the connecting piping system accommodates the settlement.
- **API 620 (Large, welded, low-pressure storage tanks)**
  - Does not specify a limit on uniform settlement, but notes that uniformity of support and avoidance of excessive settlement are much more important for tanks that have formed bottom plates than they are in the case of flat-bottom, vertical storage tanks.
- **API 625 (Refrigerated liquefied gas storage tanks systems)**
  - Notes that the amount of acceptable uniform settlement is dependent upon piping and structural connections between the tank system and adjacent structures.
- **API 650 (Welded tanks)**
  - Does not specify any limits, but specifies that the estimated settlement should be within the acceptable tolerances for the tank shell and bottom.
  - Notes that total settlement must not strain connecting piping or produce gauging inaccuracies.
  - States that that settlement should not continue to a point at which the tank bottom is below the surrounding ground surface. If a large settlement is expected, the tank bottom elevation should be raised so that the final elevation above grade will be a minimum of 150 mm after settlement.
- **API 653 (inspection, repair, alteration, and reconstruction of tanks)**
  - Does not stipulate any limit for total settlements, but notes that for existing tanks with history of successful service, it may be possible to accept greater settlement and distortion of the foundation from a true plane than new tank construction standards allow.

Rigid Body Tilting (Planar Tilting)

- **Reduces freeboard**
- **Alters the shape of the fluid surface, and places additional stresses in the shell**



Rigid Body Tilting (Planar Tilting, Global Tilting)

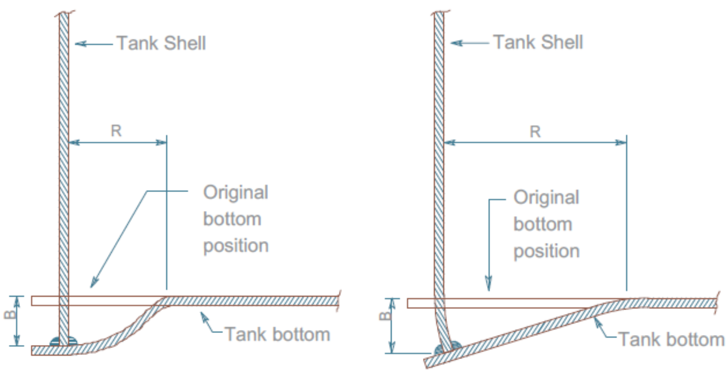
- **ACI 376**
  - Planar differential settlement ≤ 1/500
- **API 625**
  - Allows variations from the settlement limits that it specifies provided that they are accounted for in the design of the tank system and interconnecting components.
  - Comments that while large tanks may be able to accommodate significant tilting without damage, other components usually require lower value of tilt.

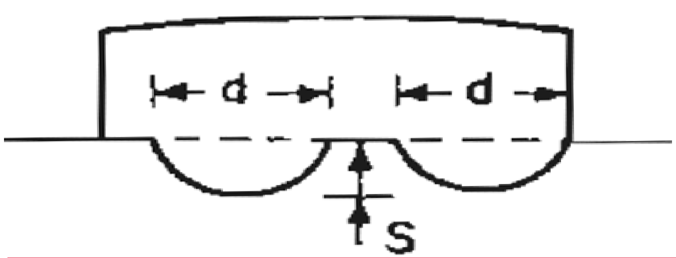
$$GT < 25.4\alpha\frac{D}{H}$$

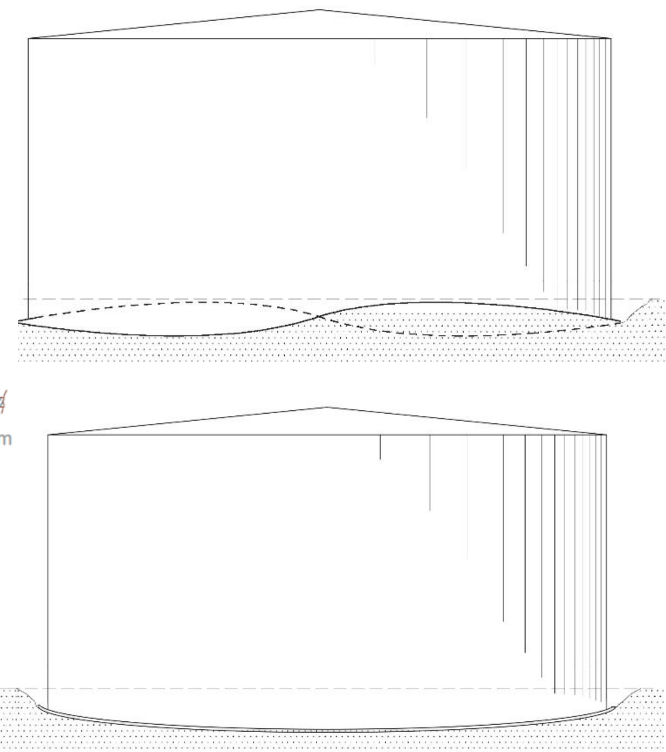
- GT= global tilt settlement, in mm
- α= 5, but often restricted to lesser values
- D= tank diameter
- H= tank height

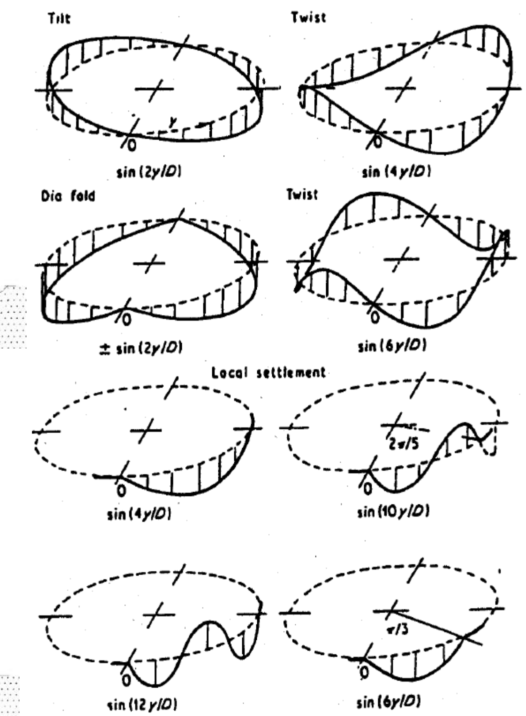
Out of Plane Settlement

- **Radial distortion or overstressing of the shell:**
  - Can result in the malfunction of a floating roof.
  - Can rupture the shell.
- **Overstressing the plate & its welds can rupture the shell-bottom plate connection.**









Out of Plane (Differential) Settlement

- **API 625**
  - DS= edge to centre of the tank settlement
  - TS= around the periphery of the tank differential settlement
  - R= tank radius

$$DS < R/240$$

$$TS < 1/1000$$

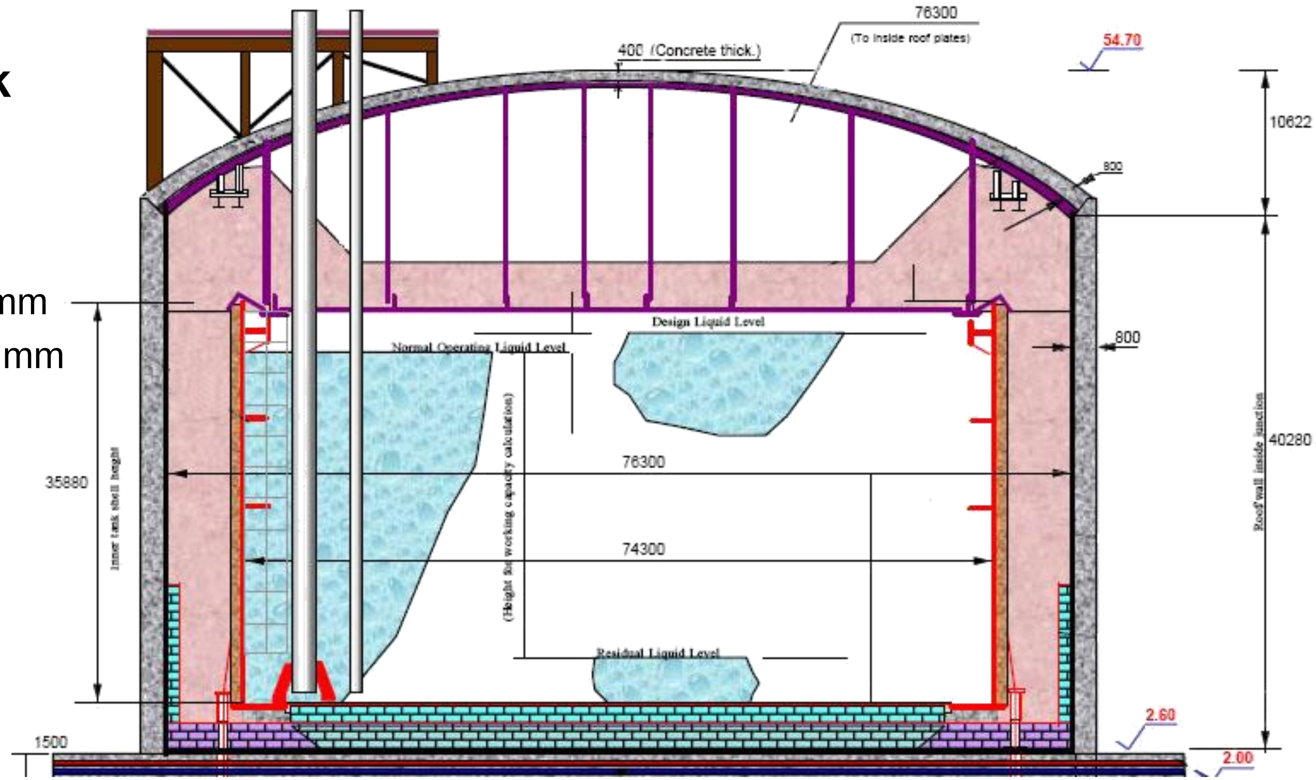
- **API 653**
  - S= deflection
  - L= arc length between two points
  - Y= yield strength
  - E= Young's modulus
  - B_B= maximum height of bulge or depth of local depression, in mm
  - R= radius of inscribed circle in bulged area or local depression, in m

$$|S| \leq \frac{11L^2Y}{2EH}$$

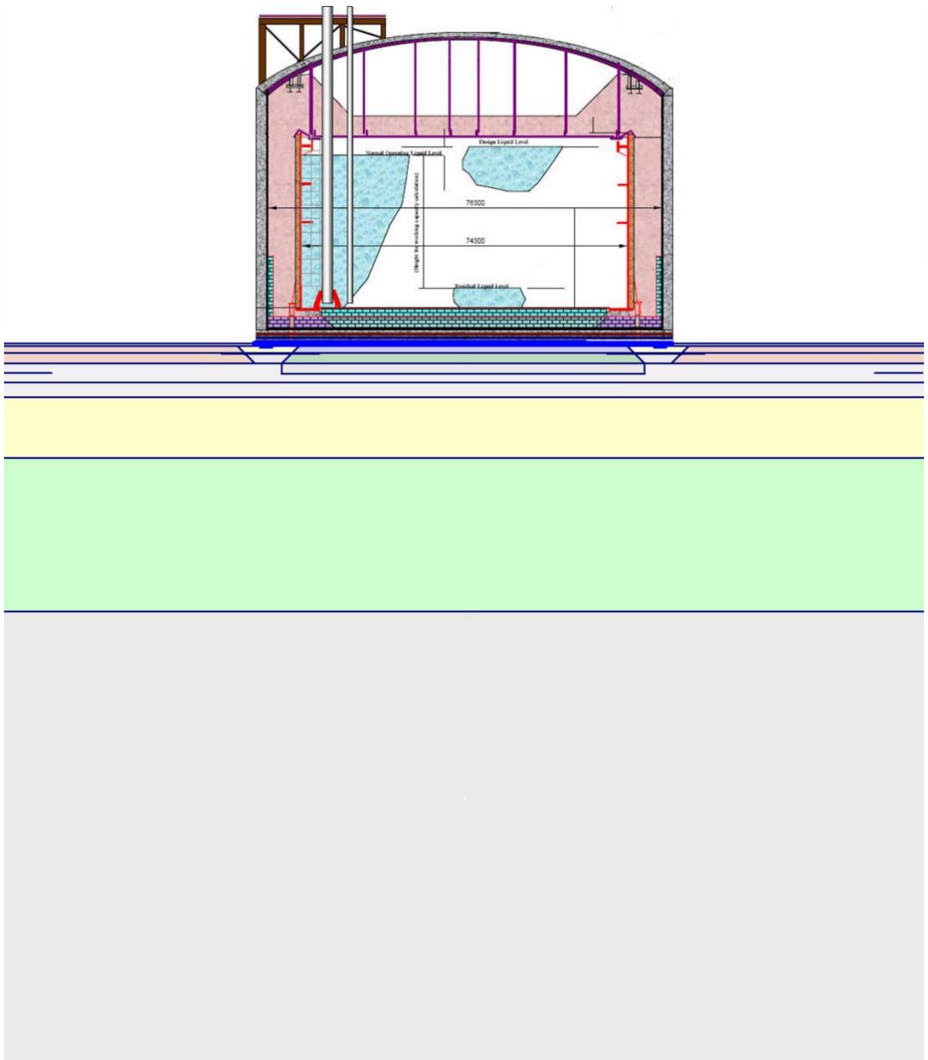
$$B_B \leq 30.8R$$

Case History: Rasgas LNG Tank T-6, Ras Laffan Qatar

- **Tank Volume: 140,000 m^3**
- **Internal Steel Tank**
  - Internal diameter= 74.3 m
  - Height= 35 m
  - LNG height= 34 m
- **External Concrete Tank**
  - External diameter= 76.3 m
  - Height= 50$^+$ m
  - Shell thickness= 800 mm
  - Roof thickness= 400-800 mm
  - Slab thickness= 500-1000 mm



Ground Conditions

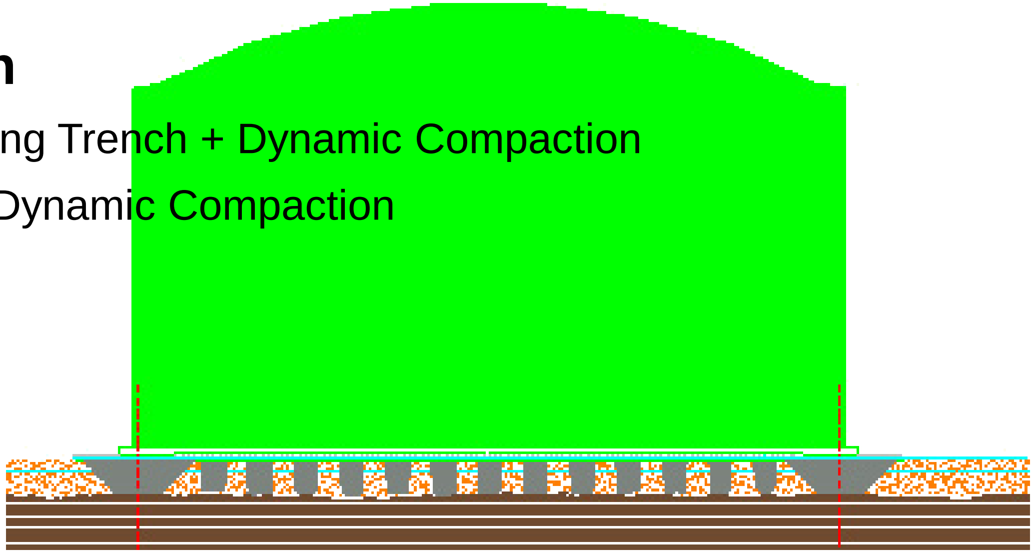


No	Layer	Thickness (m)
1	Loose sand above GWL	1.5
2	Loose soil below GWL	2
3	Weathered limestone	3
4	Upper limestone	3
5	Lower limestone	12
6	Calcareous siltstone	30
7	Calcareous sandstone	90

	Layer	1	2	3	4	5	6	7
Elev	Top	+1.5	0.0	-2.0	-5.7	-8.5	-20.5	-50.5
	Bottom	0.0	-2.0	-5.7	-8.5	-20.5	-50.5	-140
γ_{sat}	[kN/m³]	14	14	20	25	25	17.5	21
E_{ref}	[MN/m²]	20	10	75	1500	3000	300	600
ν	[-]	0.33	0.33	0.3	0.3	0.3	0.2	0.3
c	[kN/m²]	0	0	0	0	0	0	0
φ	[°]	32	32	40	NA	NA	NA	NA

Design Criteria & Ground Improvement Solution

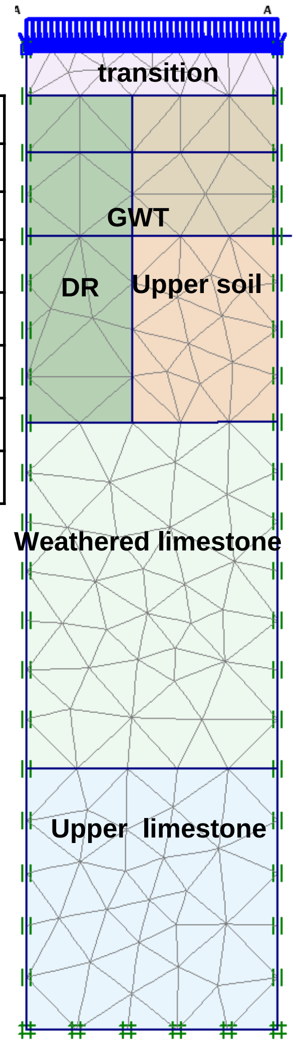
- **Design & Acceptance Criteria**
  - Differential Settlement
    - 1/700
      - Within the ring of the wall & the supported roof and before concreting the joint in the mat.
      - Within the general tank area after concreting the joint and during the hydrotest.
  - Total settlement:
    - Edge of the tank's mat: 80 mm
    - Center of the tank: 137 mm
- **Ground Improvement Solution**
  - Under the Concrete Shell: Shear Ring Trench + Dynamic Compaction
  - Inside tank: Stone filled columns + Dynamic Compaction

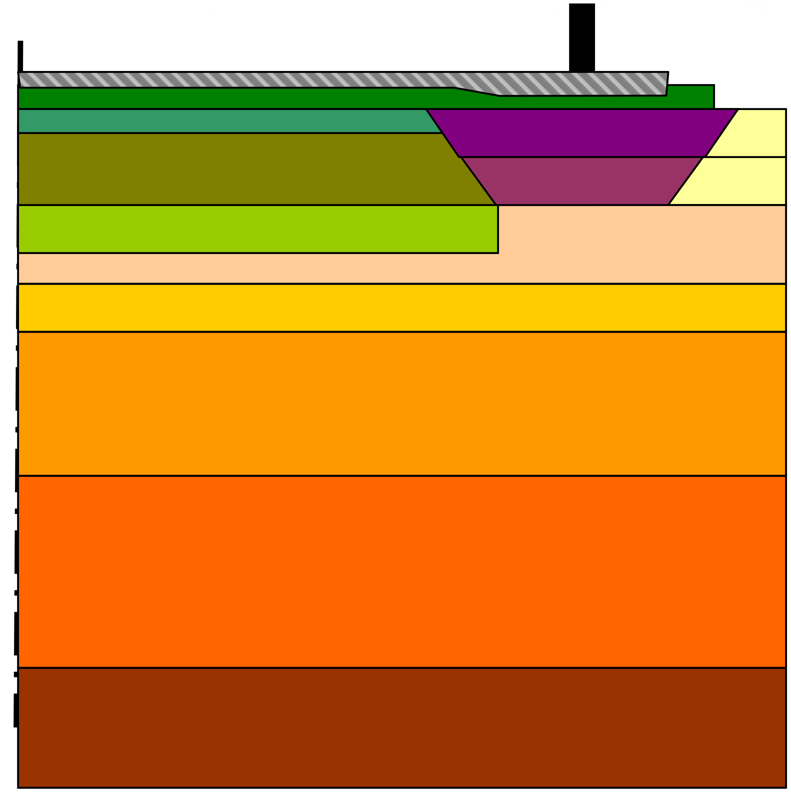


Design

	Layer	1	2	3	4	5	6
Elev	Top	+2.25	+1.5	-0.6	-3.5	0	-1.5
	Bottom	+1.5	0.0	-3.5	-5.5	-1.5	-3.5
γ_{sat}	[kN/m³]	20	15	15	25	20	20
E_{ref}	[MN/m²]	80	45.2	45.2	45.2	59.43	44.57
ν	[-]	0.3	0.3	0.3	0.3	0.3	0.3
c	[kN/m²]	0	0	0	0	0	0
φ	[°]	36	33	33	40	42	42

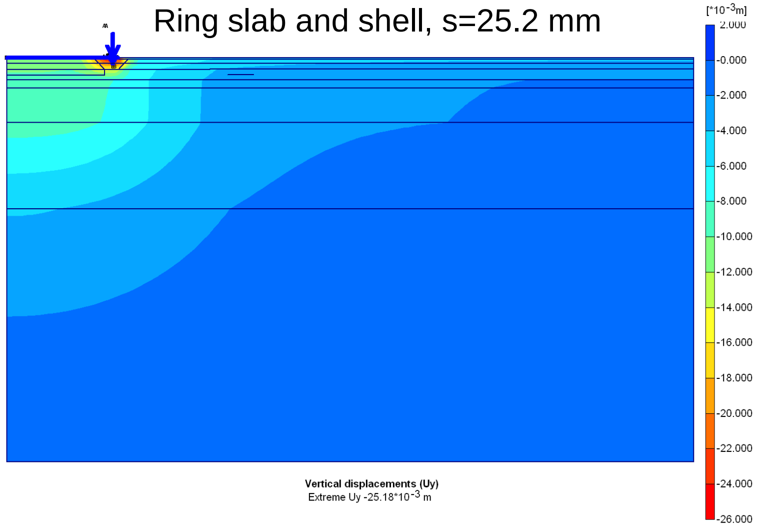
1 transition layer
2 DR1 layer in tank
3 DR2 layer in tank
4 DR3 layer in tank
5 DC layer above ground water level in trench
6 DC layer below ground water level in trench



