Western

Application of Centrifuge Testing for Sustainable Infrastructure

Prof. Hesham El Naggar, PhD., P.Eng., FEIC, FASCE

Geotechnical Research Centre

May 2, 2019

Advantages of Centrifuge Modeling

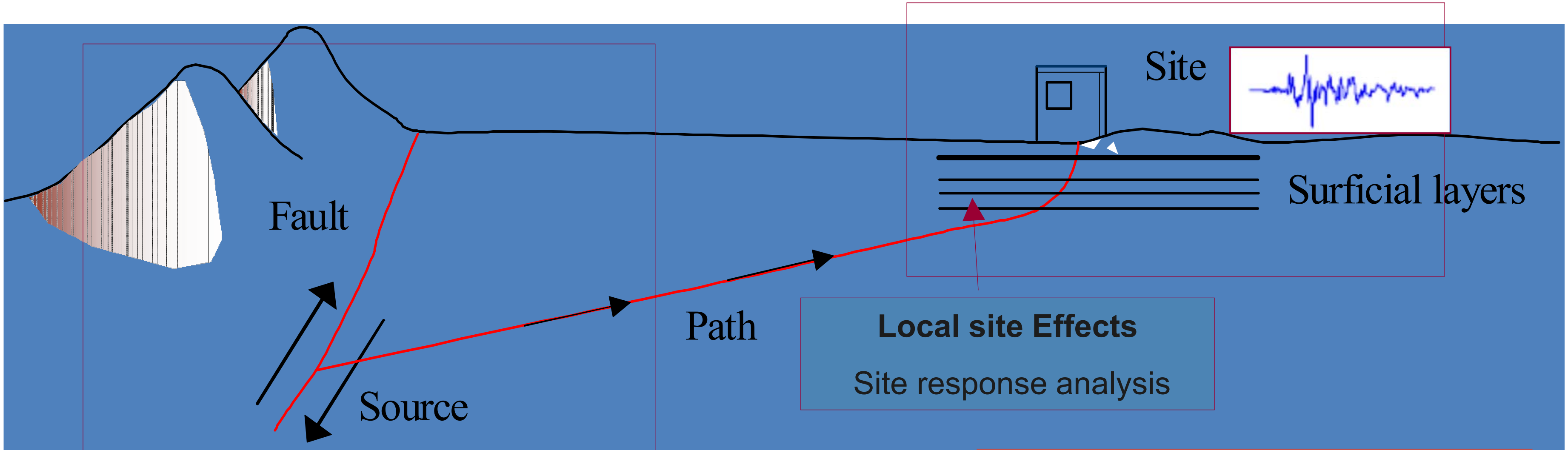
Centrifuge testing offers several advantages:

1. Small cost and fast construction time compared to full scale testing,
2. Allows gathering abundant and reliable information.
3. Enables simulating complex problems and studying the physics involved in these problems (e.g. behaviour of structures resting on or embedded in stratified soils and subjected to earthquakes or waves).
4. Can expedite long term process such as consolidation and low frequency loads.

Western Experience with Centrifuge Testing

- Centrifuge Modeling of Tapered Piles in Sand (Sakr, MESc 1999).
- Investigation on Seismic Site Response and Soil-structure Interaction In Soft Soils (Rayhani, PhD. 2007).
- Static and Seismic Soil Culvert Interaction (Abuhajar, PhD 2013)
- Performance Of Micropiled Raft In Sand And Clay–centrifuge And Numerical Studies (Alnauim, PhD 2014).
- Hybrid Foundations for Wind Turbines in Cohesive Soil (Alsharedah, PhD on-going)

Investigation on Seismic Site Response and Soil-structure Interaction In Soft Soils