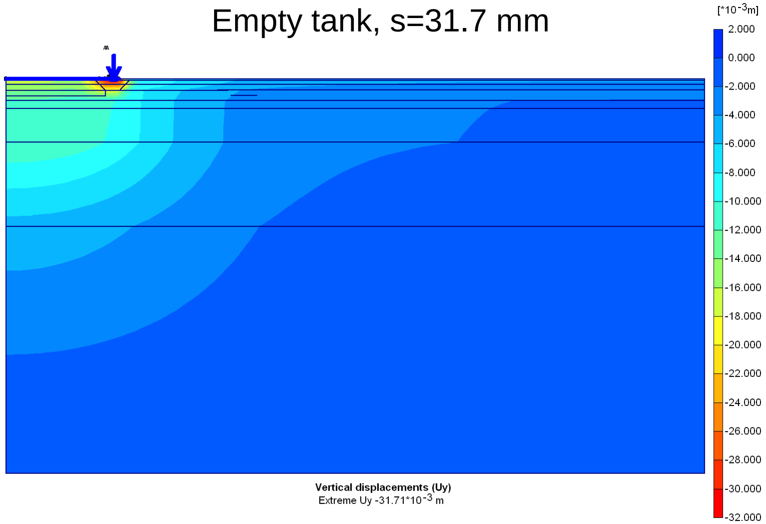


Design & Numerical Analysis

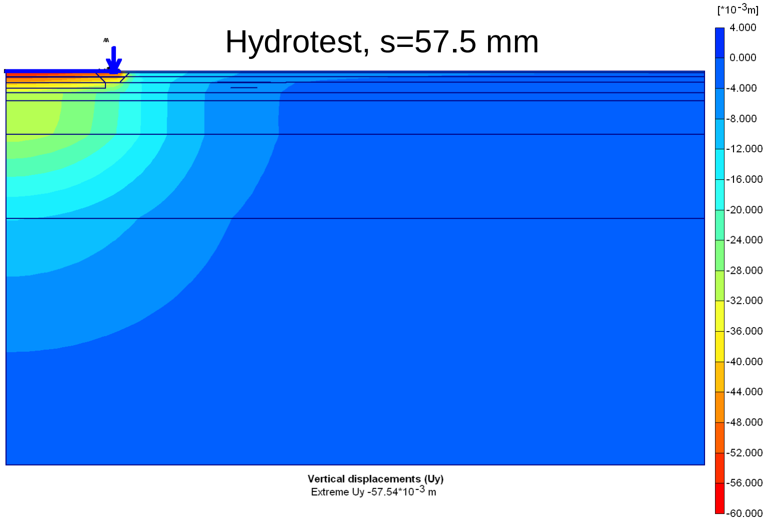
Ring slab and shell, s=25.2 mm



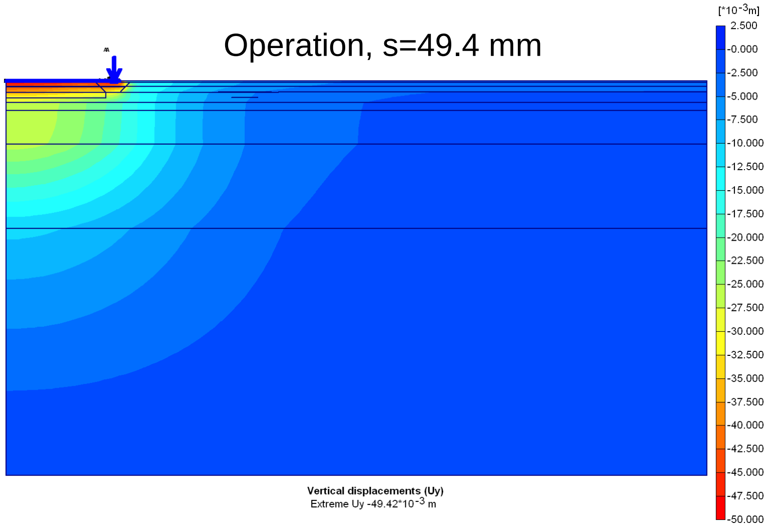
Empty tank, s=31.7 mm



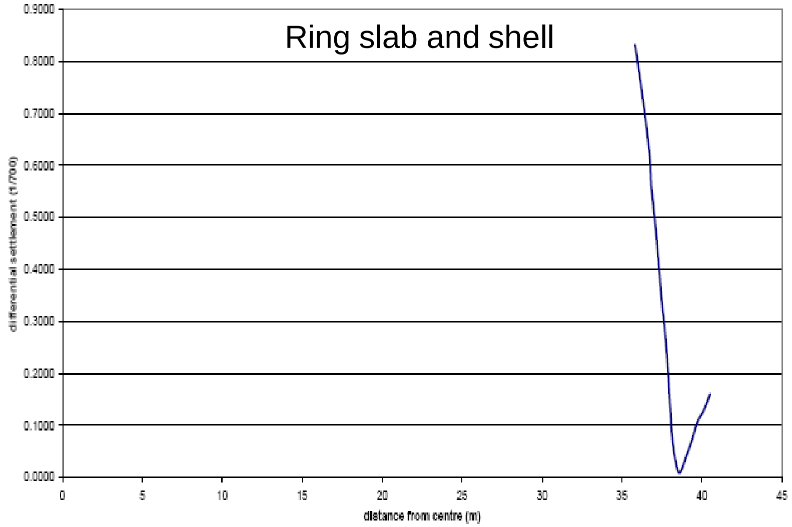
Hydrotest, s=57.5 mm

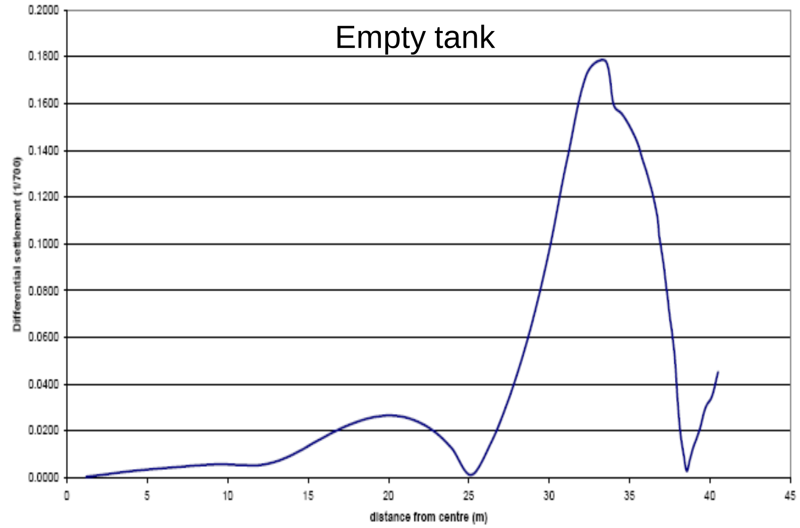


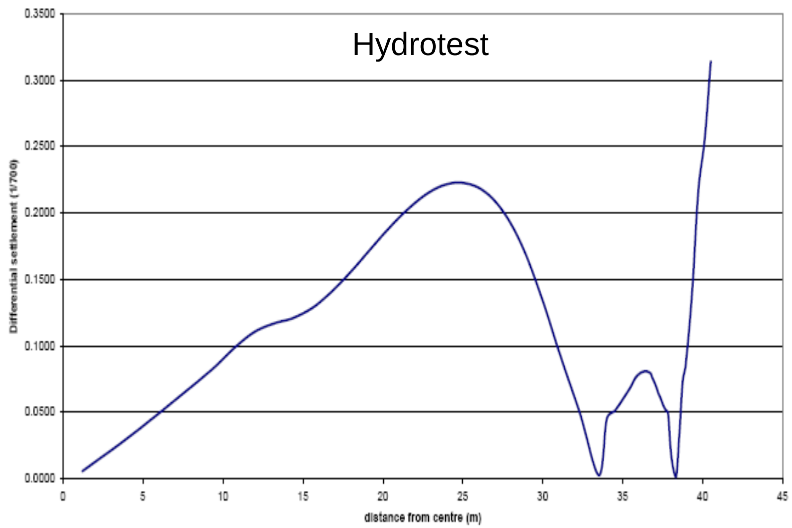
Operation, s=49.4 mm

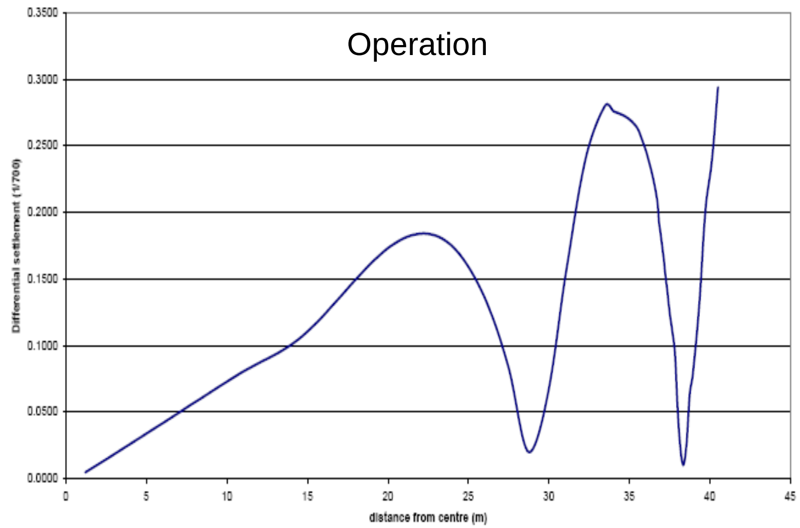


Design & Numerical Analysis





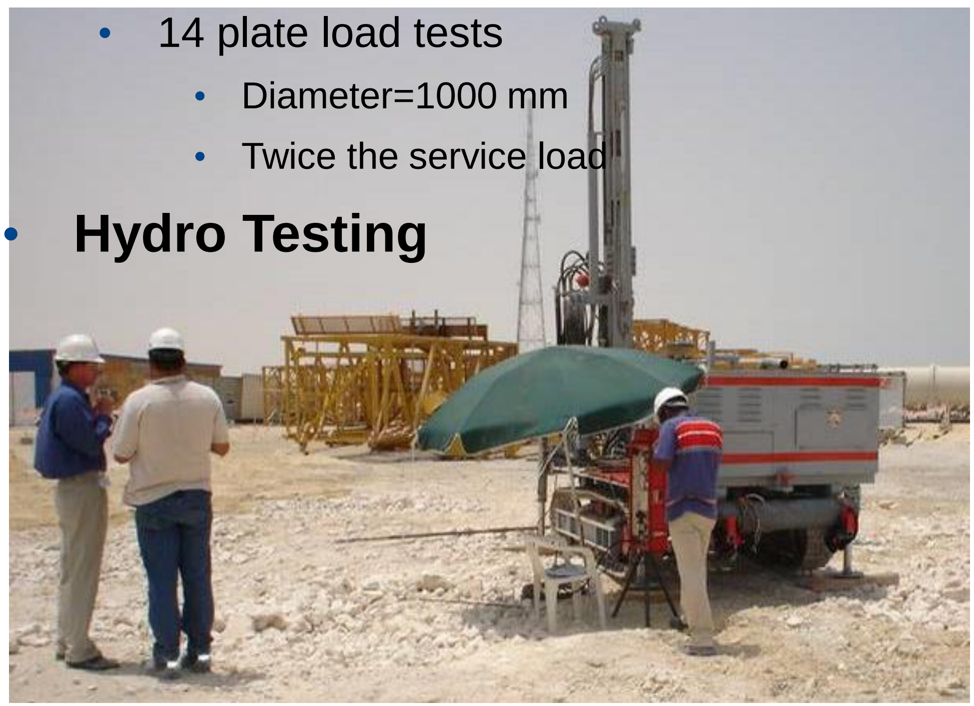


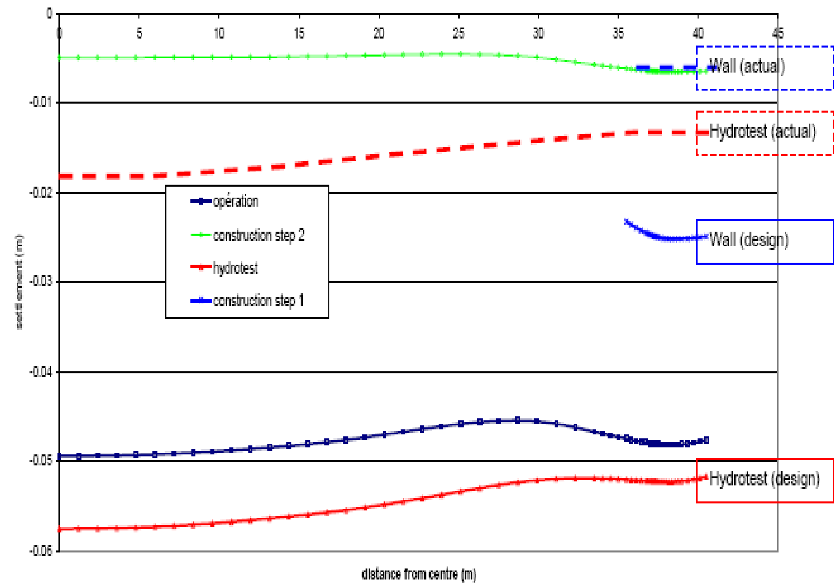


Dynamic Compaction & Dynamic Replacement

Testing & Quality Control

- **Geotechnical**
  - 25 Menard Pressuremeter Tests
    - $E_{y\ average}$
      - DR columns= 75 MPa
      - Trench= 70 MPa
      - DC=37 MPa
  - 14 plate load tests
    - Diameter=1000 mm
    - Twice the service load
- **Hydro Testing**





Solution	Load case	$U_{Ycentre}$ (cm)	U_{Yedge} (cm)	$U_{Ycentre}$ (cm)	U_{Yedge} (cm)	Maximum Differential settlement
		DESIGN		ACTUAL		DESIGN
DR + DC	Step 1 (Ring beam+Wall)	-	2.143	-	0.478	0.69/700
	Step 2(Roof)	0.666	0.835		0.617	0.25/700
	HydroTest	5.222	4.635	1.800	0.900	0.32/700
	Operation	4.481	4.286			0.30/700

Case History: Oil Tanks in Louisiana, USA

- **Five oil tanks**
  - Tank material: Steel
  - Diameters
    - Three tanks: 39.6 m
    - Two tanks: 45.7 m
  - Height = 12.8 m
  - Maximum tank load: 130 kPa
  - Additional platform fill load: 16 kPa

Ground Conditions

- **Fill: to depths of 0.15 to 1.2 m**
- **Soft to medium stiff silty clays with some trace of organic matter and localised sand pockets: to depths of 4 to 6 m**
- **very soft clay with silt and sand: to depths of 20 to 24 m.**
- **Thin sand layer was also identified at an approximate depth of 21 m.**
- **Medium stiff to stiff clay with fine sand pockets and shell fragments: to depths of up to 32 m.**
- **Stiff to very stiff silty to sandy clays to depths of about 34 m**
- **Very dense silty sands**

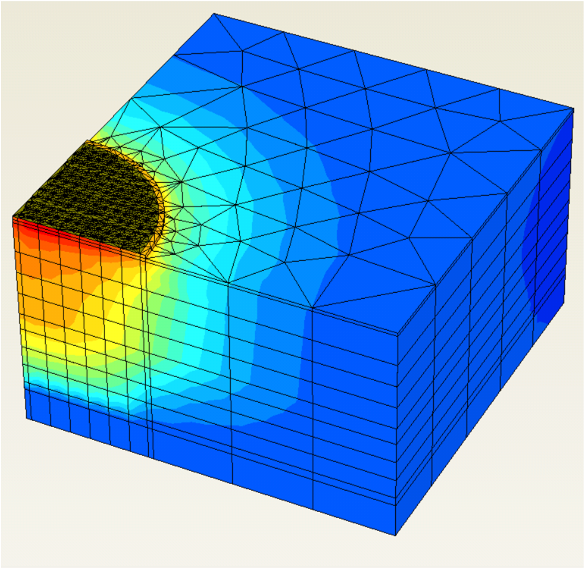
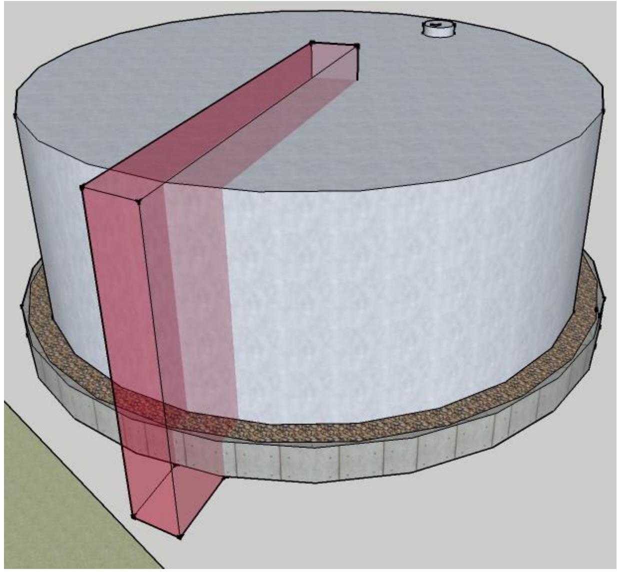
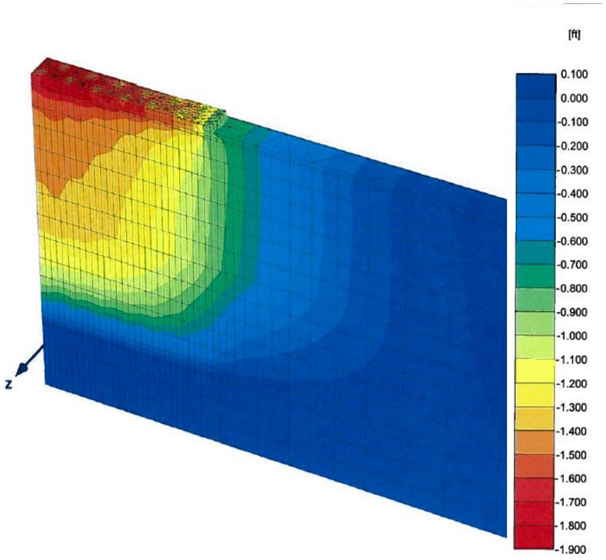
- **Groundwater level was quite high and at less than 1 m below ground level.**

Acceptance Criteria & Ground Improvement Solution

- **Acceptance Criteria**
  - Center Deflection: 100 mm
  - Uniform settlement: 200 mm
  - Tank bottom settlement: 50% of API 653 three years after hydro testing
- **Ground Improvement Solution: Controlled Modulus Columns (CMC)**
- **CMC to depth of 21 m (thin sand layer) with replacement ratio**
  - Diameter: 318 mm
- **CMC to depth of 34 m with lower replacement ratio**
  - Diameter: 470 mm

Design

- **3D finite element analysis modeling a quarter of the tank**
- **3D thin slice of the tank**
- **Hand calculation using Terzaghi's analysis method for rafts on floating piles**

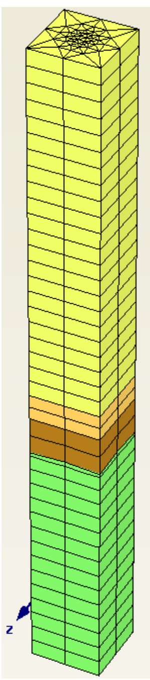
CMC Installation to depth of 34 m (World Record at that Time)

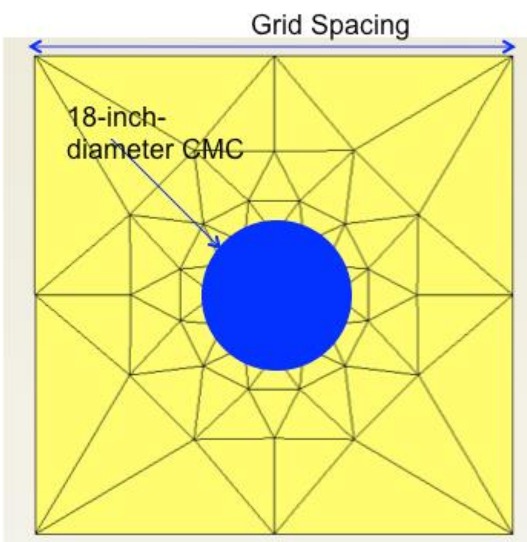
Testing & Quality Assurance

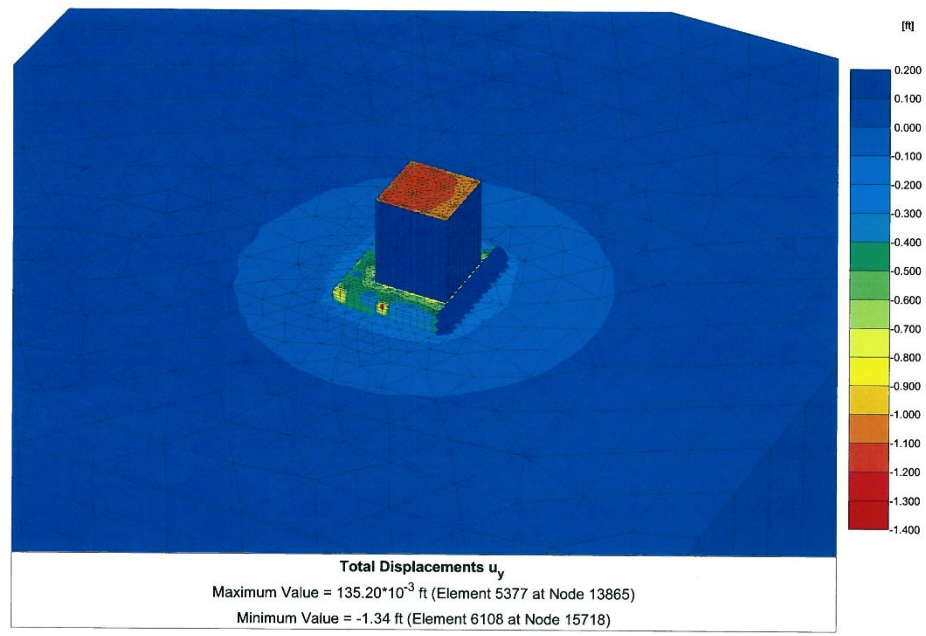
- **Zone Load Test**
  - Test area: 13.7 x 13.7 m^2
  - Load box: 6.1 x 6.1 x 9.75 m^3
  - 30 CMCs
  - Loaded to maximum design load of 143 kPa
- **10 Vibrating wire Piezometers:** measure pore water pressure
- **9 Vibrating wire rebar strain gages in CMCs:** measure stresses
- **5 Multi-depth settlement gages:** measure stain in different layers
- **1 Horizontal extensometer**
- **3 Inclinometer**
- **4 Settlement plates**
  - Three months monitoring
  - Max on load transfer platform: 107 mm
  - Min (on CMC head): 64 mm
  - Max differential: 43 mm (between CMC & grid centre)

Extrapolation of Results to 3 Years

- **Finite element analysis**
  - 3D single CMC simplified unit cell with time dependant consolidation analysis
  - 3D model of field conditions with adjustment of parameters to account for the results of the 3 month monitoring
  - Predicted settlement on load transfer platform after 3 months: 96 mm

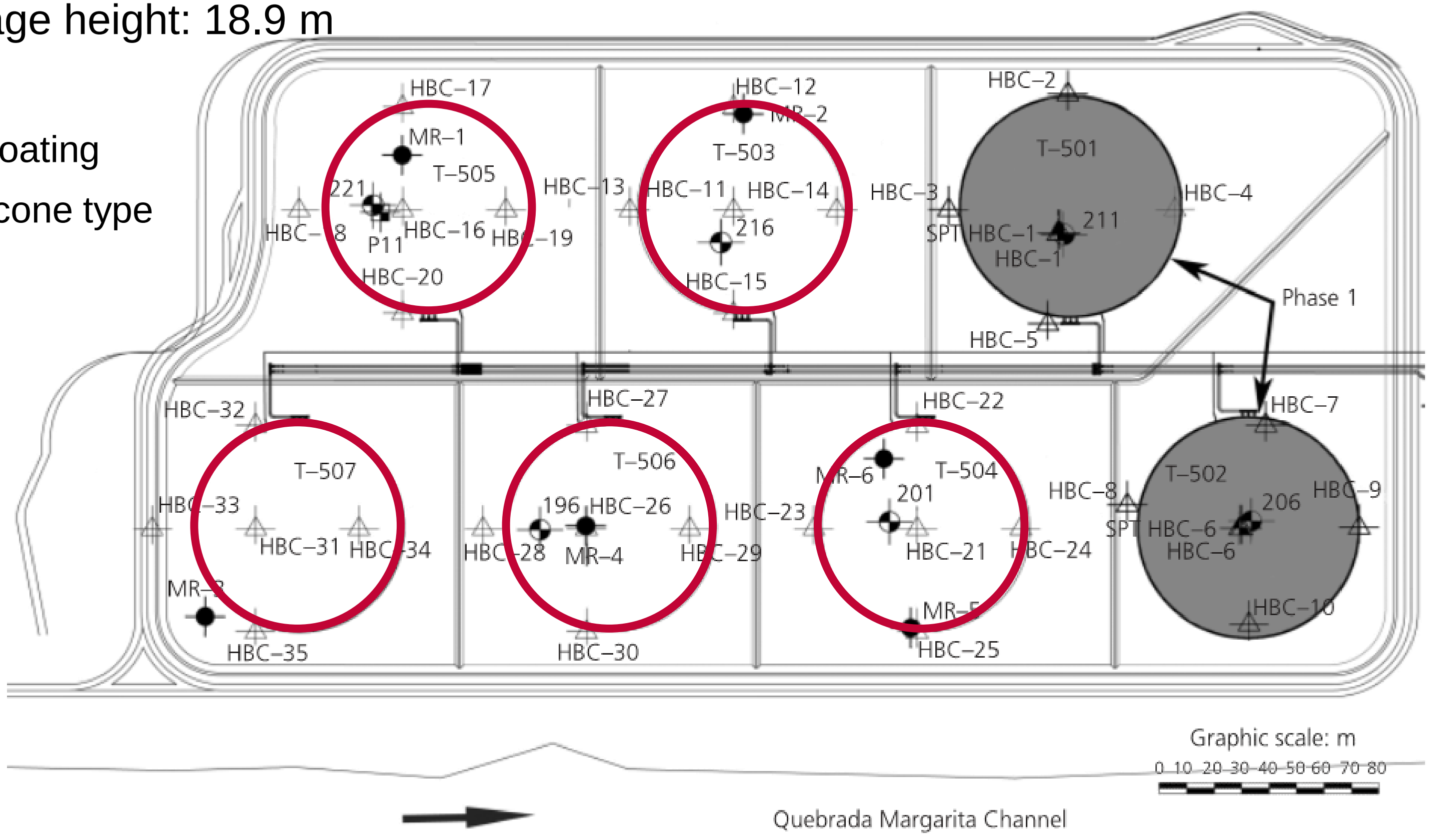




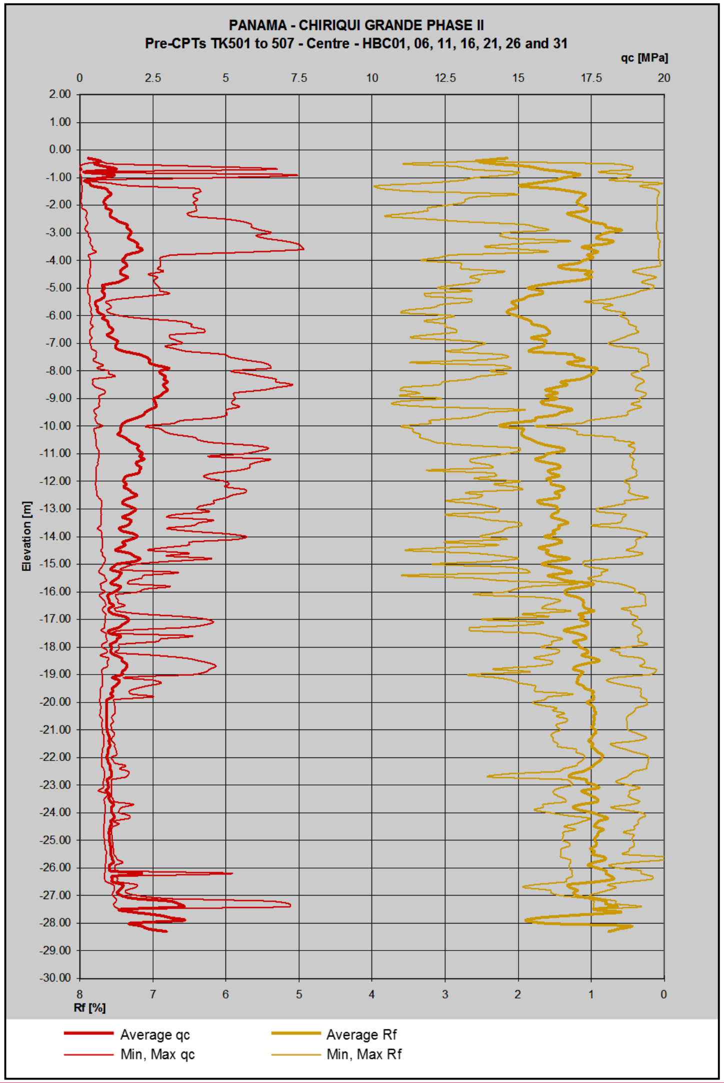


Case History: Chiriqui Grande Oil Tanks- Phase II, Panama

- **Five crude oil steel tanks**
  - Diameter = 76.2 m
  - Height = 20.4 m
  - Product storage height: 18.9 m
  - Roof system:
    - Internal :floating
    - External: cone type

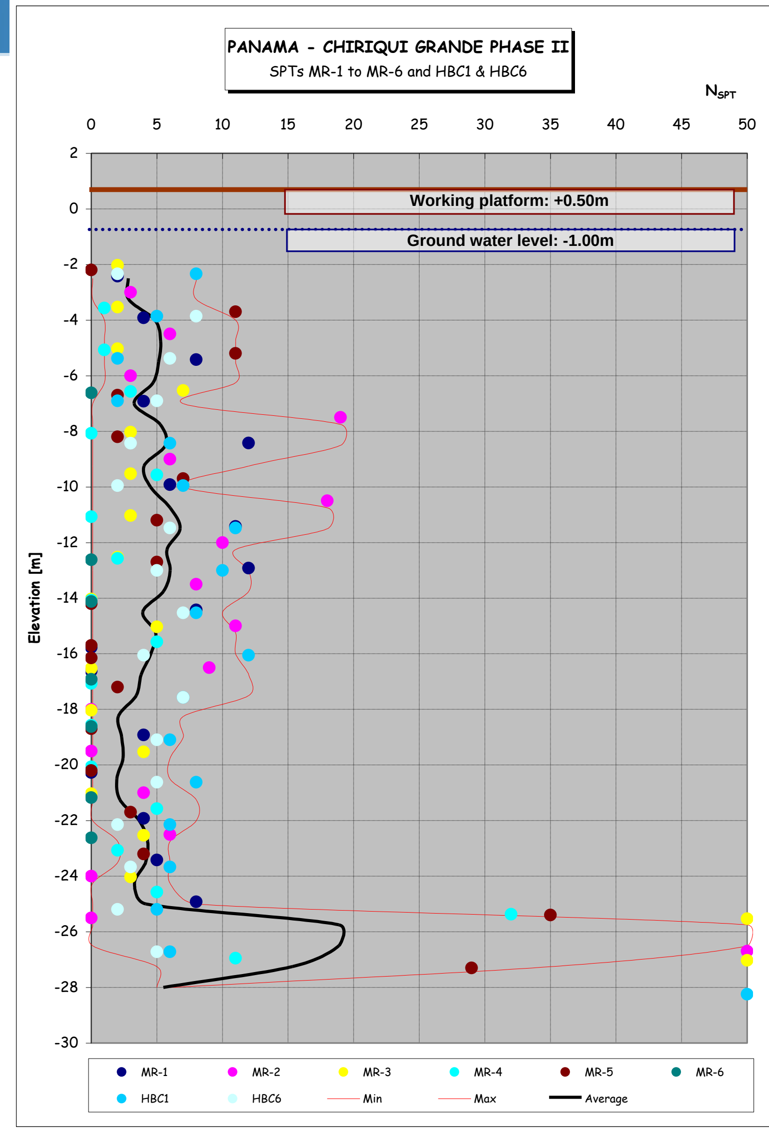


Ground Conditions based on 8 SPTs & 35 CPTs



Layer « M »

Layer « C »



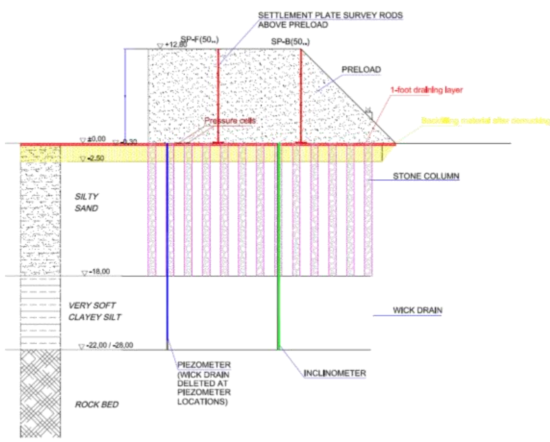
Design Criteria & Ground Improvement Solution

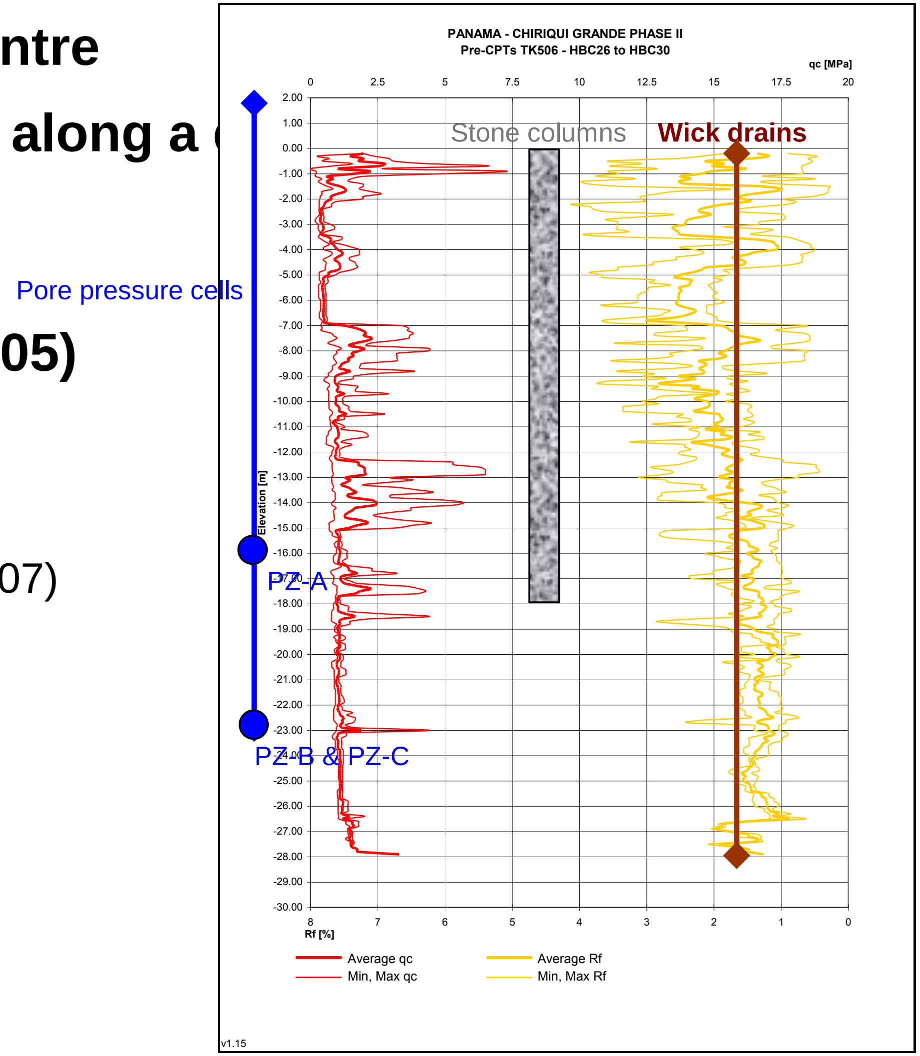
- **Bearing Capacity ≥ 200 kPa with safety factor= 3**
- **Settlements**
  - Ring wall total settlement ≤ to 200 mm
  - Centre to edge dishing ≤ to 150 mm
  - Out of plane settlement ≤ 10 mm in an arc distance of 24.4 m
- **Ground Improvement Solution**
  - Wet top feed Stone Column
    - Diameter: 1.06 m (17.1% replacement ratio)
    - Spacing: 2.44 m triangular
  - Prefabricated Vertical Drains (wick drains)
    - Spacing 0.91 m
    - Depth: up to 28 m
  - Preloading
    - Height: 13 to 14 m (285 kPa)
    - Preloading placement duration: 3 to 4 weeks
    - Preloading period from placement completion: 7 to 9 weeks
    - Treatment diameter per tank: 95 m

Prefabricated Vertical Drains, Stone Columns & Preloading

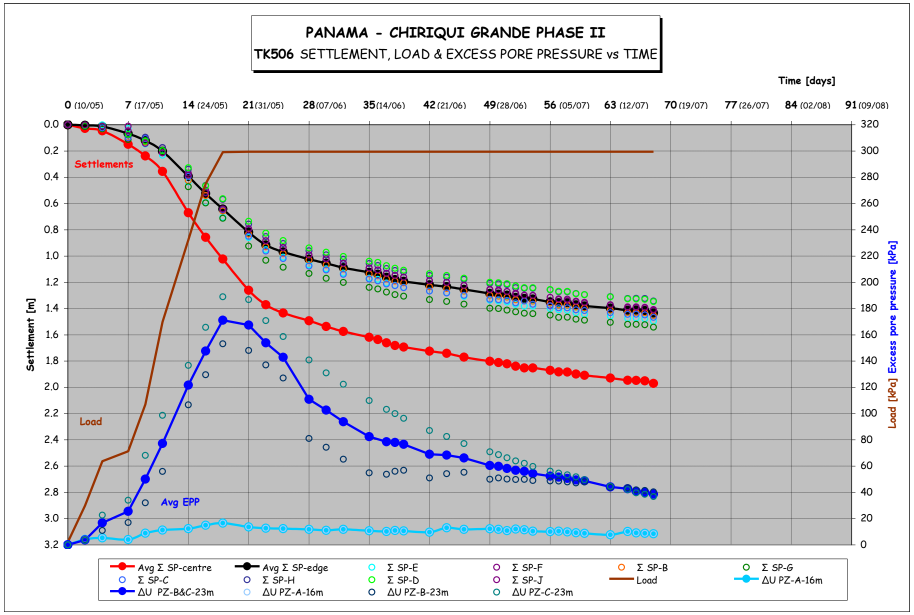
Testing & Quality Assurance

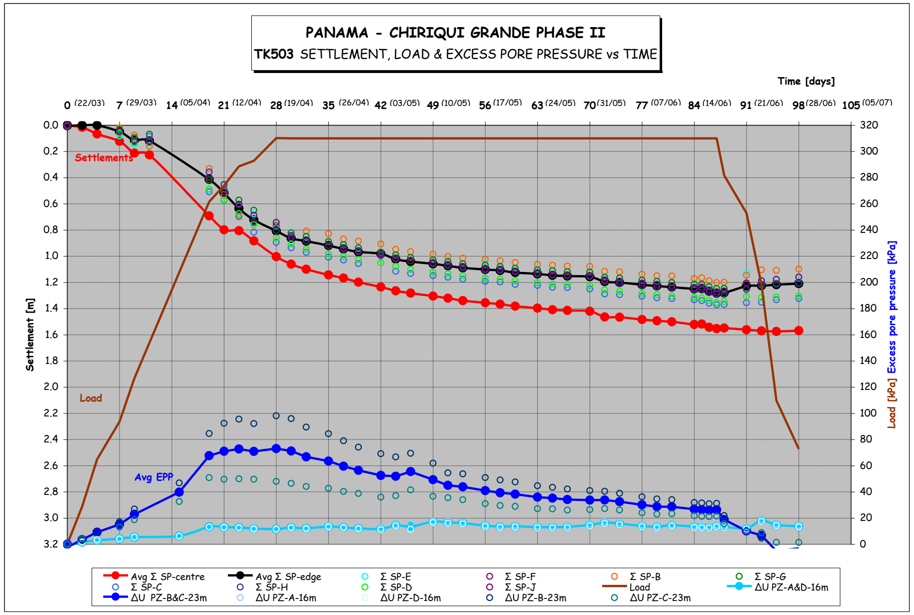
- **8 settlement plates on shell, 1 at centre**
- **3 pore pressure transducers in line along a**
  - -16 m RL (PVD & Stone Columns)
  - -23 m RL (PVD only)
- **3 Inclinometers (only in T-503 & T-505)**
- **3 total earth pressure cells**
  - 2 Stone Columns
  - 1 in between Columns (only in T-505 & T-507)





Typical Preloading Settlement & Monitoring Results





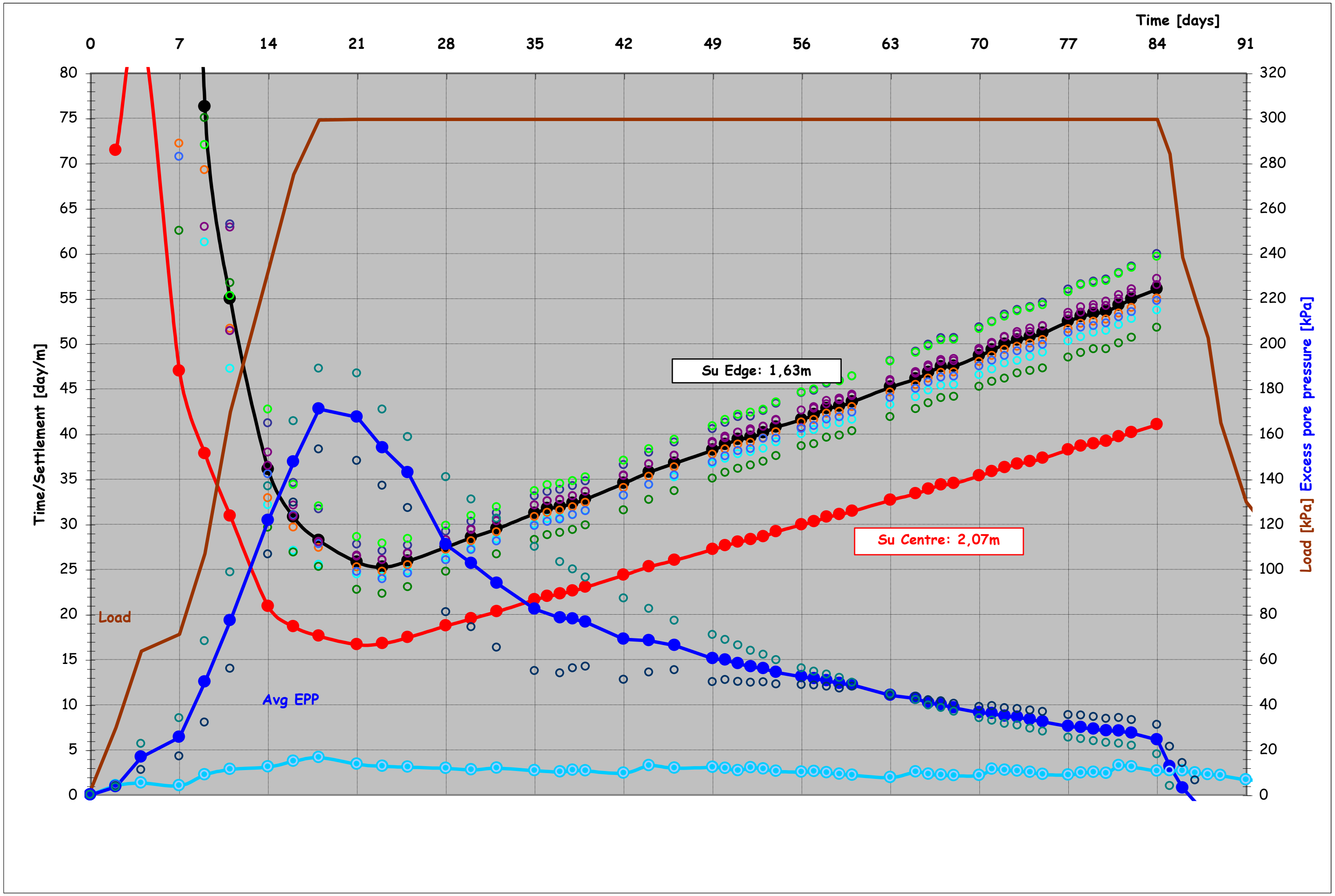
Elevation -16m (Zone with stone columns & wick drains)