Local site Effects

Site response analysis

Probabilistic seismic hazard analysis with ground motion generation

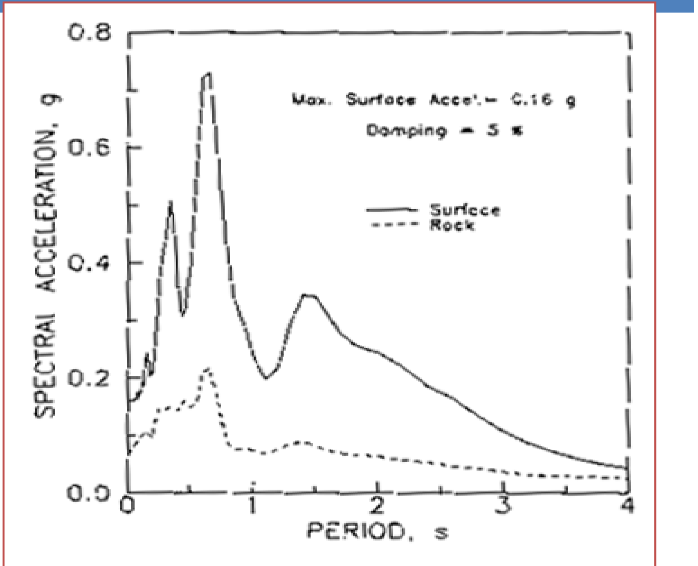


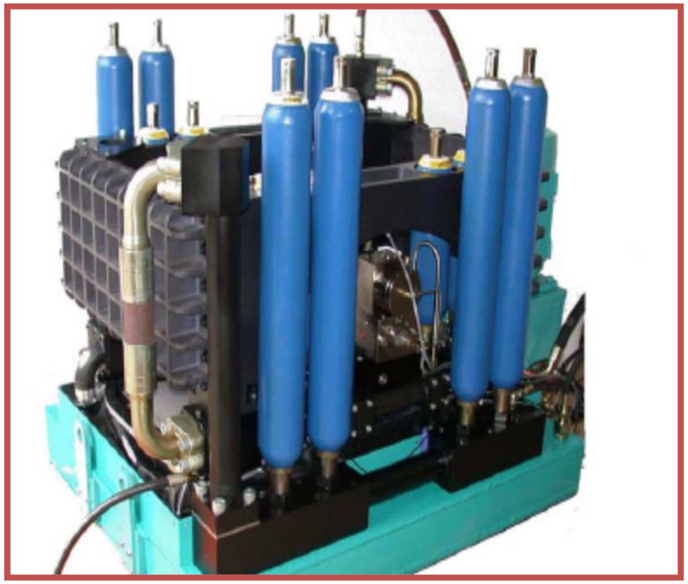
Fig. 5 Response Spectra for Treasure Island and Yerba Buena Island Station Records (90 degree components)

-Previous earthquakes: Kobe (1995), Northridge (1994), and Loma Prieta (1989) highlighted the role of local site conditions on strong motions and associated major damage.

-Mexico City, 1985: Damage patterns demonstrated site effect

Centrifuge Model/Container/Shaker

- Payload (mass) 400 kg
- Container ext. dimensions: 1m x 0.5m x 0.6m
- Max. earthquake acceleration: 40g (50%)
- Max. centrifuge acceleration: 80g
- Max. shaking force: 160 kN
- Material for rings: Aluminum Alloy
- Rubber material : G = 1.06 MN/m2