Average Excess Pore Pressure of Pz-A in kPa ➔No clogging efffect

Elevation -23m (zone with wicks drain only)

Average Excess Pore Pressure of Pz-B and C in kPa

Tank T-506 Hyperbolic Analysis

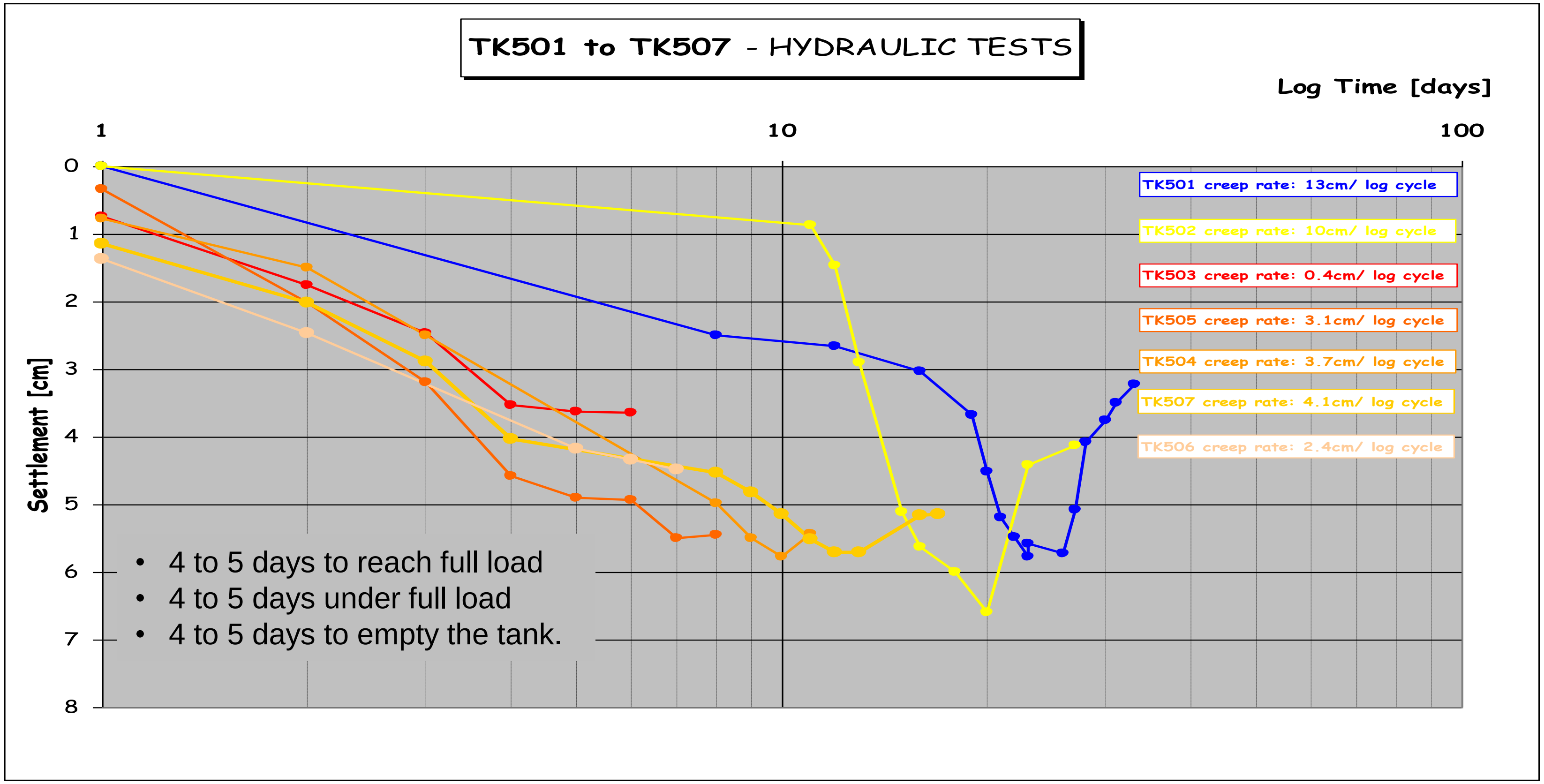


Tentative Back Analysis & Creep Prediction

	Tank #	503	504	505	506	507	Averages
Geometry	Treatment base elevation (m)	-27.0	-26.5	-27.0	-27.0	-26.5	-26.8
	Demucking base elevation (m)	-3.5	-4.0	-3.5	-4.0	-6.2	-4.2
Preload intensity		310 kPa	292 kPa	286 kPa	300 kPa	294 kPa	296.4
Primary consolidation prediction	Oedometric + lateral under preload (ult.settlements hyperbolic method)	1.63	1.97	1.92	2.07	1.74	1.86
Settlements	Settlement at end of preloading (centre)	1.55	1.78	1.77	2.05	1.62	1.75
	Settlement at end of preloading (edge)	1.28	1.32	1.24	1.50	1.33	1.33
	Settlement ratio (edge/centre)	0.83	0.74	0.70	0.73	0.82	0.76
	Time elapsed since installation of half of the preload	76 days	70 days	61 days	73 days	71 days	70 days
Degree of consolidation achieved	From settlements	95%	90%	92%	99%	91%	94%
	From pore pressures	96%	96%	90%	91%	90%	93%

- Less settlements for T-503 due to better ground conditions
- Settlements at the edges: 70 - 83% of settlements at the centre
- Degree of consolidation: 90 - 96% (settlement or excess pore water pressure)

Estimated Future Settlements



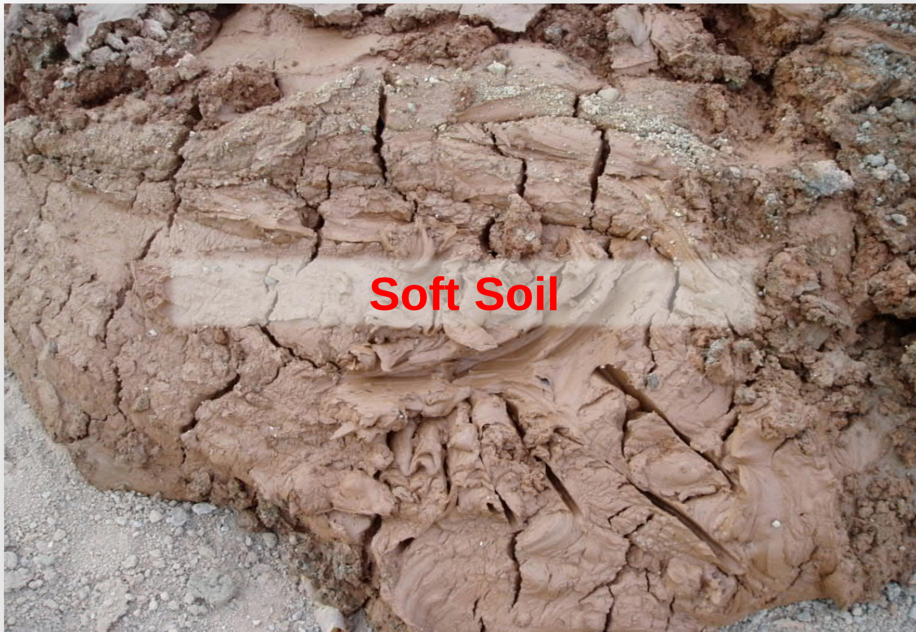
- Estimated re-compression + 50 year creep settlements: 16 - 20 cm in centre of tanks
- Estimated dishing settlements ≤ 6 cm
- Estimated re-compression during hydrotest: 4 to 5 cm
- Actual settlement during: 3.5 to 5.7 cm

Thank You

Development of Deep Soil Mixing Technique for Earth Retention in Malaysia

- Prasad. P.V.S.R., Yee. Y.W., and Raju. V.R

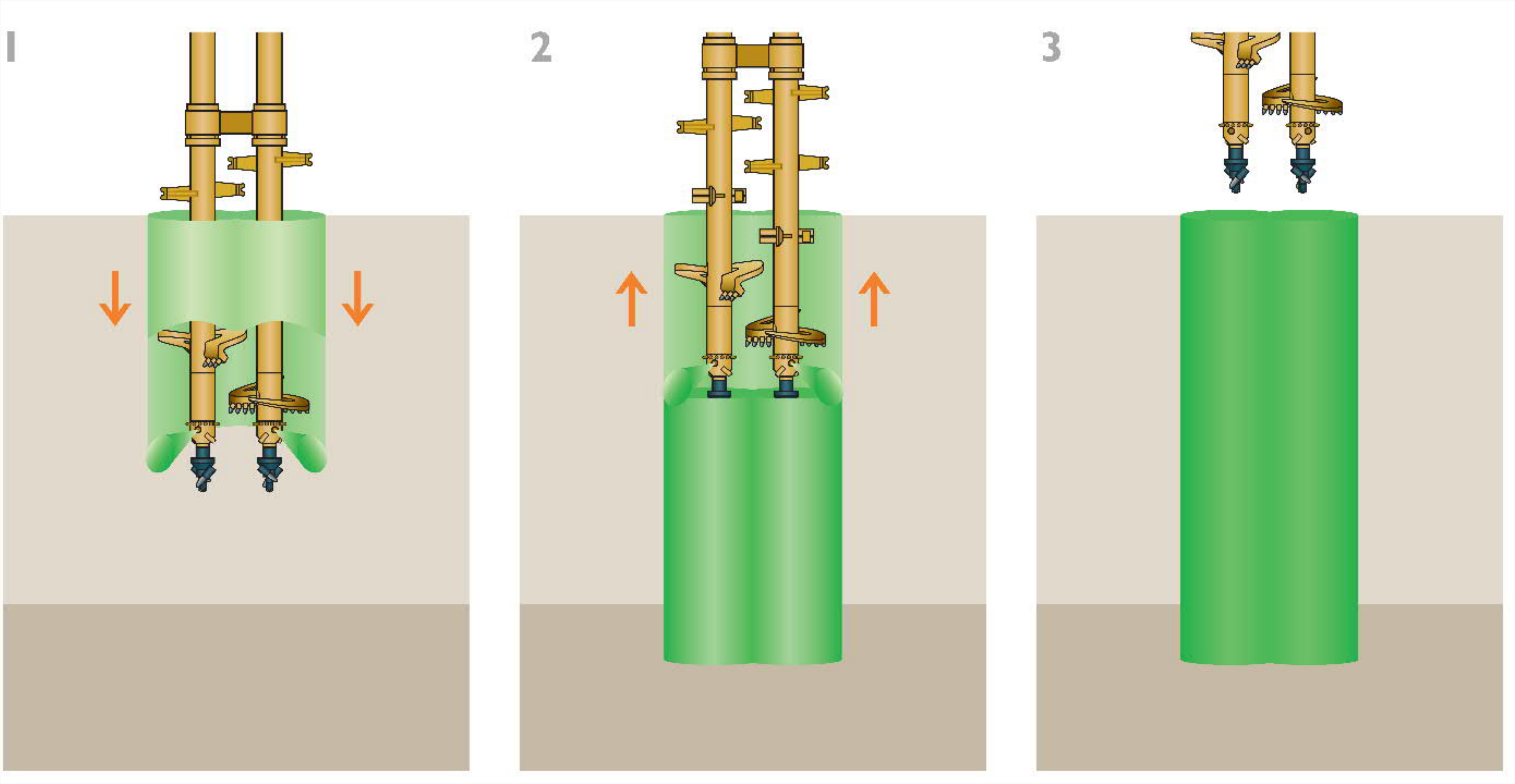
Deep Soil Mixing (DSM)



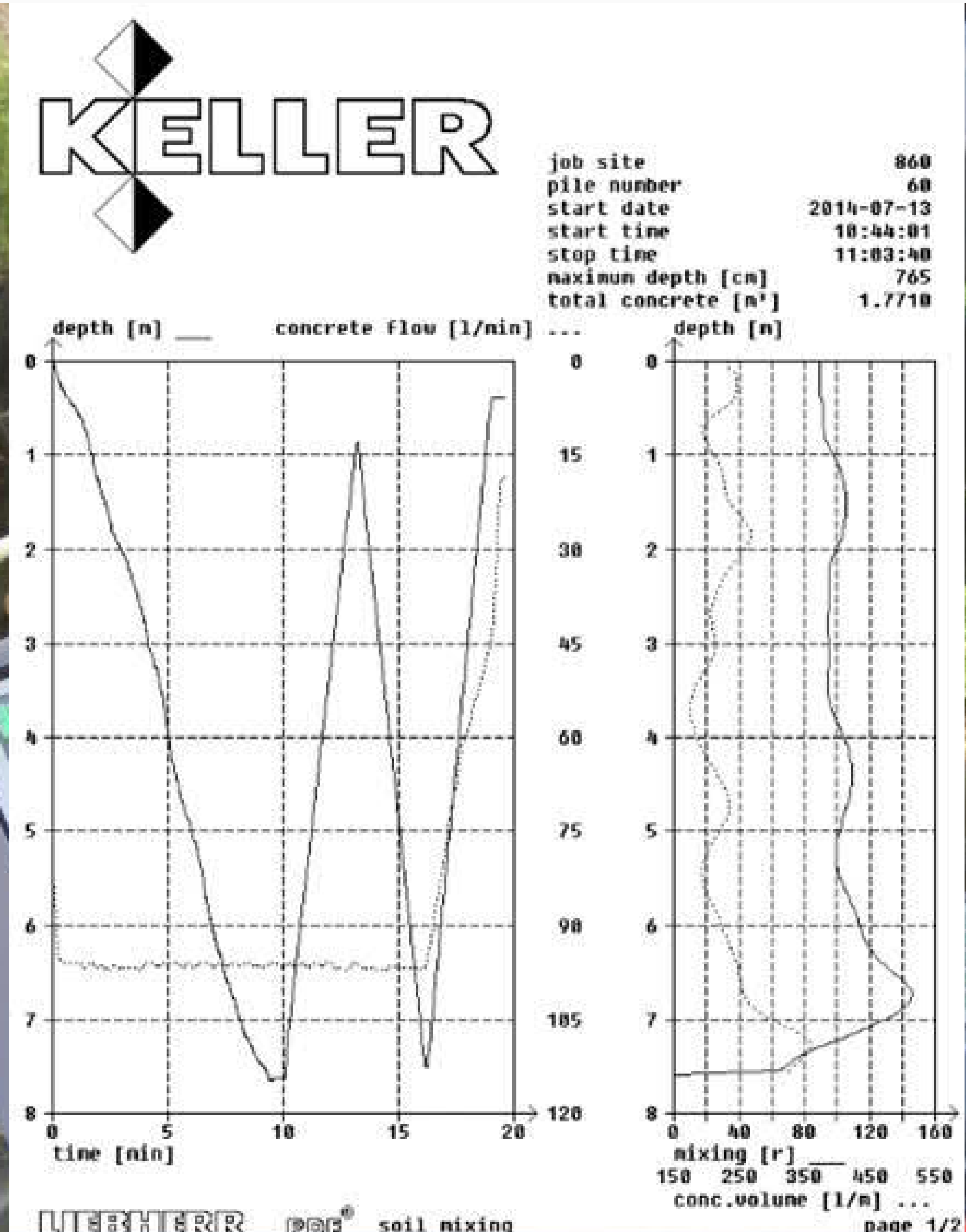
- The mechanical mixing of in-situ soils with a binder
- Increase in shear strength, stiffness and reduced permeability

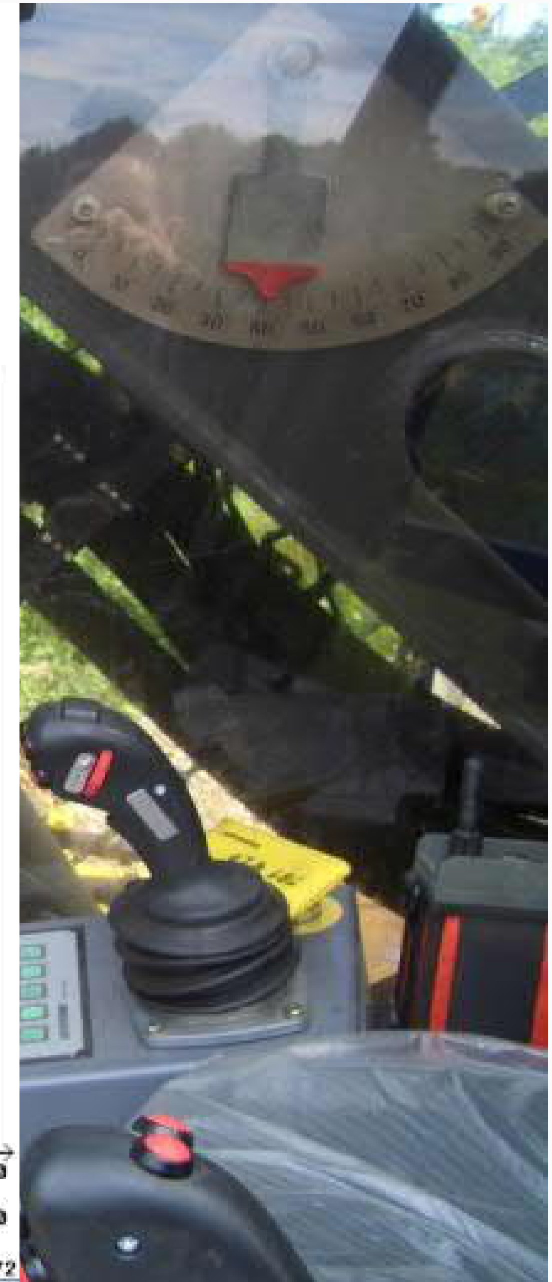
The DSM Process

The DSM Process



Quality Assurance - Automated Data Acquisition & Control





Applications

- Soil Stabilization
- Foundation Support
- Excavation Support
- Tunnel Breakthrough
- Cut-off walls
- Slope Stabilisation

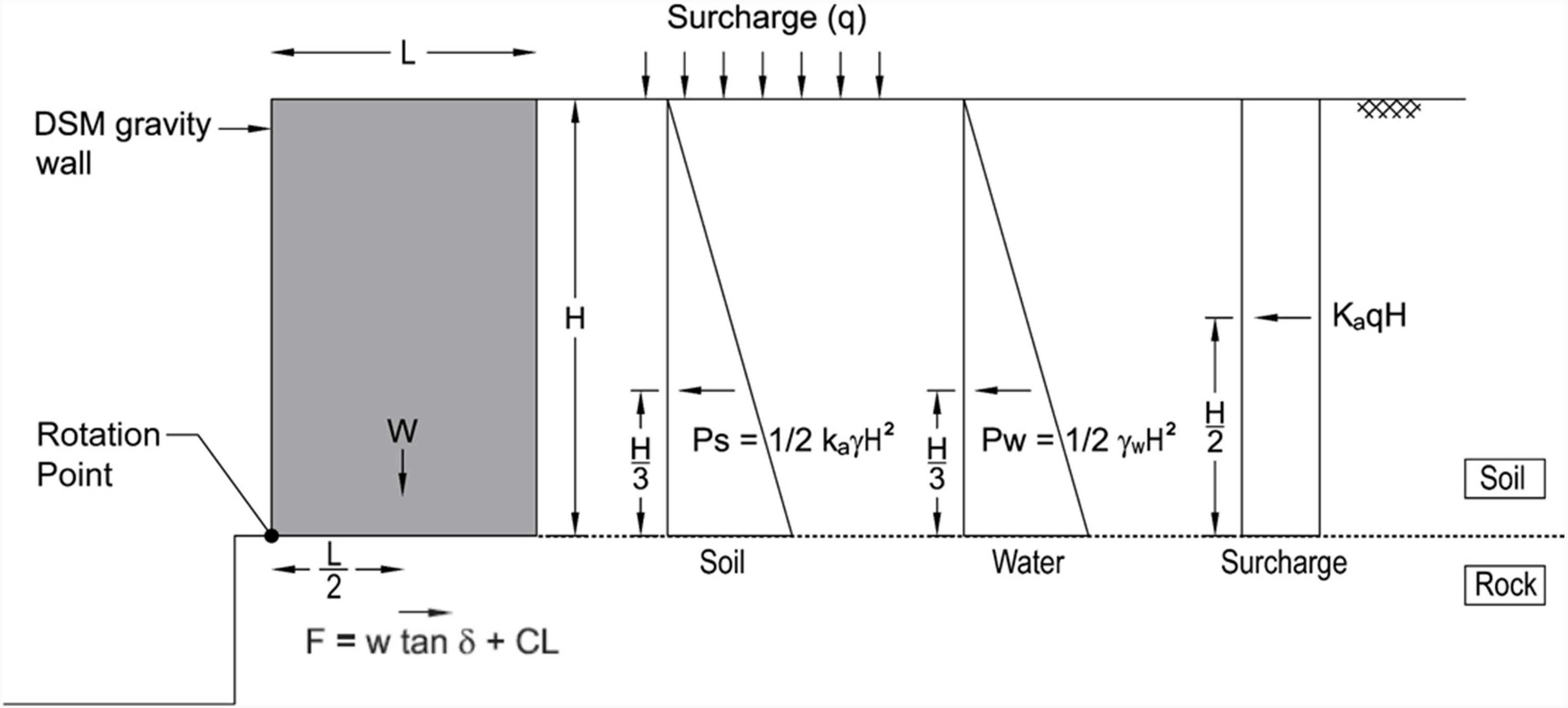
Earth Retention (using Bending Resistance)

Alternative Retaining System

- No rock socketing
- No bending
  - No steel & concrete
  - No anchors, no struts

- Use gravity and friction !

Earth retention using a DSM Gravity Wall



2007 - Fraser business Park – 5m deep

- 7m deep basement excavation
- 5m deep DSM block

2008 - Southgate city – 6m deep

- 7m deep basement excavation
- 3m to 6m deep DSM block

KELLER

2011 - Wisma IAV, Jalan Pasar – 8m

- Max. 9.5m basement excavation
- 5m to 8m deep DSM block

2015 - Wisma Infinitum – 9m deep

- 16m deep basement excavation
- 8m to 9m deep DSM Block

2015 – KVMRT Maluri Portal – 10m

- Max. 15m deep basement excavation
- 10m deep DSM Block

KELLER

Conclusion

- Earth retention to 10m depth is possible using DSM without
  - drilling in Limestone
  - steel and concrete
  - spoil !
  - leaks !!
- However special care should be taken with respect to
  - geotechnical engineering on site and the design office
  - quality control and monitoring during construction
- An innovative method for excavation support !
  - Particularly over KL Limestone


THANK YOU
KELLER
KELLER
WORK IN PROGRESS
WORK IN PROGRESS
LRB 125
KELLER
KELLER
LIEBHERR

19th SEAGC & 2nd AGSSEA Conference
31st May – 2nd June 2016

Twenty Years of CMC Successful Application

Jerome RACINAIS[1], Benjamin THOMAS[1] & Richard ONG[2]
[1]Menard SNC, Nozay, France
[2]Menard Asia, Subang Jaya, Malaysia

menard ASIA

Presentation Objective

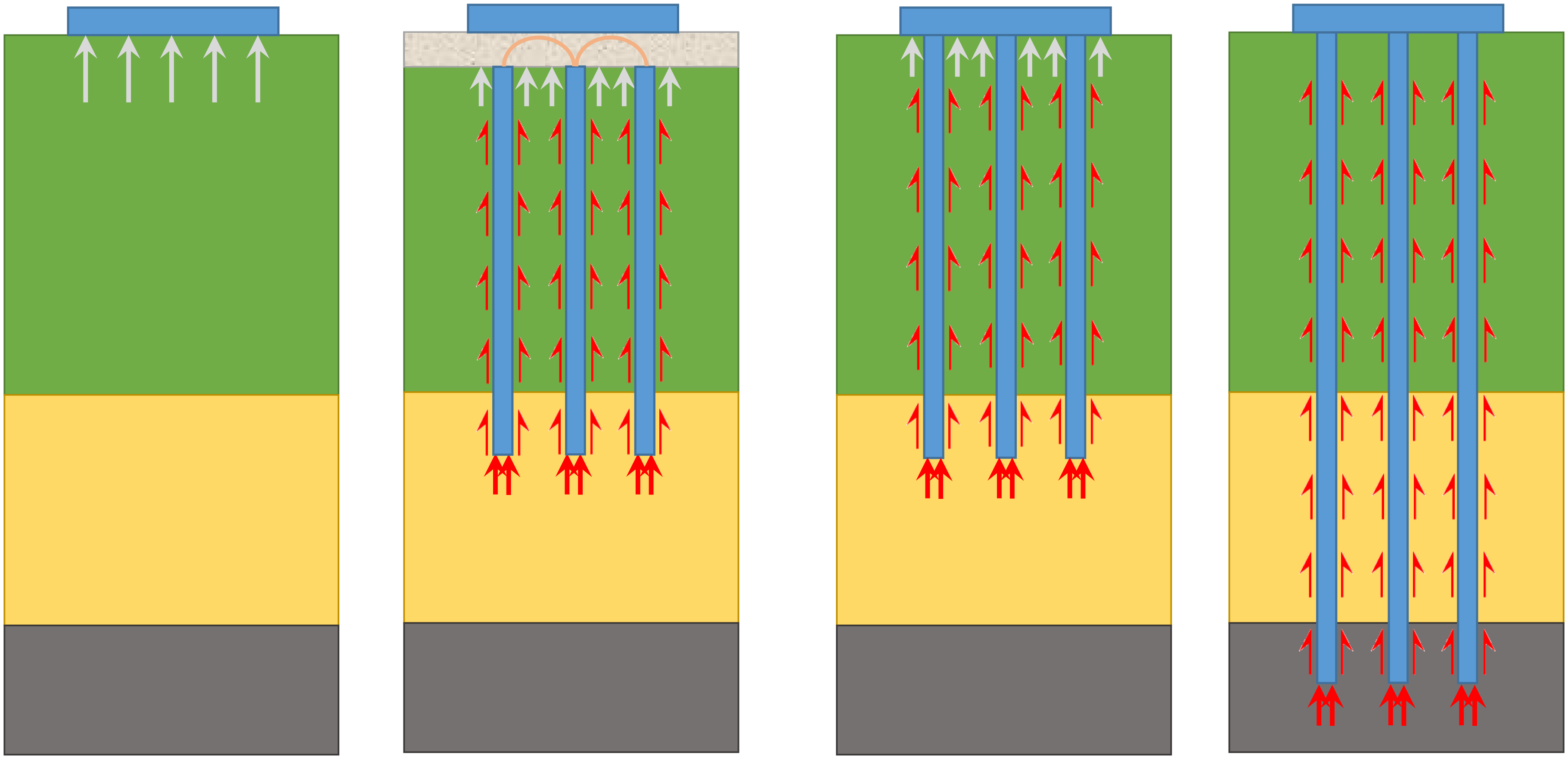
menard ASIA

What is CMC?

Foundation Systems

menard ASIA

RIGID INCLUSIONS → Controlled Modulus Column (CMC)



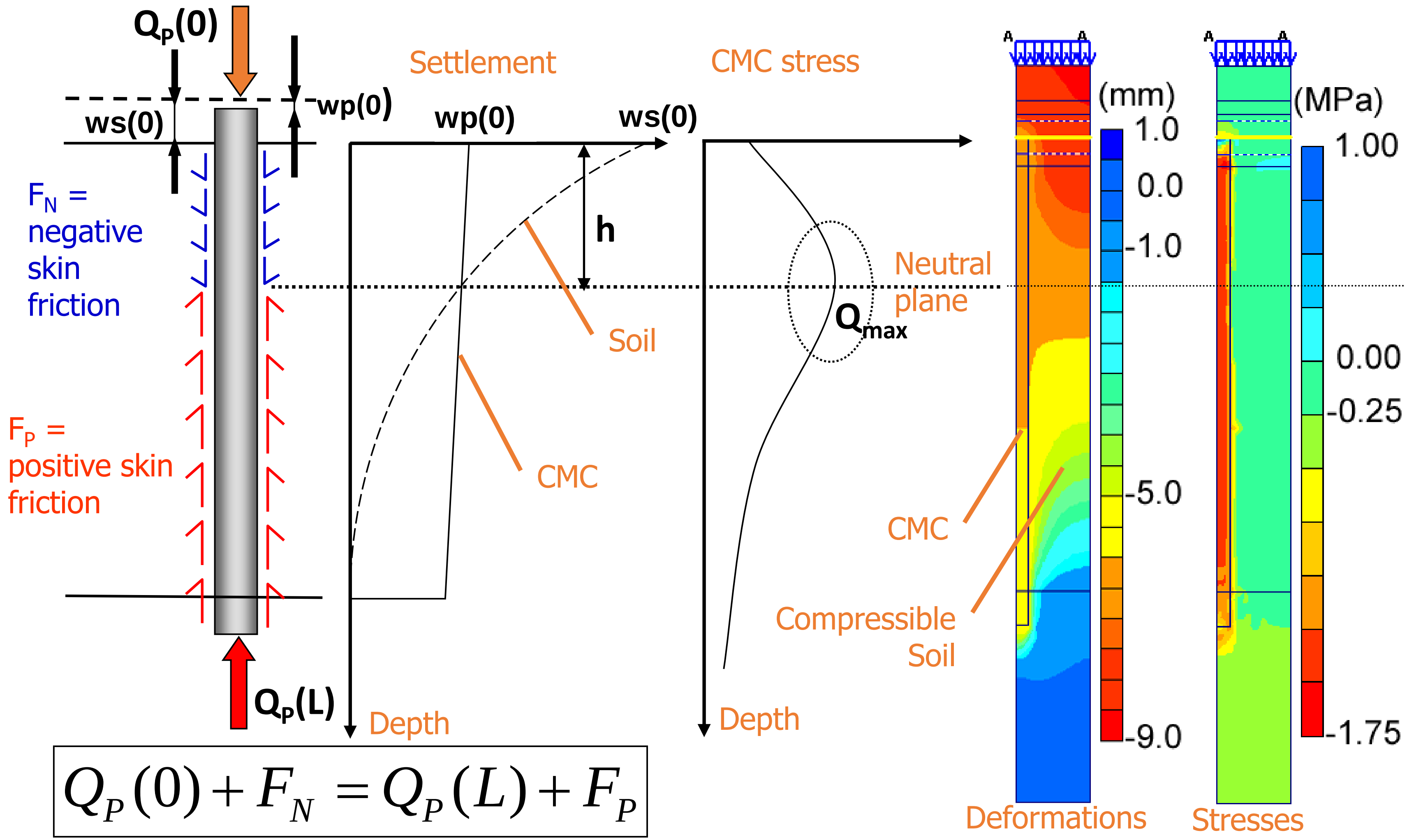
Shallow Foundation

Classical Rigid Inclusion Foundation

Piled Raft Foundation

Deep Foundation

Soil-Inclusion Interaction



$$Q_P(0) + F_N = Q_P(L) + F_P$$

Force Equilibrium

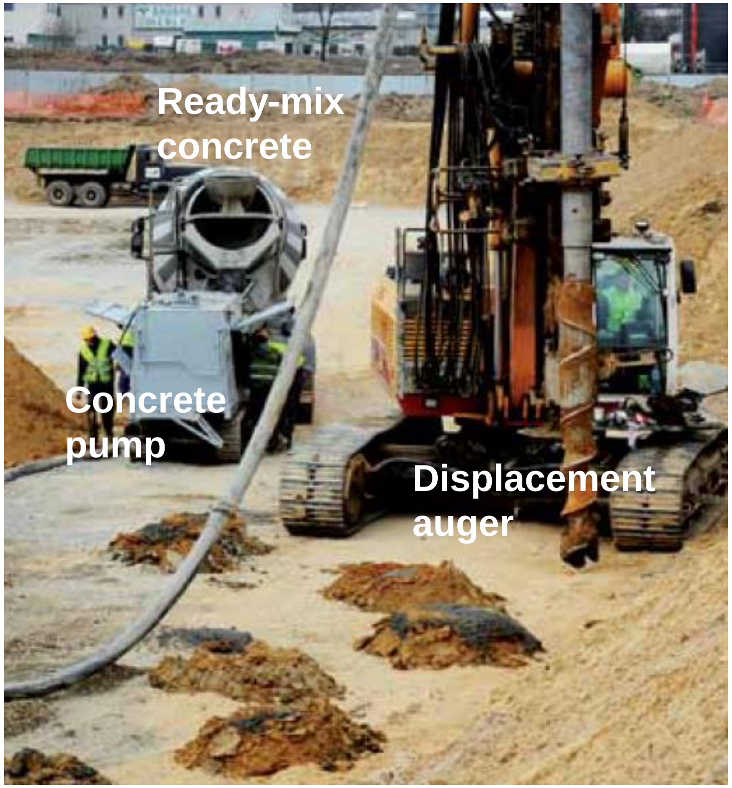
Stresses Mobilisation

Controlled Modulus Columns (CMC)

CMC construction uses a ***displacement auger*** powered by a very large torque and high static downward thrust. Soil is ***displaced*** laterally with ***minimum spoil*** and ***no vibration***. Cement grout flows under ***low pressure*** (<5 bars) as the auger retracts. No in-situ soil mixing or high pressure grouting takes place.

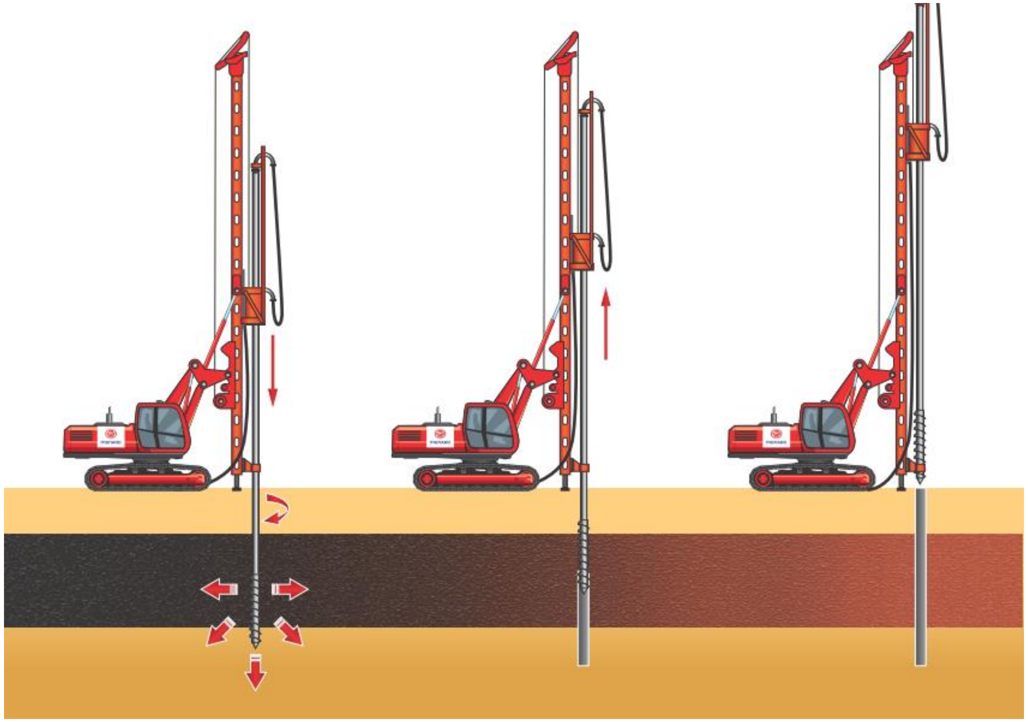
Installation of CMC

Reinforcement with ***displacement*** using Controlled Modulus Columns (CMC) – column forms with cement – a type of rigid inclusions



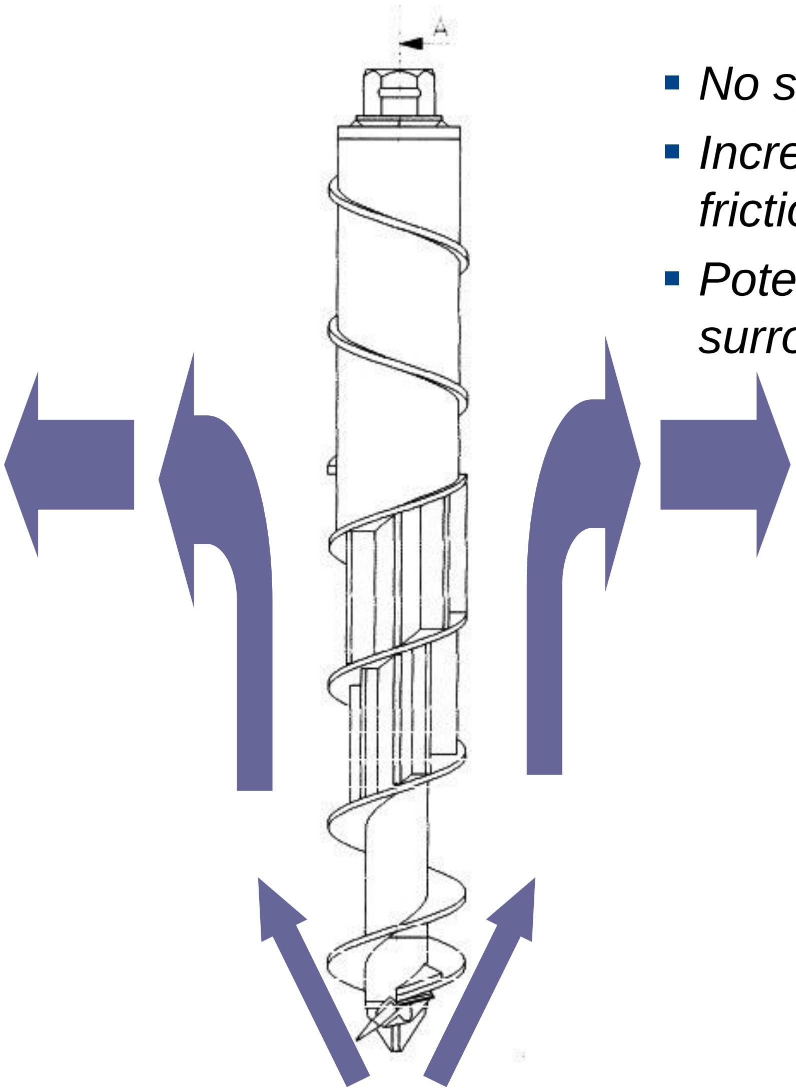
CMC rig with displacement auger

(1) Penetration **(2) Withdrawal** **(3) Completion**

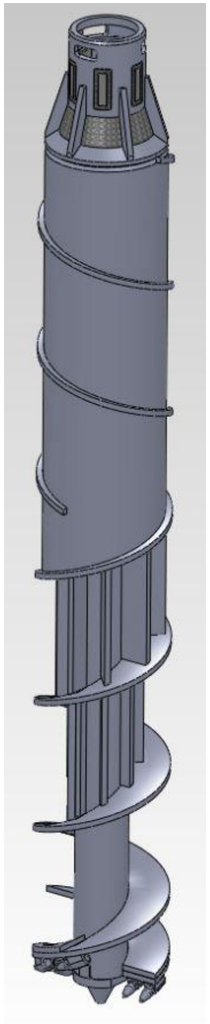


Soil displacement | **Injection of cement grout** | **Full column**

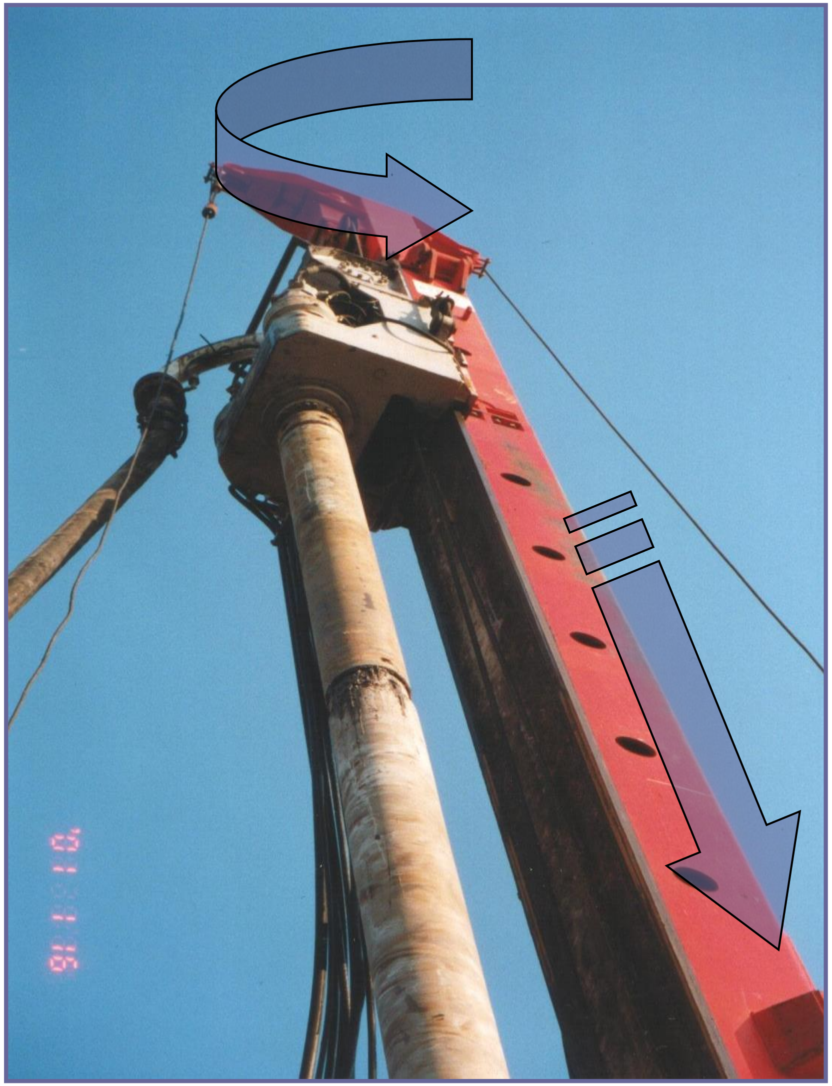
Special Displacement Auger



- *No spoil / No vibration*
- *Increase the unit shaft friction between CMC/soil*
- *Potential increase of the surrounding soil density*