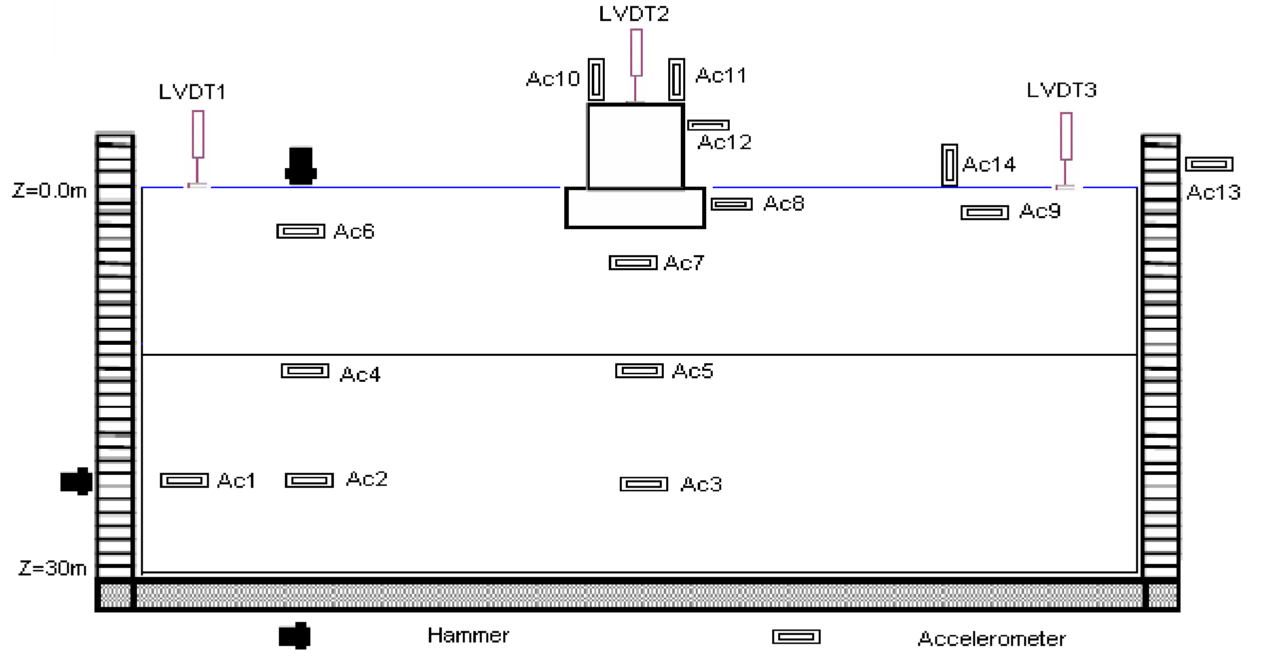


ACTIDYN Earthquake Simulator MODEL QS 67-2
Payload: 1m x 0.5m x 0.6m, 400kg mass

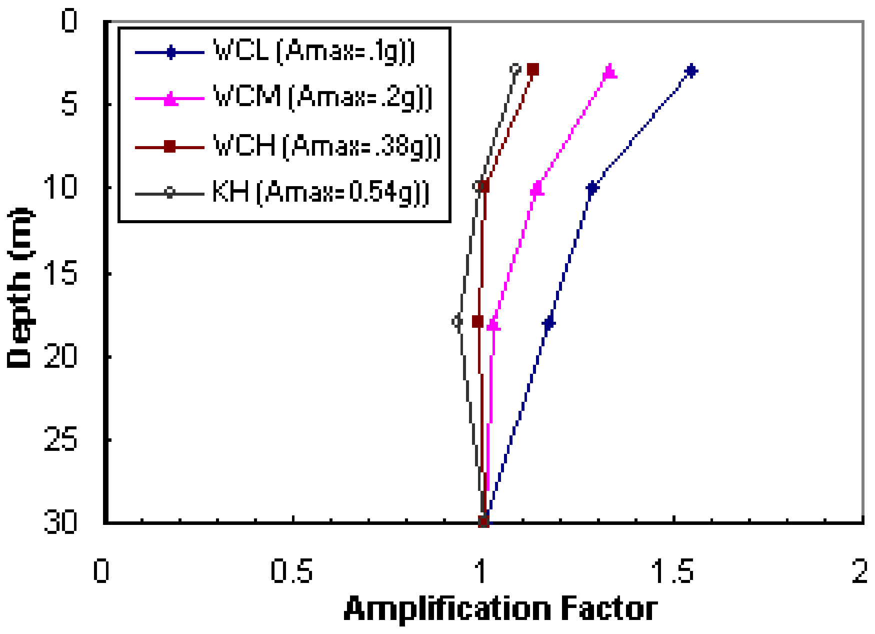
Freq.: 40 -200Hz
Max Force: 220 kN
Max. Disp.: 2.5mm
Max. Velocity: 0.75m/s
Max lat g: 40 –60g
G-field: 10 -80g

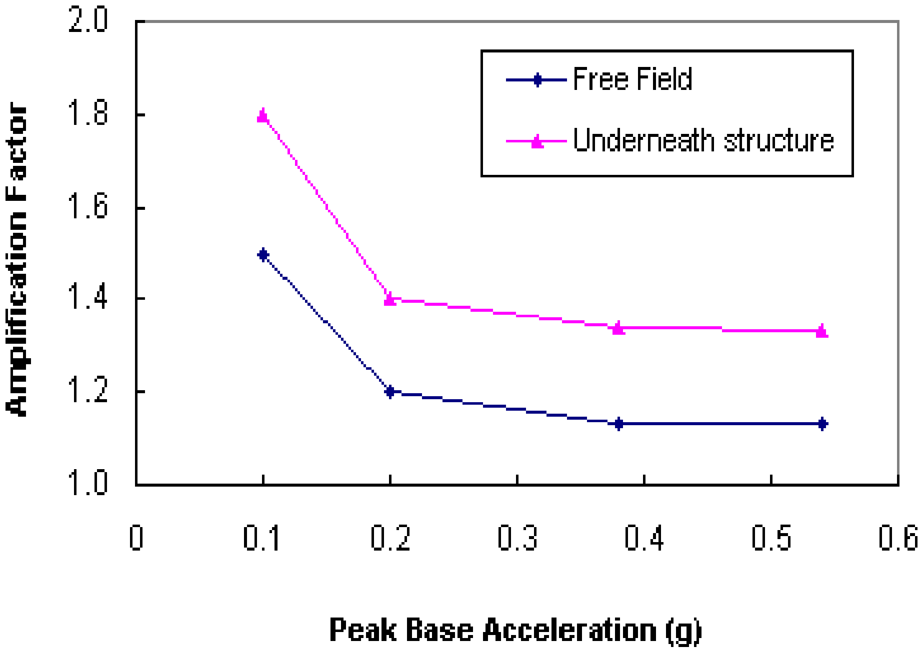
- Pretest on Dummy Sample
- Hammer Test (g levels)
- T-bar and CPT Test (soil strength)
- P-Wave (soil stiffness)
- West Canada Earthquake
- Kobe Earthquake

	Prototype		Centrifuge Test (scale 1:80)	
Input acceleration	**Peak Acc. (g)**	**Dominant Frequency (Hz)**	**Peak Acc. (g)**	**Dominant Frequency (Hz)**
A0.5×WC2475	**0.1**	**0.93**	**8**	**74.5**
A1×WC2475	**0.2**	**0.93**	**16**	**74.5**
A2×WC 2475	**0.38**	**0.93**	**32.5**	**74.5**
Kobe (1996)	**0.54**	**1.85**	**43**	**148**