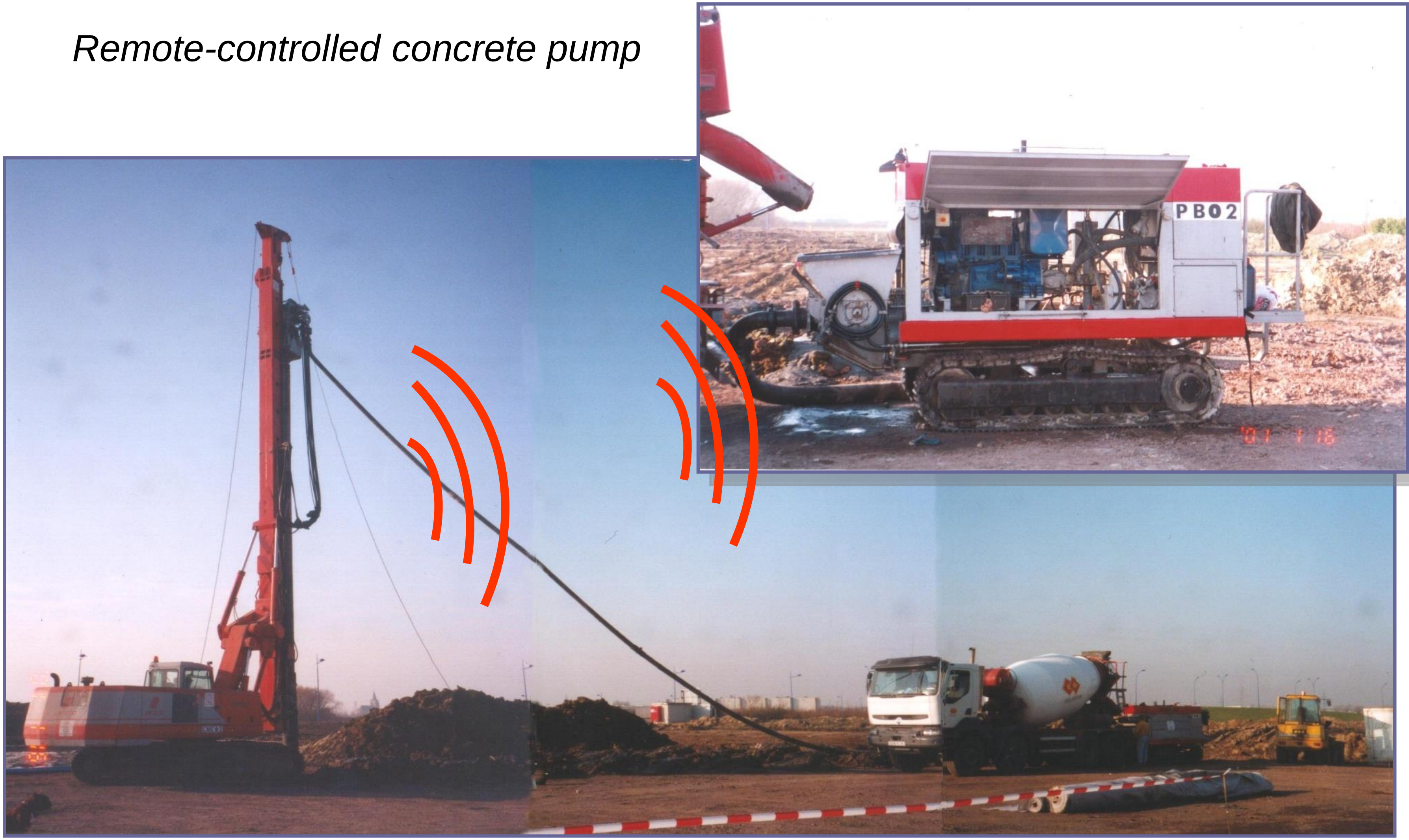
CMC Installation Rig



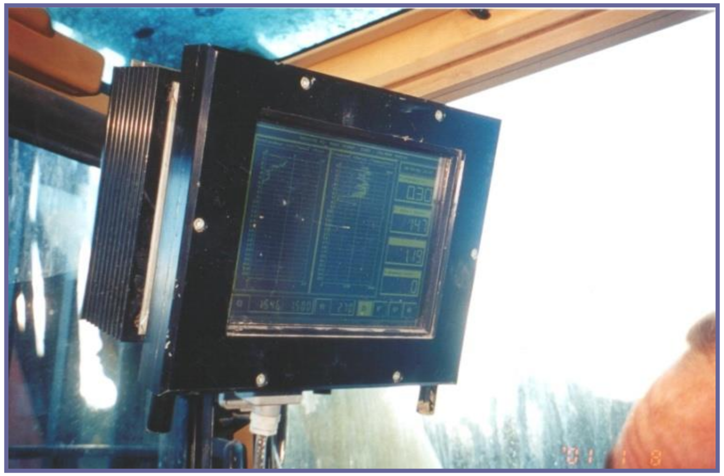
- *High rotation torque*
- *High static down thrust*

Machinery Setup

Remote-controlled concrete pump



Monitoring on CMC Rig



Control of parameters embedded in the cabin of the rig

Installation Process

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Installation by Displacement Method

Minimum vibration & disturbance; minimum spoilt

(suitable for sensitive soil & structure)

Typical CMC Characteristics

- Diameter: 100 – 600 mm
- Grout strength: 5 – 35 MPa
- Allowable load: 5 – 70 tonnes/CMC
- Spacing: typical 0.8 – 3.5 m
- Area replacement: typical 1% - 8%
- Length: typical 10 m to 25 m

Merits & Limitations

Merits:

- ***Densify*** and ***reinforce*** existing ground to create a composite mass
- ***Urban friendly technology:*** minimum spoils & noise; vibration free
- ***Fast production:*** 500 to 3,000 lin.m per rig per shift
- High settlement reduction: up to a factor 10
- High bearing capacity enhancement
- ***No in-situ mixing*** of soil, results more consistent.
- No structural link with foundations
- No casing, no drilling mud, works in very soft soil SPT=0

Limitations:

- Not for ***high rise buildings***
- Need ***steel reinforcement*** in columns if high moment anticipated

20 years of successful application?

Development of CMC

- First CMC project in the North of France for the foundation of a stadium with inclusions of around **5 m** in length in **<u>1996</u>**.
- The man behind it…

- ***Prof. Jean M. Cognon (ENPC & Menard)***
- Jun **<u>2014</u>** – Genesis Raceland project: CMCs up to **50 m**.

Development of CMC

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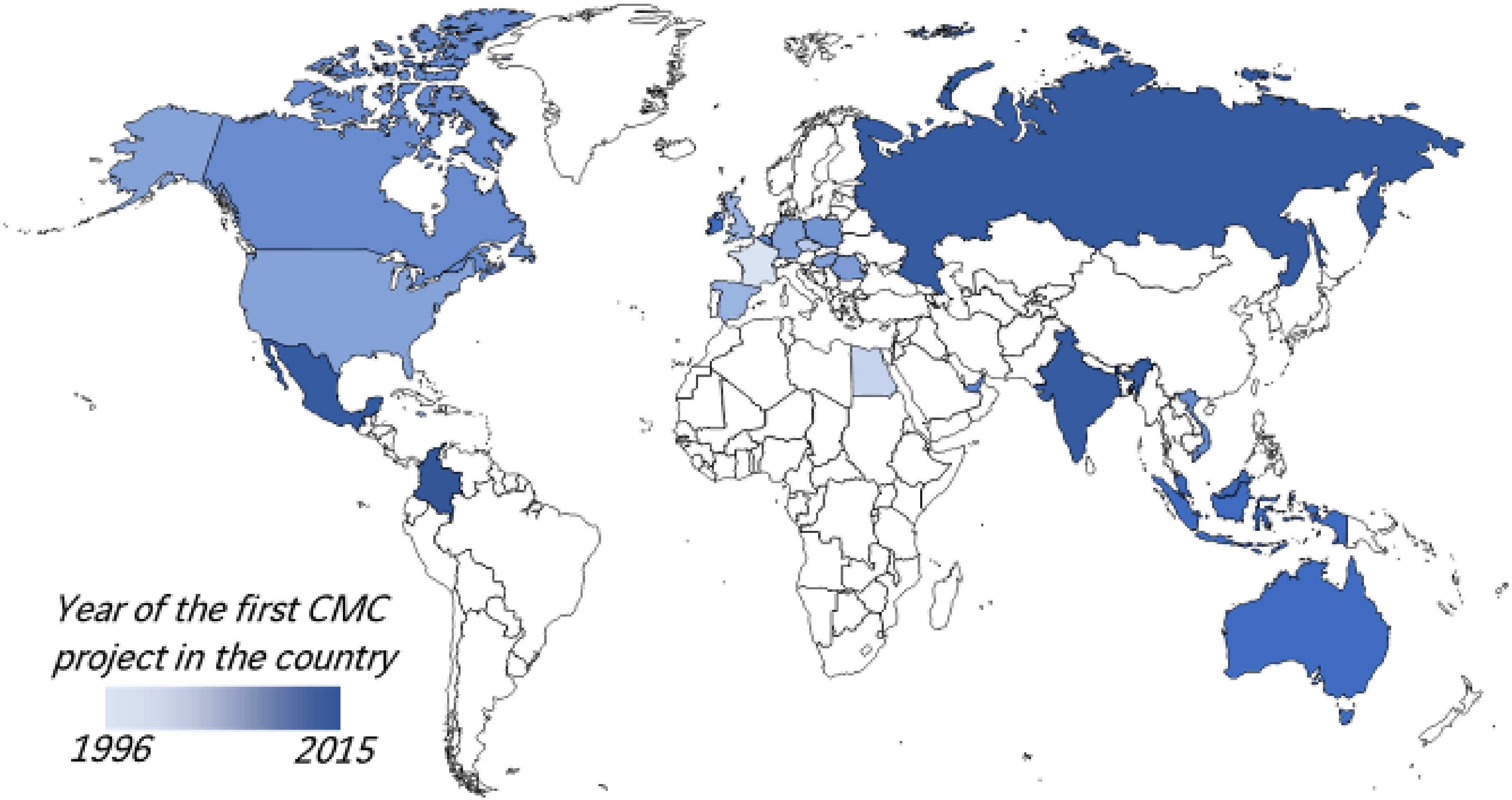
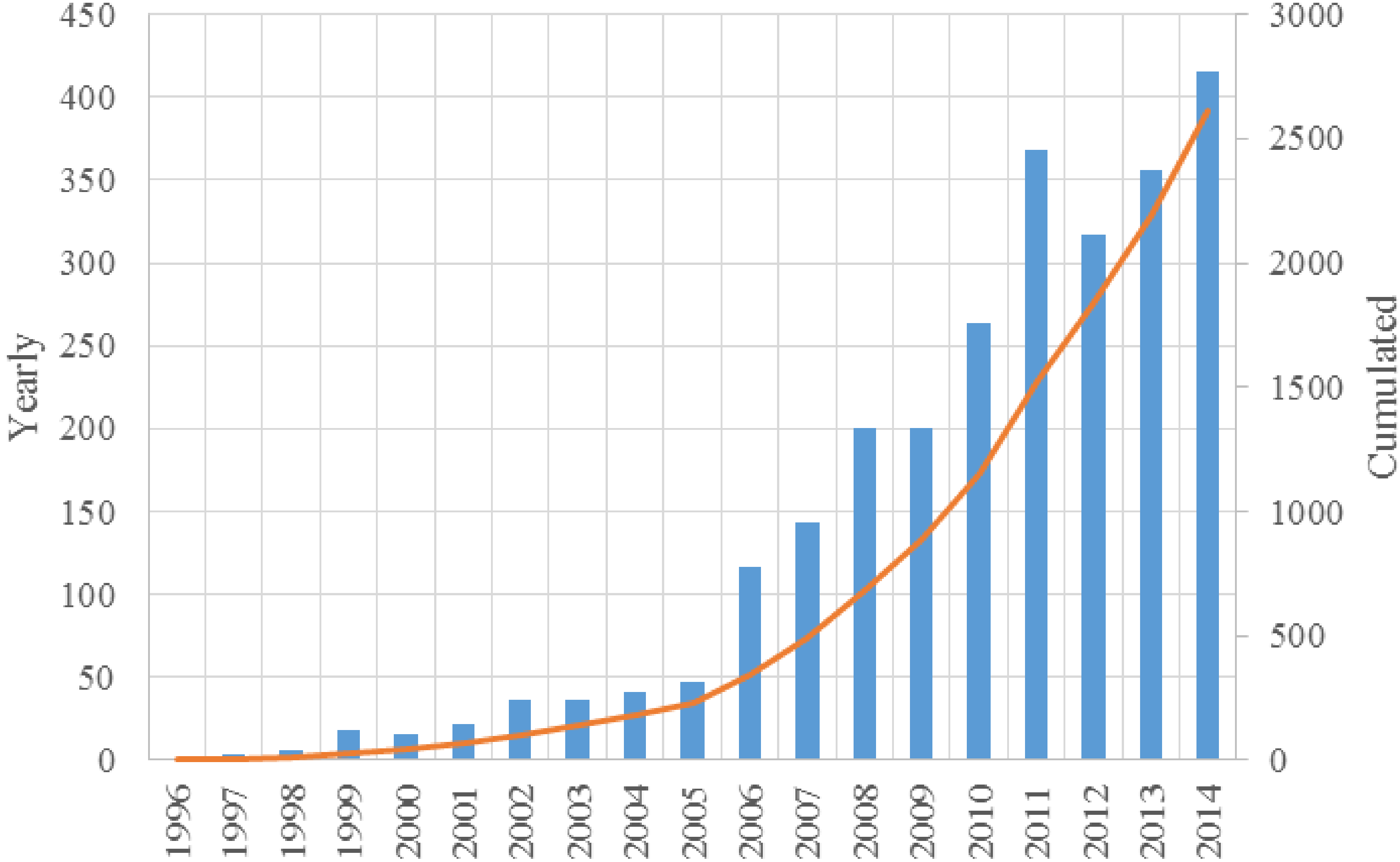
- CMC projects worldwide



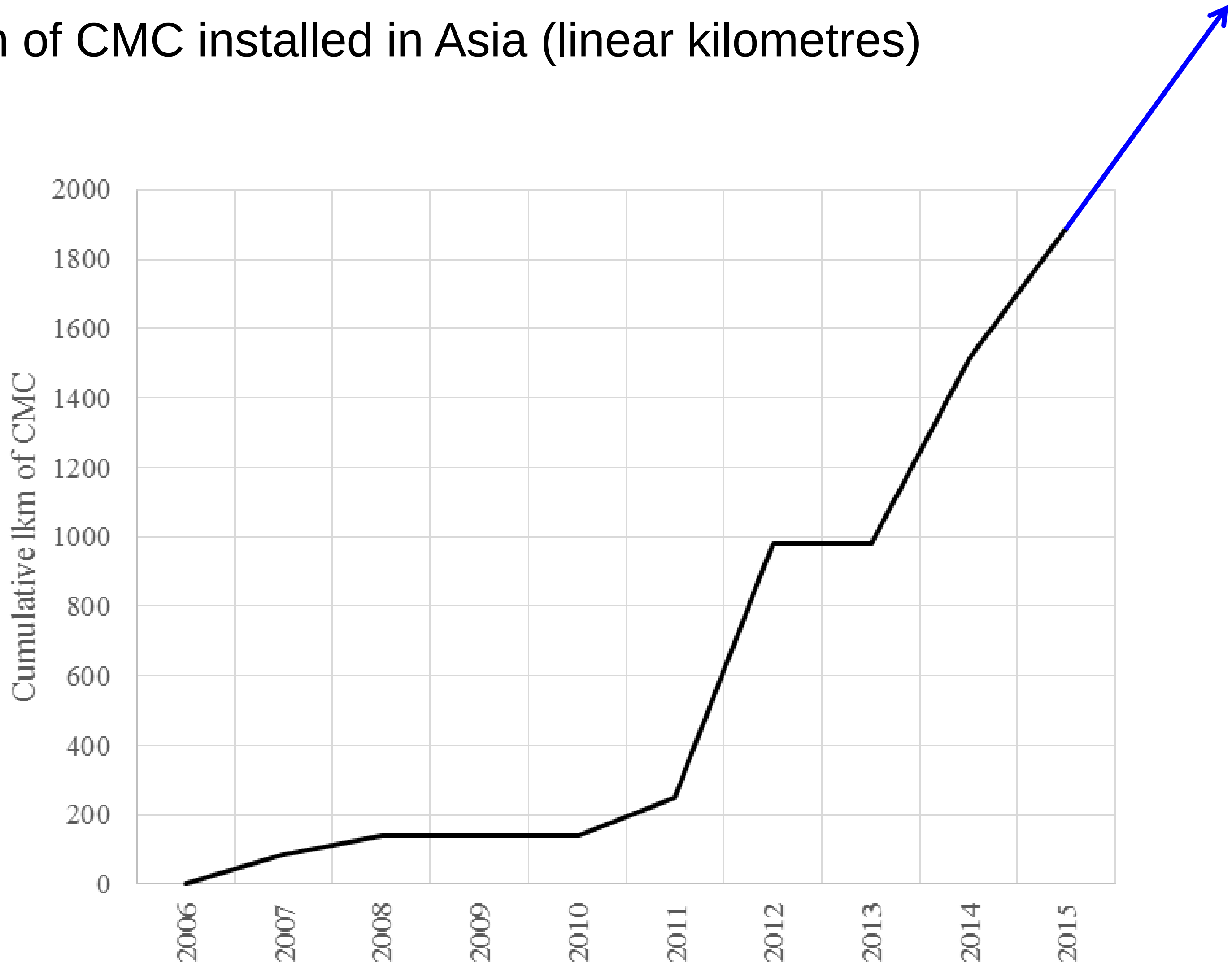
Development of CMC

- Evolution of number of CMC projects worldwide



Development of CMC

- Evolution of CMC installed in Asia (linear kilometres)



ASIRI: Amélioration des Sols par Inclusions RIgides

- Milestone in rigid inclusion design & execution

Chapter 1: Description, history, initial developments and launching of the national project

Chapter 2: Operating mechanisms

Chapter 3: Design methods

Chapter 4: Design considerations

Chapter 5: Justifications

Chapter 6: Geotechnical investigations

Chapter 7: Executions conditions

Chapter 8: Controls and instrumentations

383 pages. Edited in 2013

ASIRI: the Partners

Applications of CMC

Applications

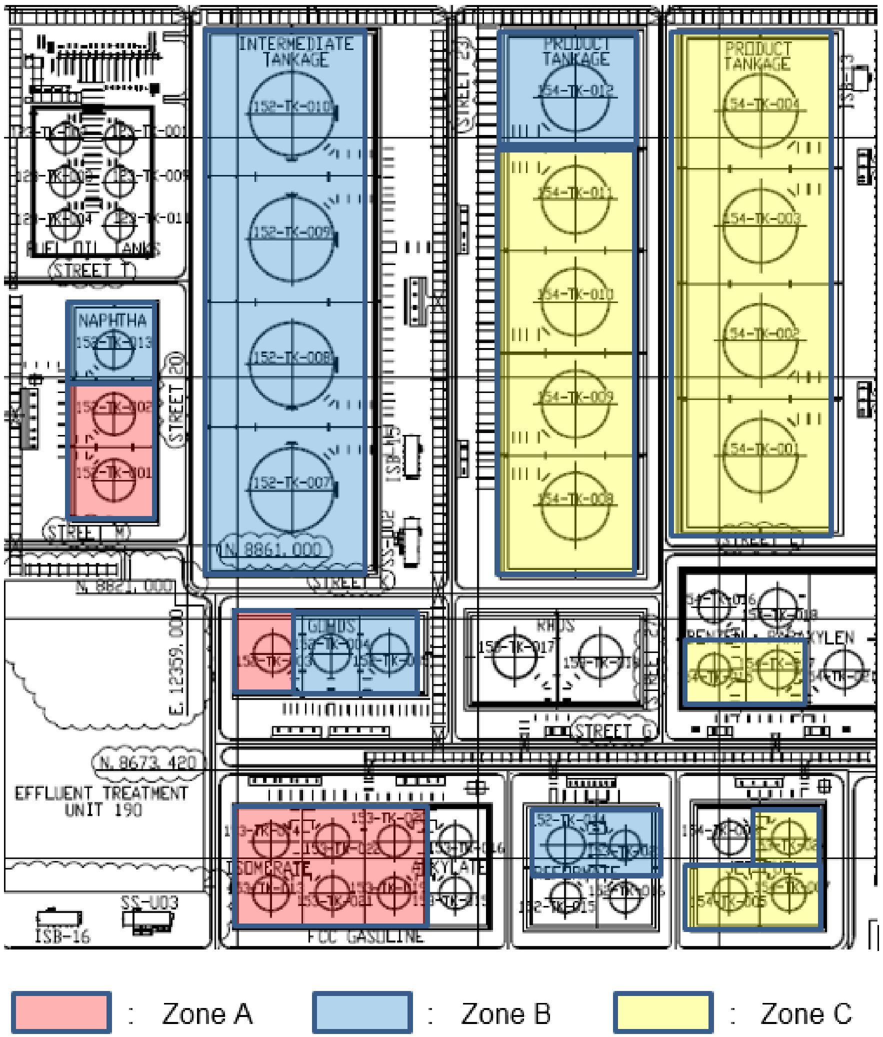

Tanks, silos, wastewater treatment plant


Road and railway embankments


Wind turbines


Industrial warehouses and factories

Nghi Son Refinery, Vietnam



Scope of Works:

- CMC for 32 tanks
- Diameter: 24 m to 69 m
- Design loads: 139 kPa to 210 kPa + platform load + pad load
  (= up to 350 kPa)
- Associated engineered earthworks

Works Execution:

- CMC linear: 225,000 lin.m
- Earthworks: 20,500 m^3
- Production: 5 months
- 2 to 3 rigs

Nghi Son Refinery, Vietnam

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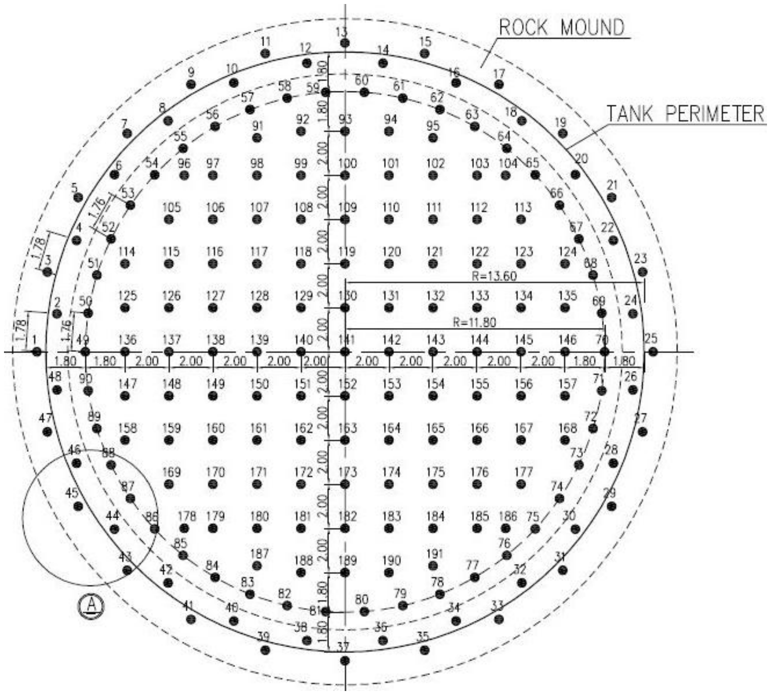
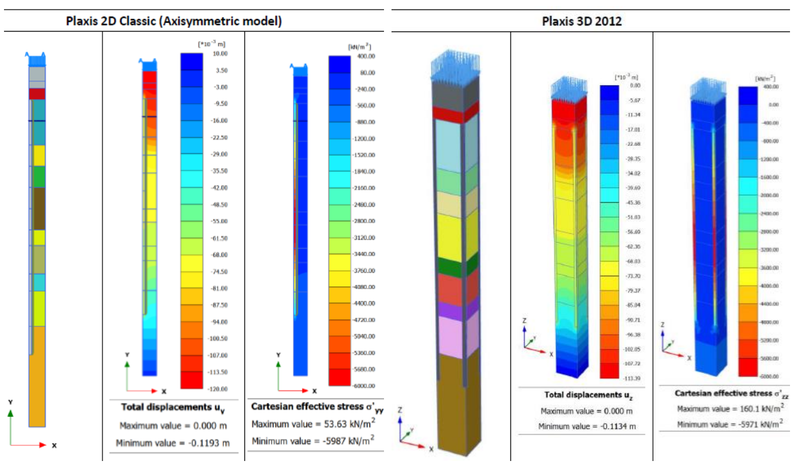