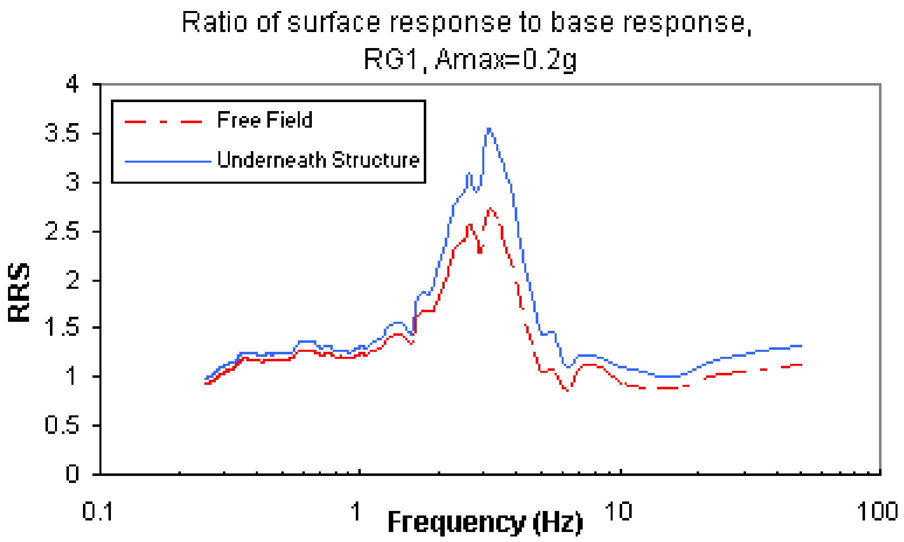


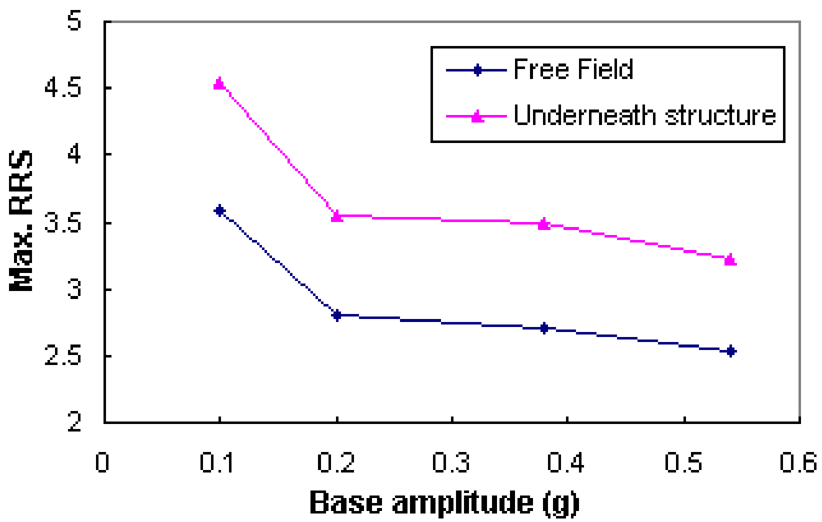
Earthquake Amplification

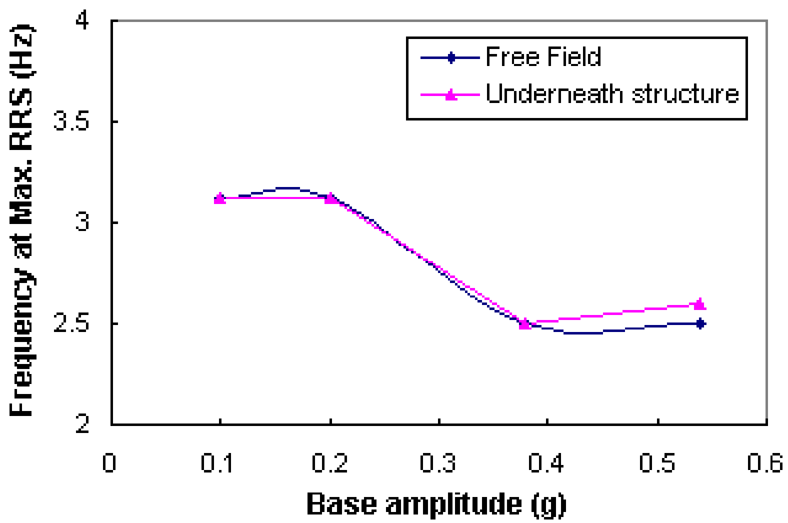




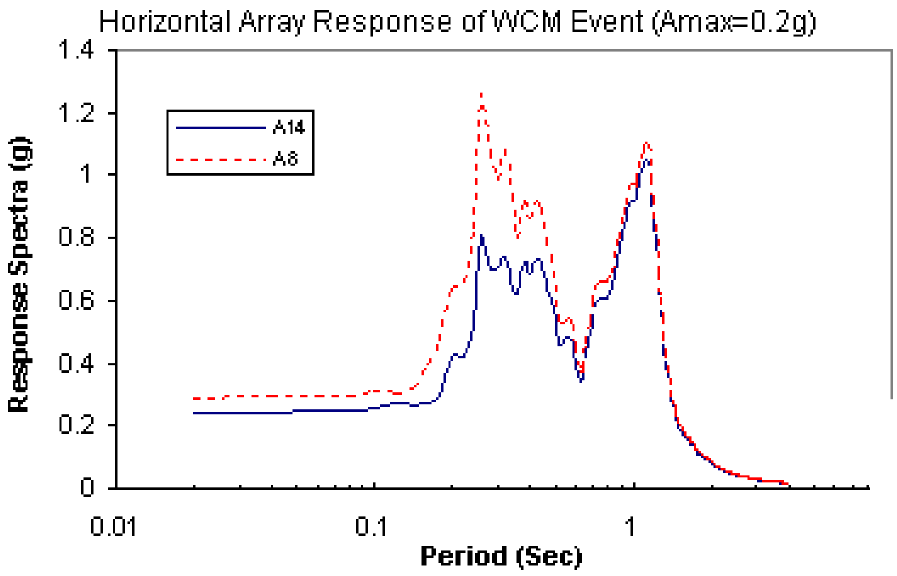
Ratio of Surface to Base Response (RRS)

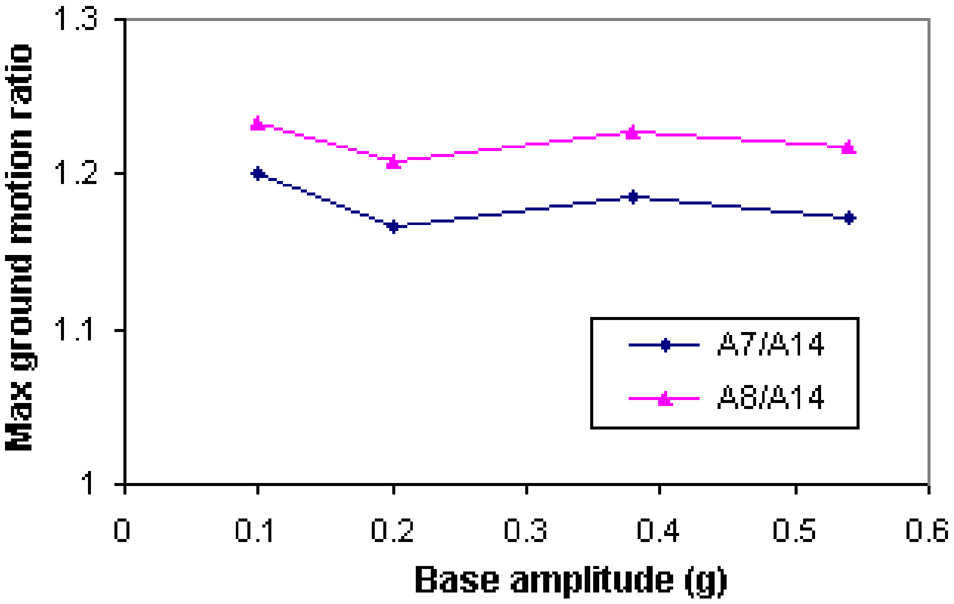

Ratio of surface response to base response,
RG1, Amax=0.2g





Soil-Structure Interaction





References

1. Rayhani, M. and El Naggar, M.H. 2012. Physical and numerical modeling of seismic soil-structure interaction in layered soils. Journal Geotechnical and Geological Engineering, Vol. 30, No. 2, pp. 331-342.
2. Rayhani, M. T. and El Naggar, M.H. 2008. Dynamic Properties of Soft Clay and Loose Sand from Seismic Centrifuge Tests. Journal of Geotechnical and Geological Engineering, Vol. 26, No. 5, pp.593-602.
3. Rayhani, M. T. and El Naggar, M.H. 2008. Numerical modeling of seismic response of rigid foundation on soft soil. International Journal of Geomechanics, ASCE, Volume 8, Issue 6, pp. 336-346.
4. Rayhani, M. T. and El Naggar, M.H. 2008. Seismic Response of Sands in Centrifuge Tests. Canadian Geotechnical Journal, Vol. 45, No. 4, pp. 470-483.
5. Rayhani, M.T., El Naggar, M.H. and Tabatabaei, S.H. 2008. Nonlinear Analysis of Local Site Effects on Seismic Ground Response in the Bam Earthquake. Journal of Geotechnical and Geological Engineering, Vol. 26, No. 1, pp. 91-100.
6. Rayhani, M. T. and El Naggar, M.H. 2008. Characterization of Glyben for Seismic Applications. Geotechnical Testing Journal, ASTM, Vol. 31, No. 1, pp. 24-31.
7. Rayhani, M. T. and El Naggar, M.H. 2007. Centrifuge modeling of seismic response of layered soft clay. Bulletin of Earthquake Engineering, Vol. 5, No. 4, pp. 571-589.

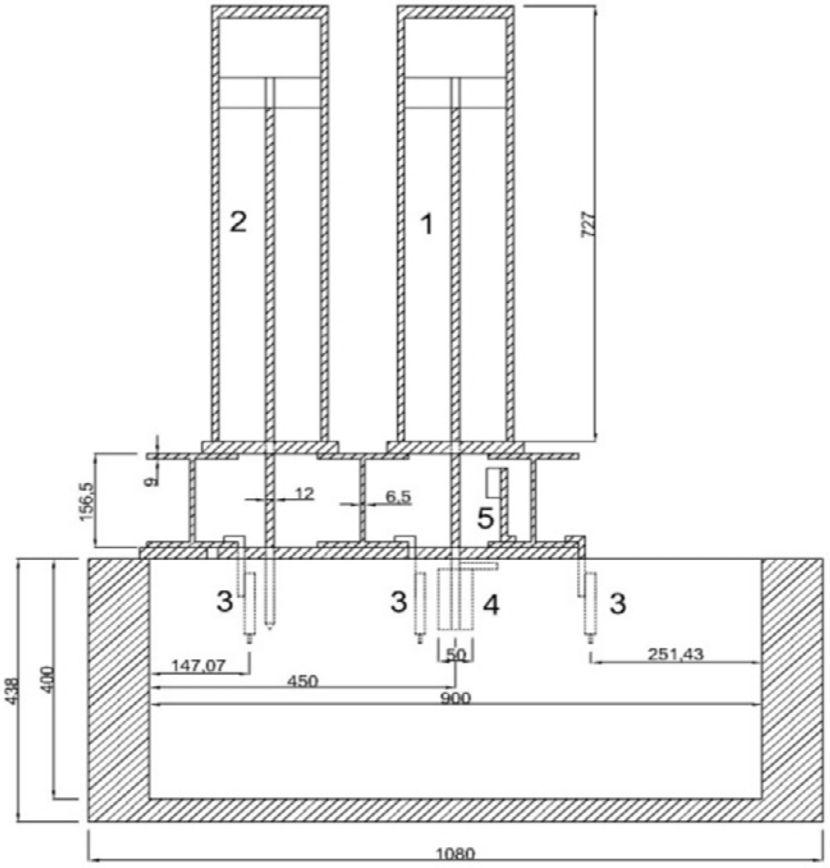
Centrifuge Testing of Piled Rafts

- The centrifuge testing program was carried out at C-Core Centrifuge facility located at Memorial University, St. John's, Newfoundland.
- The tests were conducted under 50g centrifugal acceleration.
- The results of the vertical loading test were used to calibrate the 3D finite element model for the current investigation.