Applications

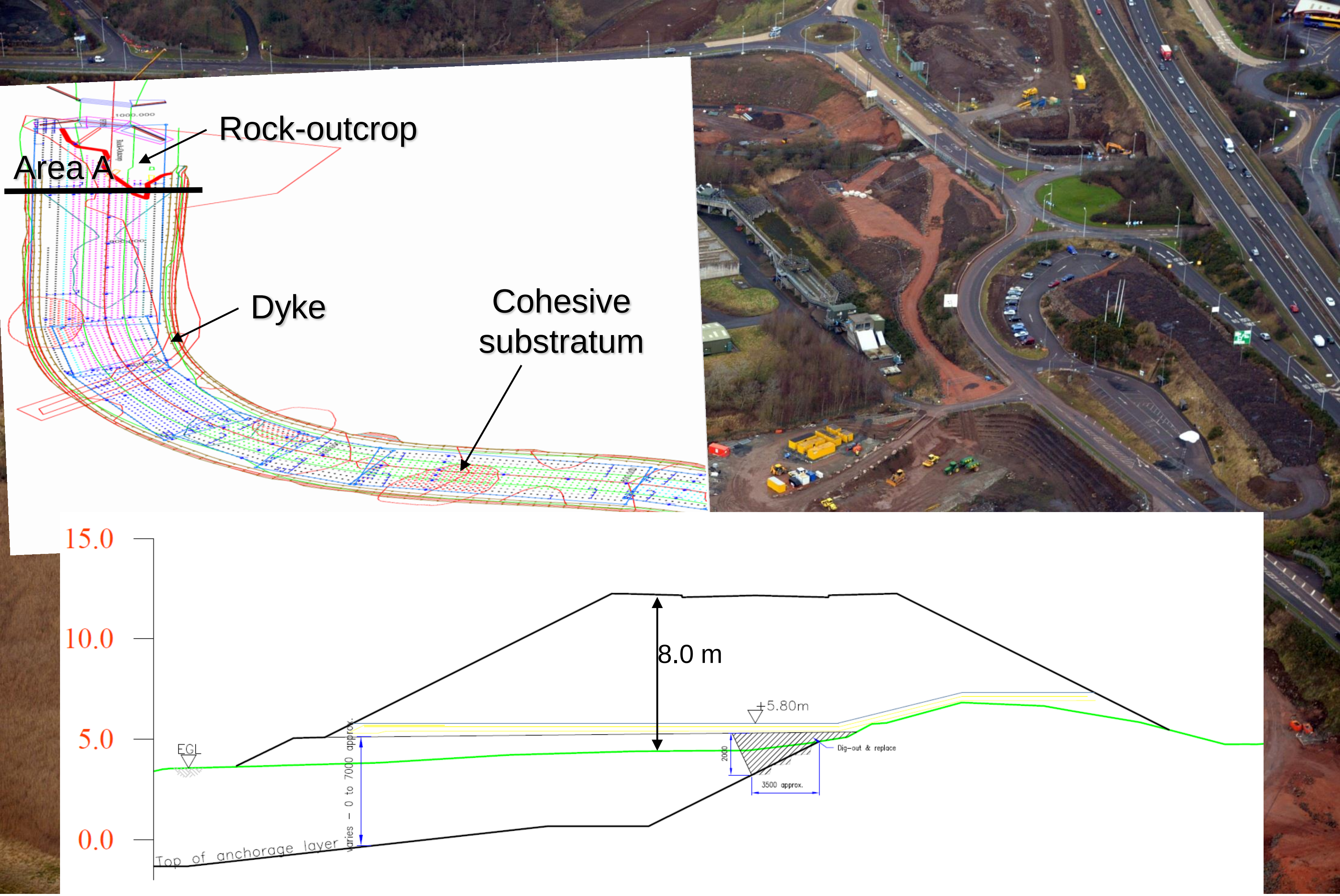

Tanks, silos, wastewater treatment plant


Road and railway embankments

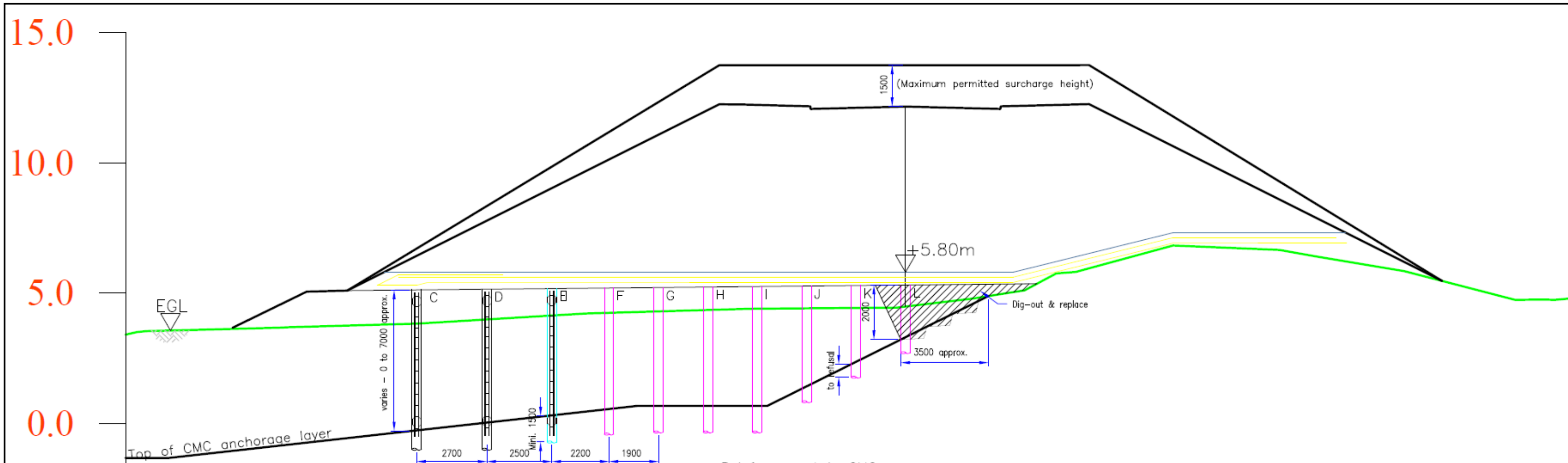

Wind turbines


Industrial warehouses and factories

Bridge Crossing, Scotland


Rock-outcrop
Area A
Dyke
Cohesive substratum
15.0
10.0
5.0
0.0
8.0 m
+5.80m
EGL
Dig-out & replace
2000
3500 approx.
varies – 0 to 7000 approx.
Top of anchorage layer

Bridge Crossing, Scotland



Reinforcement in CMC

1. Reinforcement to be placed in CMC row A to E
2. Reinforcement to consist in 4 units of 16 mm rebar with B8 links at 400 mm spacing of grade 500B
3. Reinforcement length is to extend to the top of anchorage layer (about 6 m) with defined tolerance (CMC can be overdrilled to accomodate the reinforcement length)

Dig–out & replace around rock outcrop

1. soil to be dig out and replace around rock outcrop up to 2m deep from Existing Ground Level with Selected Well Graded Granular material class 1A/1B (see FRC–P–___E–000–D–NT–SKE–90051 for benching details)
2. Dig–out & replacement works to be executed before adjacent CMC installation
3. CMC at rock crop position shall be ommited, CMC around rock crop shall be executed from depth of 2.0 m when possible, including within the dig–out and replaced area.

Longitudinal CMC spacing is 1.9 m for every row

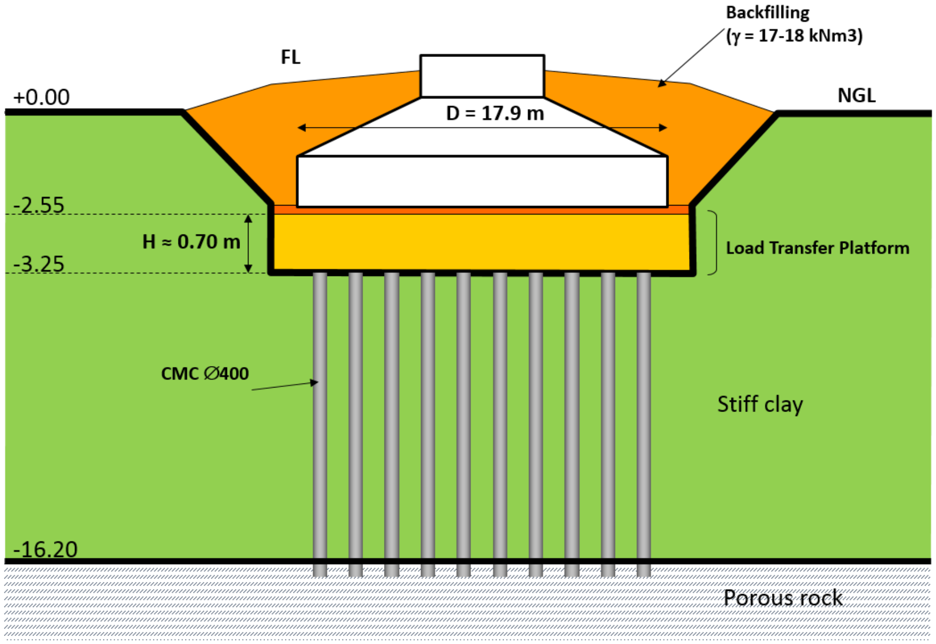
Area A
Typical cross–section – Chainage 960.0 m
Scale 1:200

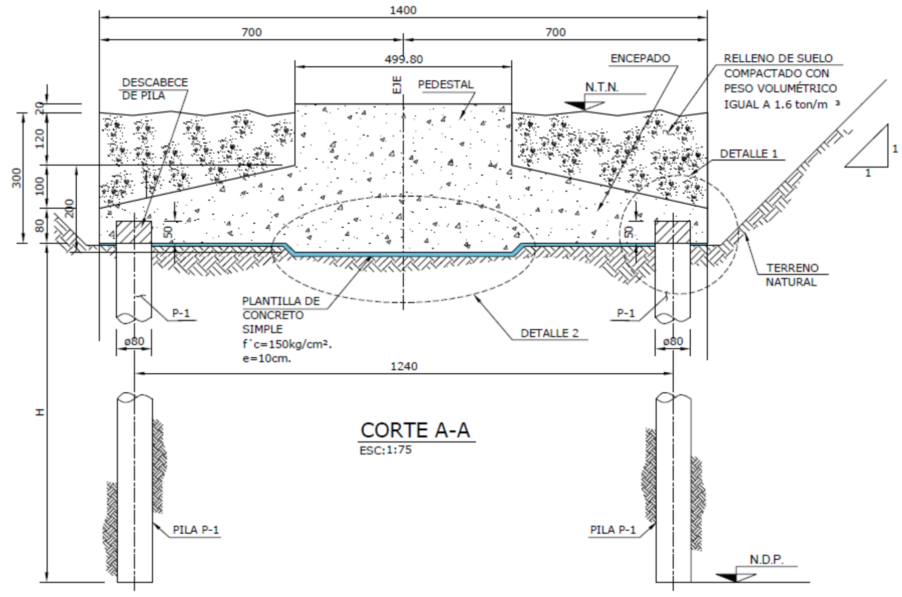
Applications

Typical Wind Farm Application

Shallow foundations on rigid inclusions

Deep foundations

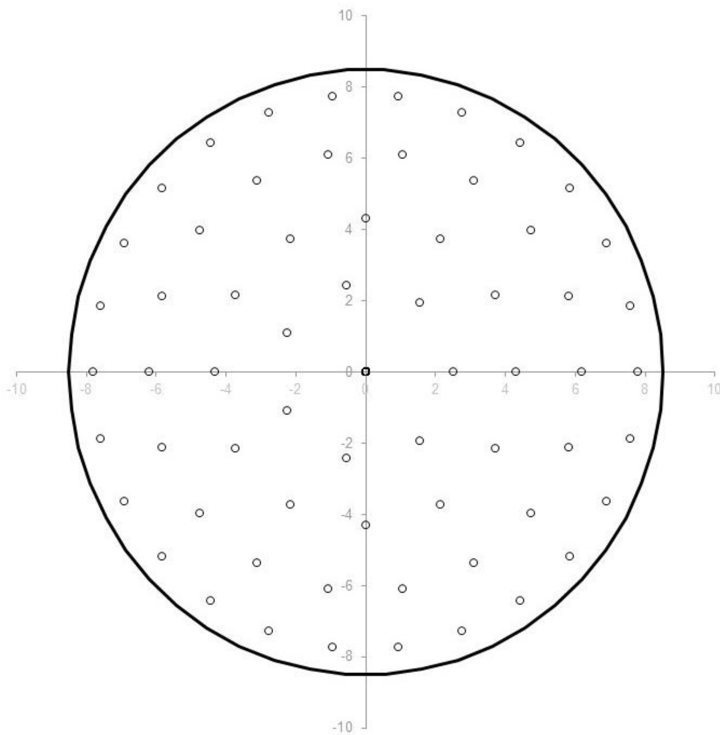


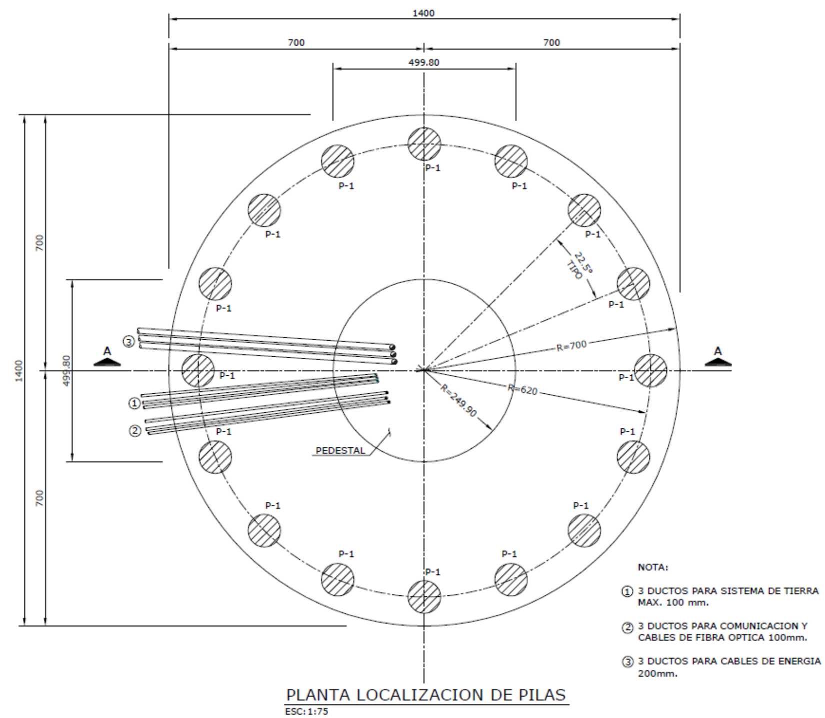


Typical Wind Farm Application

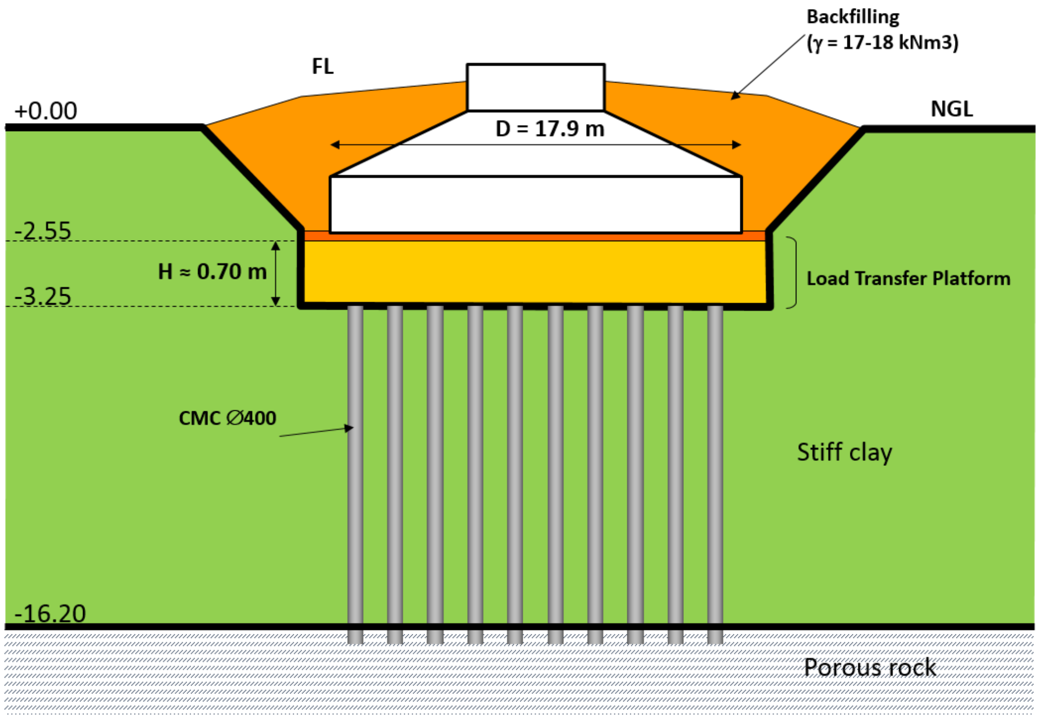
Shallow foundations on rigid inclusions

Deep foundations





Typical Wind Farm Application



Typical Wind Farm Application

Applications


Tanks, silos, wastewater treatment plant


Road and railway embankments


Wind turbines


Industrial warehouses and factories

Warehouses/Factories in Asia

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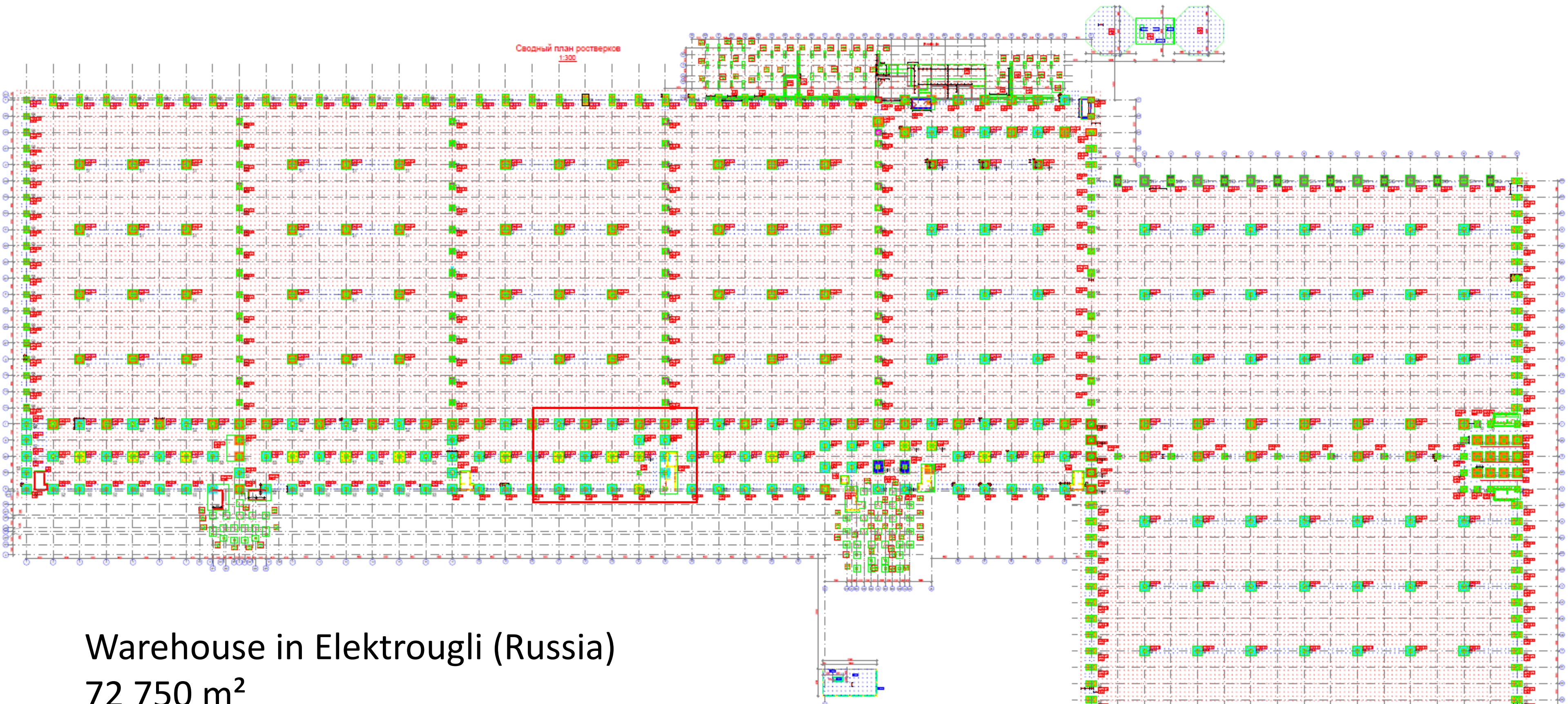
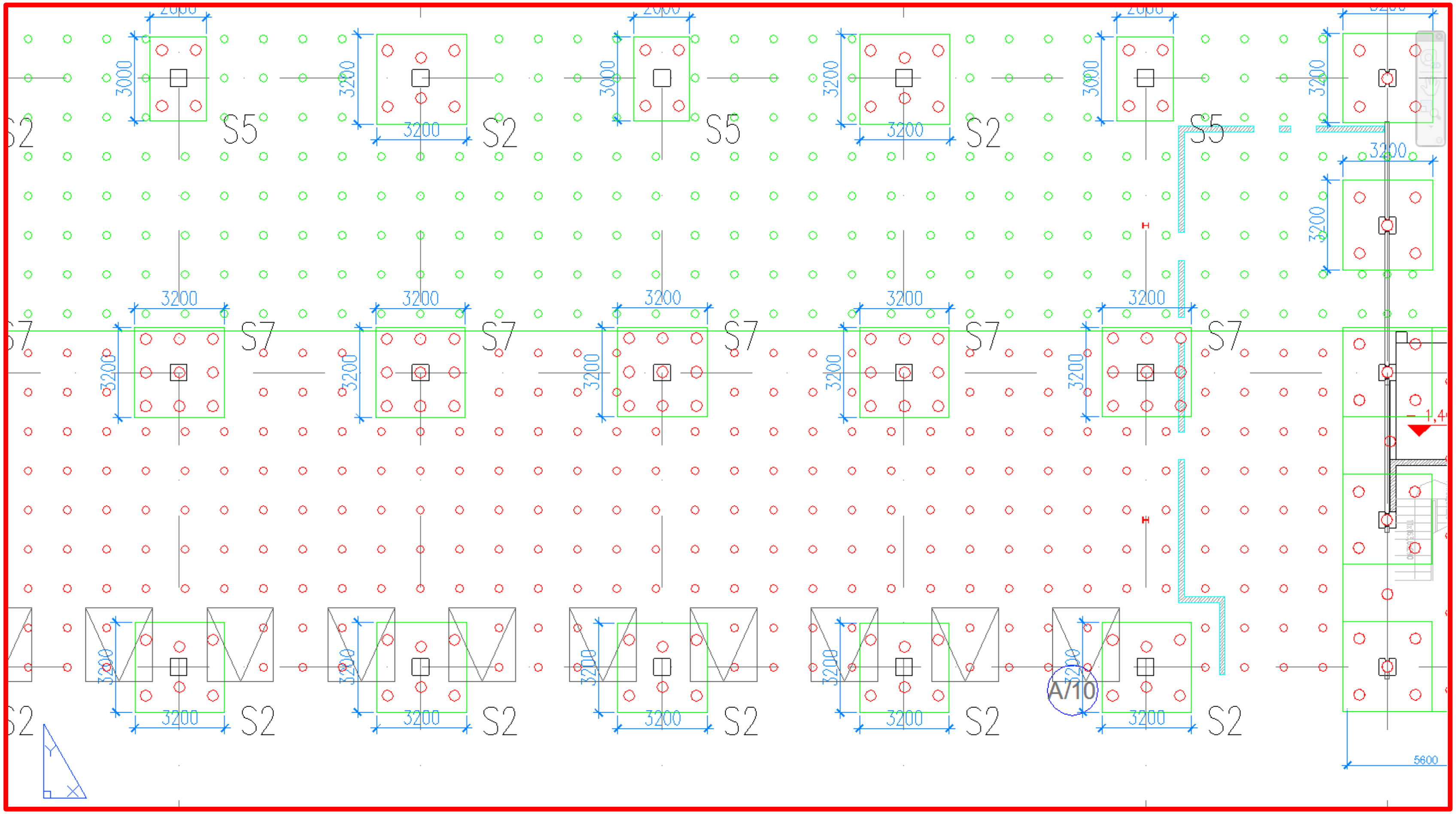
- BioXcell (Malaysia): 2,500 sqm
- Garuda Maintenance Facilities (Indonesia): 30,365 sqm
- Saigon Shipyard (Vietnam): 50,000 sqm