Model dimensions in both model and prototype scales

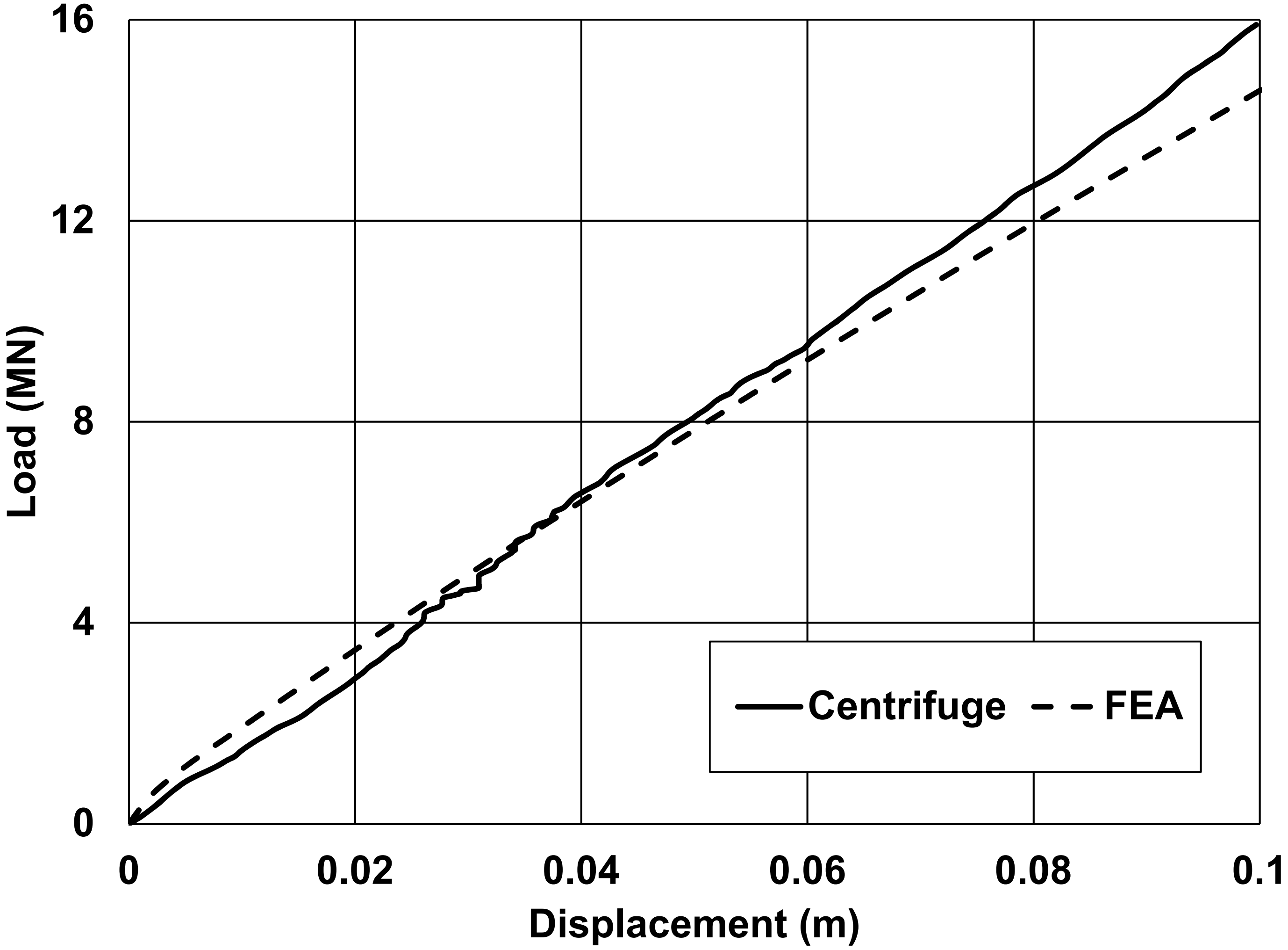
	Symbols	Model	Prototype
Diameter (mm)	D	9.53	150
Materials	-	PVC	Concrete
Pile length	L_p	200 mm	10 m
Modulus of Elasticity	E_o	71 GPa	41.7 GPa
Raft thickness	t	16.4 mm	0.6 m
Raft width (square)	B	105 mm	5.25 m
Number of piles	-	4	4
Axial rigidity	EA	207 kN	517 MN

Centrifuge Model for Piled Raft



Vertical cross-section of centrifuge package: (1) vertical actuator for; (2) sand cone for CPT; (3) LVDTs; (4) load cell; and (5) laser (all dimensions in mm).

3D Finite Element Model and 3D FEM Verification



Comparison of the FEA results with the data obtained from the centrifuge test.

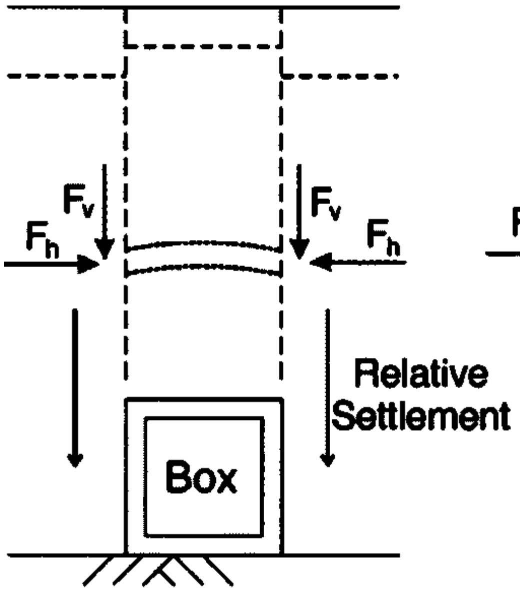
References

1. Alnuaim, A., El Naggar, M.H. and El Naggar, H. 2018. Performance of micropiled rafts in clay: numerical investigation. Computers and Geotechnique, Vol. 99, pp. 42-54.
2. Alnuaim, A.M., Elnaggar, H. and El Naggar, M.H. 2017 Evaluation of piled raft Performance using a verified 3D nonlinear numerical model. Journal of Geotechnical and Geological Engineering. Vol 35, pp. 1831-1845; DOI 10.1007/s10706-017-0212-1.
3. Alnuaim, A., El Naggar, M.H. and El Naggar, M.H. 2016. Numerical investigation of the performance of micropiled rafts in sand. Computers and Geotechnics, Vol. 77, pp. 91-105..
4. Alnuaim, A., El Naggar, M.H. and El Naggar, H. 2015.Performance of micropiled raft in clay subjected to vertical concentrated load: centrifuge modeling. Canadian Geotechnical Journal, 52(12): 2017-2029.
5. Alnuaim, A., El Naggar, H., and El Naggar, M.H. 2015. Performance of micropiled raft in sand subjected to vertical concentrated load: centrifuge modeling, Canadian Geotechnical Journal, Vol. 52, No. 1, pp. 32-45.