Warehouses/Factories – Typical Layout

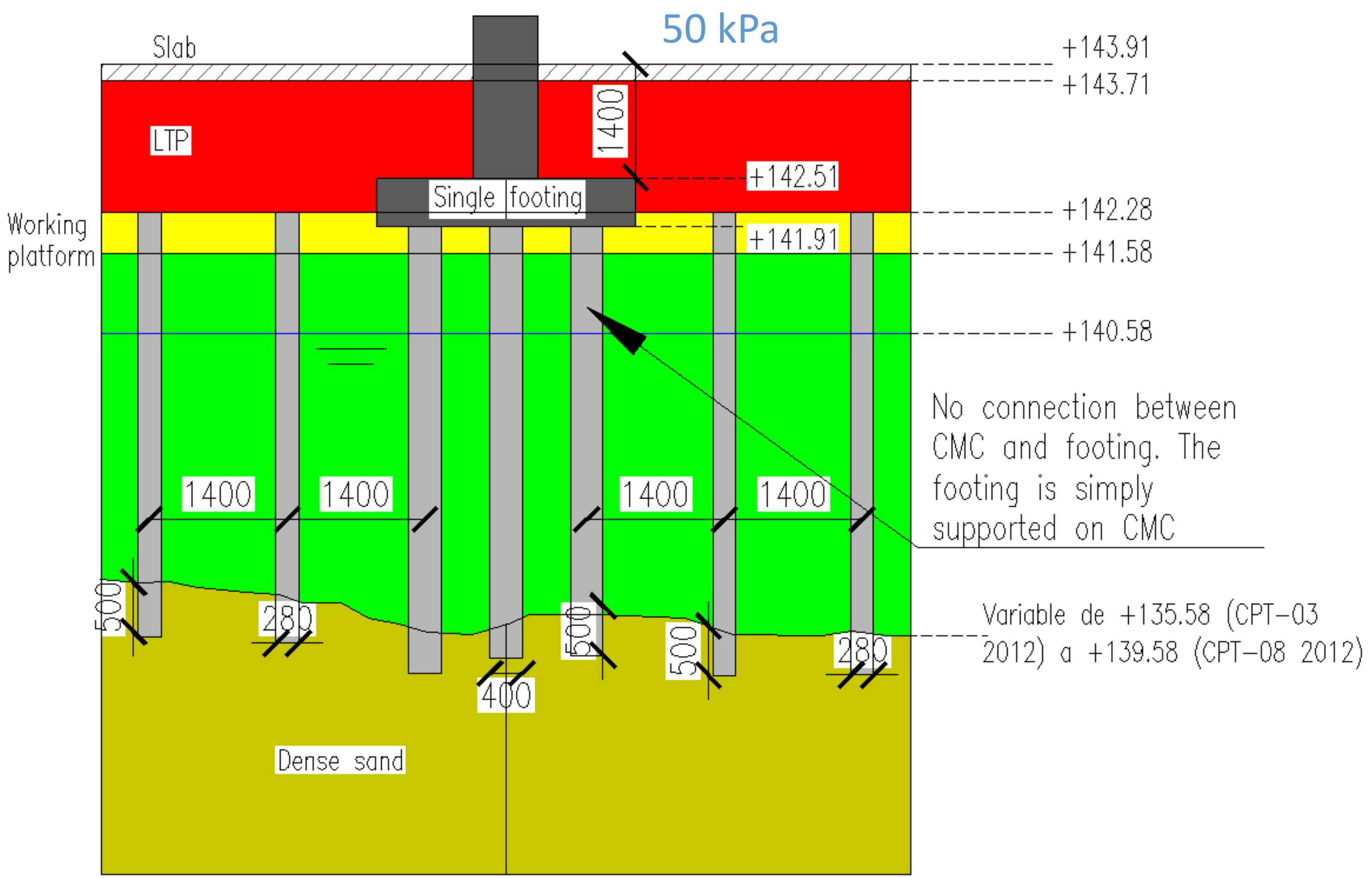


Warehouse in Elektrougli (Russia)
72 750 m²
33 854 CMC ∅280 under slab-on-grade
2 746 CMC ∅420 under isolated footings

Warehouses/Factories – Typical Layout



Warehouses/Factories – Typical X-sec



Concluding Remarks

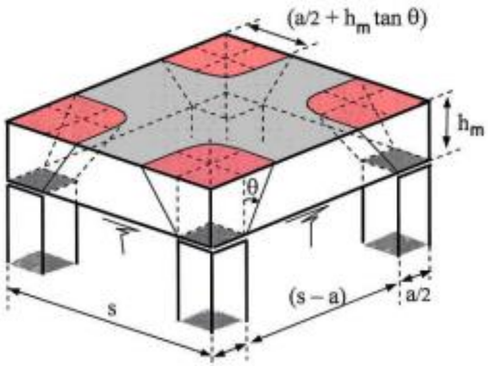


Figure 2.29: The proposed load transfer mechanisms.

Based on the proposed geometry (Fig. 2.29), a determination can now be made of the share of surcharge Q_p and weight W_p of the granular layer being redirected towards the inclusions via the load transfer zone. Both the efficiency E and capacity G of the granular transfer platform relaying surcharges towards the inclusions can also be deduced in the case of square-section inclusions or pile caps.

$$Q_p = q\left(a^2 + 4\,a\,h_m \tan\theta + \pi\,h_m^2 \tan^2\theta\right) \quad \text{for } h_m \le h^* = \frac{s-a}{2\tan\theta} \tag{2.17}$$

$$W_p = \gamma\left[a^2\,h_m + 2\,a\,h_m^2 \tan\theta + \frac{\pi}{3} h_m^3 \tan^2\theta\right] \quad \text{for } h_m \le h^* = \frac{s-a}{2\tan\theta} \tag{2.18}$$

$$E = \frac{W_p + Q_p}{s^2(\gamma\ h_m + q)} \quad \text{and} \quad G = \frac{\left(a^2 + 4\,a\,h_m \tan\theta + \pi\ h_m^2 \tan^2\theta\right)}{s^2} \tag{2.19}$$

For circular-section inclusions or pile caps, with diameter *a*, the following are obtained:

$$W_p = \frac{\gamma\,\pi\,h_m}{3}\left[(a/2)^2 + (a/2 + h_m \tan\theta)^2 + a\,(a/2 + h_m \tan\theta)/2\right] \tag{2.20}$$

$$Q_p = q\,\pi\,(a/2 + h_m \tan\theta)^2 \quad \text{for } h_m \le h^* = \frac{s-a}{2\tan\theta} \tag{2.21}$$

$$E = \frac{W_p + Q_p}{s^2(\gamma\ h_m + q)} \quad \text{and} \quad G = \frac{\pi\,(a/2 + h_m \tan\theta)^2}{s^2} \tag{2.22}$$

THANK YOU
for your kind attention

Q&A?

Richard Ong
Regional Technical Director
richard@menard-asia.com

MENARD ASIA

19th Southeast Asian Geotechnical Conference & 2nd AGSSEA Conference Kuala Lumpur 31 May – 3 June 2016

Design and Construction of Ground Improvements to Mitigate Liquefaction – A Case History from MacKay's to Peka Peka Expressway, New Zealand

Presentation by

Philip Robins and Tim Pervan

Project Overview – M2PP

MacKay's to Peka Peka Expressway

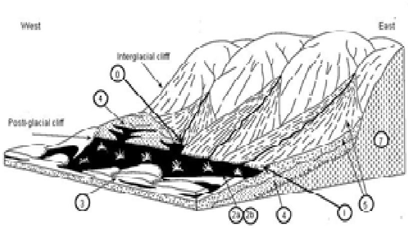
- 18km Long 4 lane expressway
- 15 road bridges the longest being 250m across the Waikanae River
- 2 significant pedestrian overpass bridges
- Project Value = $600M NZD

Presentation Overview

1. Regional Geology and Seismicity
2. Design Philosophy
3. Discussion on the ground improvement techniques used
   – Vibro Stone Columns
   – Concrete Lattice
   – Vibro Compaction & Dynamic Compaction
4. Conclusions

Geology and Seismicity

- Design PGA up to **0.98g** in a 1/2500 year ULS event.
- Extensive sections underlain by potentially liquefiable dune sands and silts



Geology and Seismicity

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- Approximately 50% of the earthwork footprint is underlain by peat.
- Ground water levels are near surface

Design Philosophy

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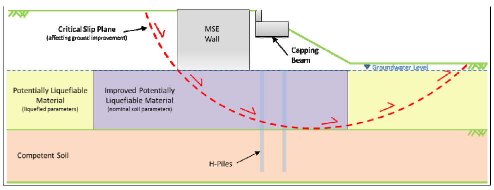
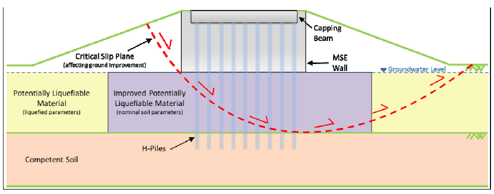
- Displacement based design adopted for all bridges
- Ground Improvement used under bridge abutments to control seismically induced displacements of bridge abutment soil.
- Displacements typically between 100mm – 200mm.





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Design Philosophy

- Ground Improvement Methods – Considered at Design Stage
  - Gravel (stone) Columns
  - Concrete / *In situ* mix lattice
  - Vibro Densification
  - Dynamic Compaction

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Stone Columns

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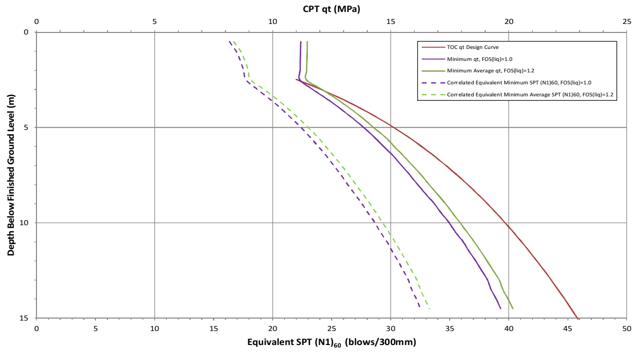
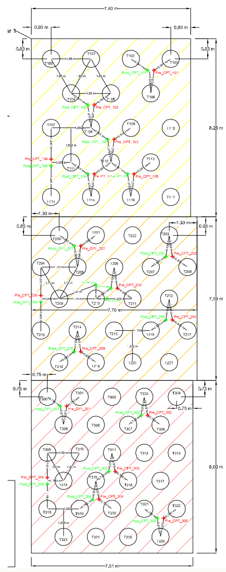
Design Philosophy

- Stone Columns & Vibro Densification
  - Improve sandy or gravelly soils
  - Intent was to densify surrounding soils – increasing horizontal effective stress
  - Shear stiffening and drainage effects not incorporated
  - Verification using CPT qt curves based on FOSliq = 1.0 and 1.2
  - Curves based on 2500 year return period



Stone Columns

- Construction method
  - Top feed columns using wet flush method
  - 65/40 gap graded ballast
  - 2000m3 U bend pounds
- Column Size and Spacing
  - Design - 600mm diameter columns at a 35% replacement ratio
  - Trialed varying spacing's (1.85m, 1.7m, 1.5m)
  - Final - 900mm diameter columns at 32.6% replacement ratio



Stone Columns

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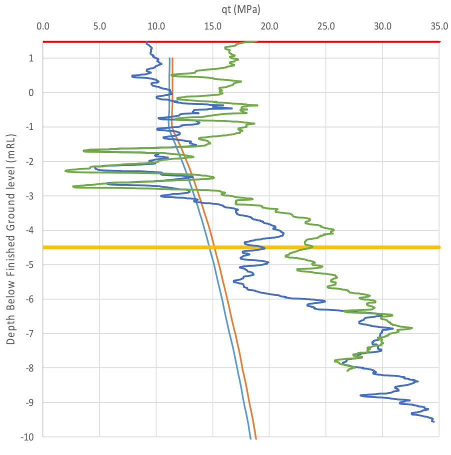
- Results
  - Good response in sands and gravels
  - Poor response in silt layers
  - Column Diameter varied between 900mm and 1100mm



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Concrete Lattice

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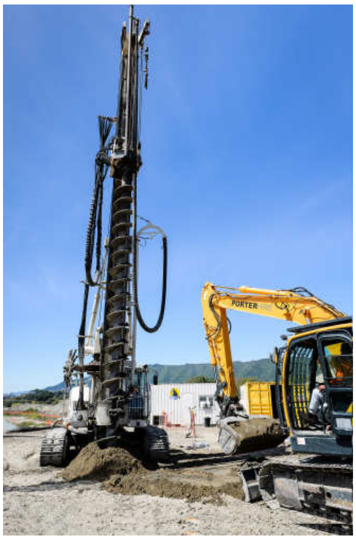

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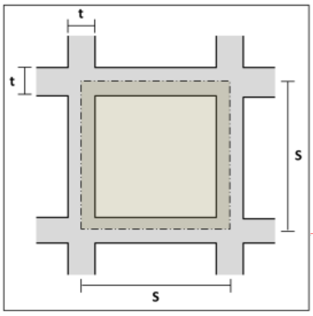
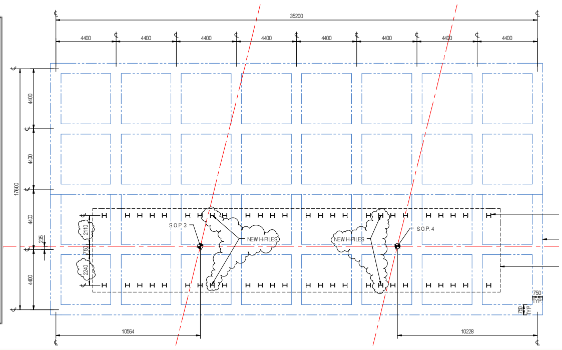
Design Philosophy

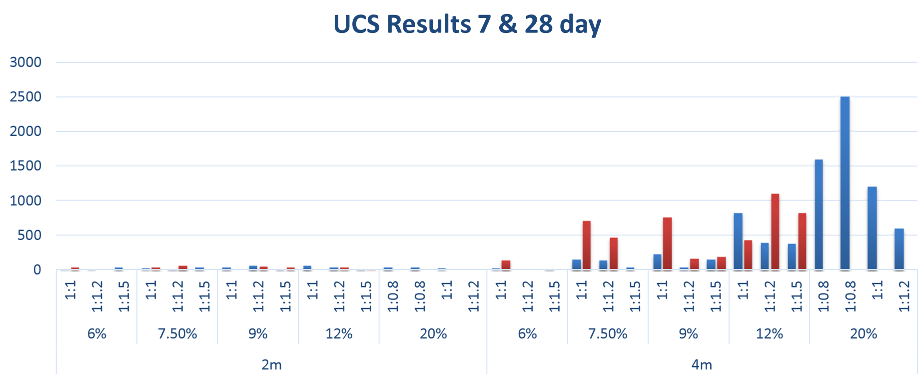
- Concrete Lattice
  - Sites containing silty soils unsuitable for improvement by stone columns
  - Improvement by increasing the cyclic stiffness in composite action.
  - Design based on the stress re-distribution method – Baez & Martin (1994)
  - OTREC (2013) design approach used applying a reduction factor to account for partial shear and flexural behavior considering all lattice elements.
  - Verification of lattice strength by converting G_{lat} – E_{mod} – UCS for on site testing





Concrete Lattice

- Initial design requirements
  - Shear stiffness = 400kPa
  - Elastic Modulus = 960kPa
  - UCS = 1200kPa
- Soil Mixing Laboratory Trials
  - Mix ratio's = 6%, 9%, 12%, 20% by weight
  - W/C Ratios = 1, 1.2, 1.5
  - Sand samples taken from 2m & 4m deep.



Vibro Compaction & DC

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Vibro Compaction

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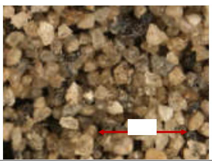
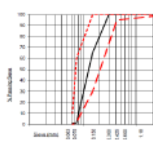
- Vibro Compaction Trials did not work
  - Sand failed to migrate down towards the probe tip

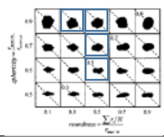
Paetawa Sand Fill (VC)

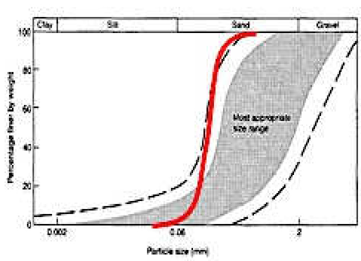




Passing 0.15mm = 65%
Passing 0.063mm (Silt) = 1%
Uniformity Coefficient (D60/D10) = 1.49

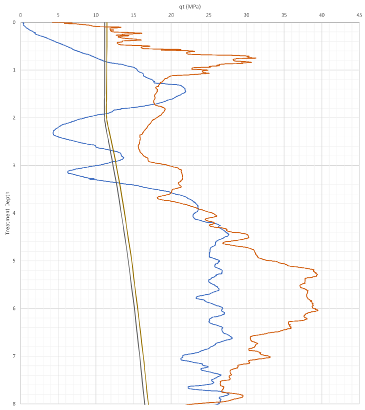
Paetawa Sand Fill (VC)





Dynamic Compaction

- Treatment depth varied between 5-8m
- Average Energy applied was between 150-200t-m/m2
- 5 x 5m grid layout – completed over 4 passes



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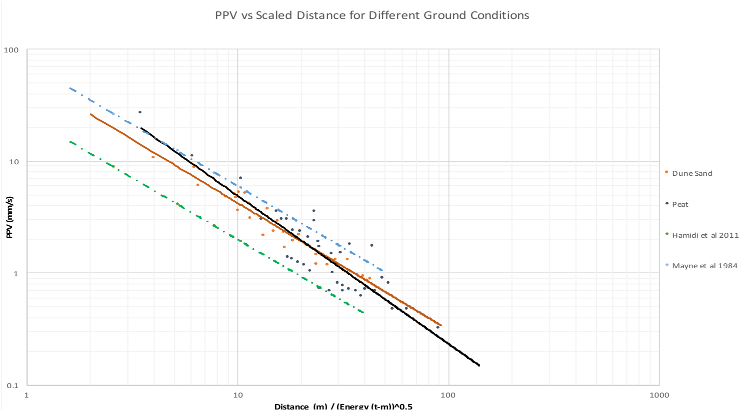
Dynamic Compaction

- Environmental Considerations
  - Regression curves generated in both sands and peat
  - Dominant frequency varied between 7Hz – 17Hz
  - With 156t-m drop energy - PPV was around 5mm/sec at 80-100m away



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Conclusions

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- Ground Improvements were required to mitigate the effects of liquefaction under bridge abutments
- Wind blown Holocene dune sands provided additional complexity when dealing with soil mixing and vibro compaction options.
- DC was an effective method of compacting sand and treating minor silt layers – associated vibrations were consistent with previous research.

Acknowledgements

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KELLER

Fletcher

MacKays to Peka Peka

NZ TRANSPORT AGENCY WAKA KOTAHI | Fletcher | HIGGINS | Beca

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Thank You

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