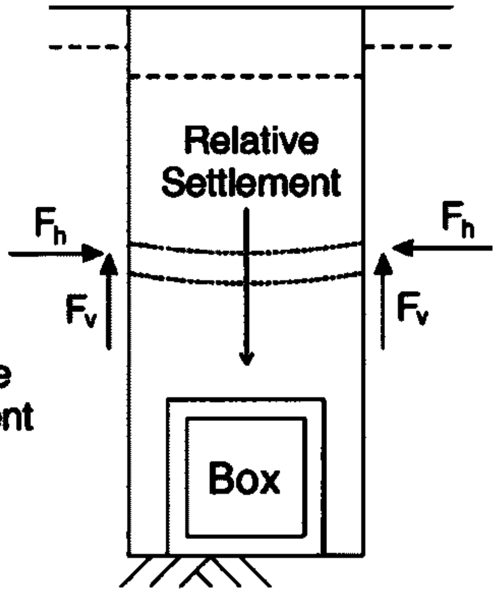
Static and Seismic Soil-Culvert Interaction

Construction of Box Culverts

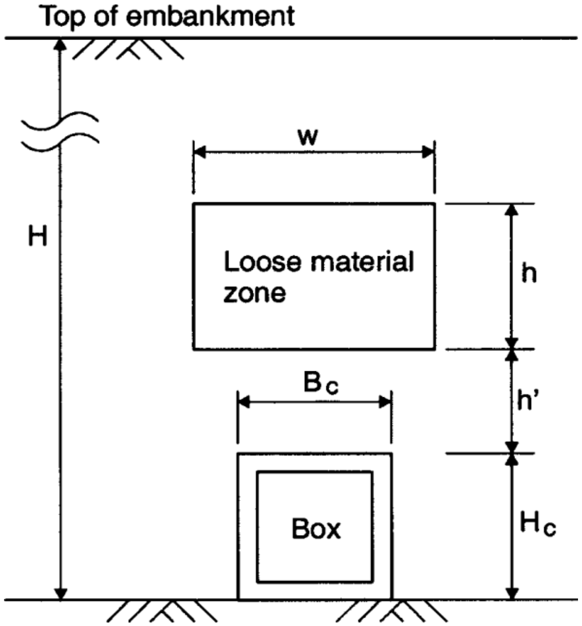
Embankment



Trench



Imperfect Trench



Kim and Yoo (2005)

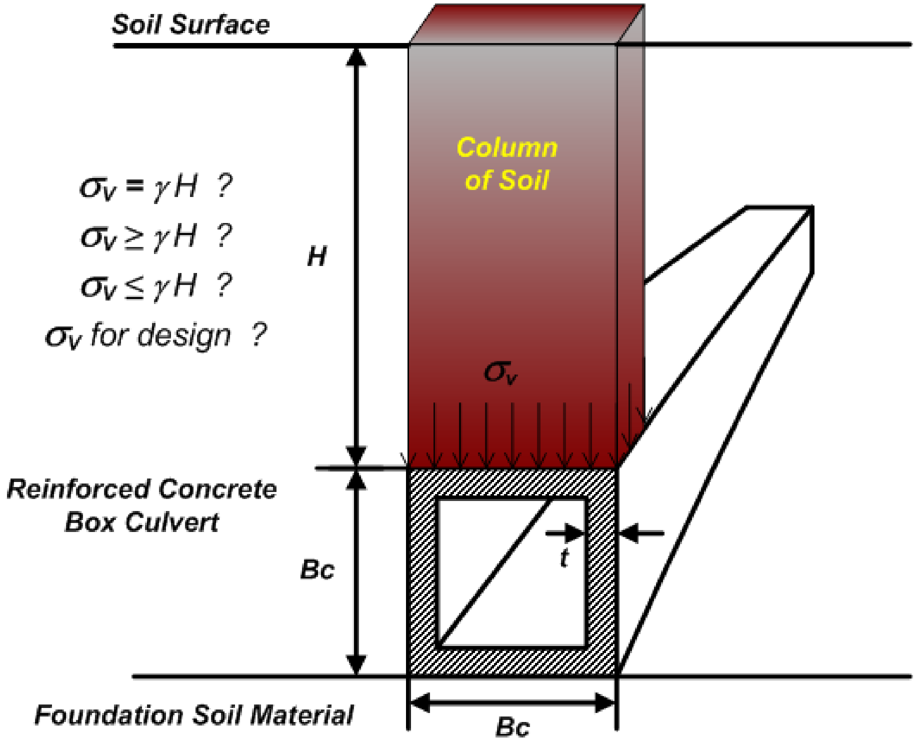
Static Earth Pressure

$\sigma_V = \gamma H$?

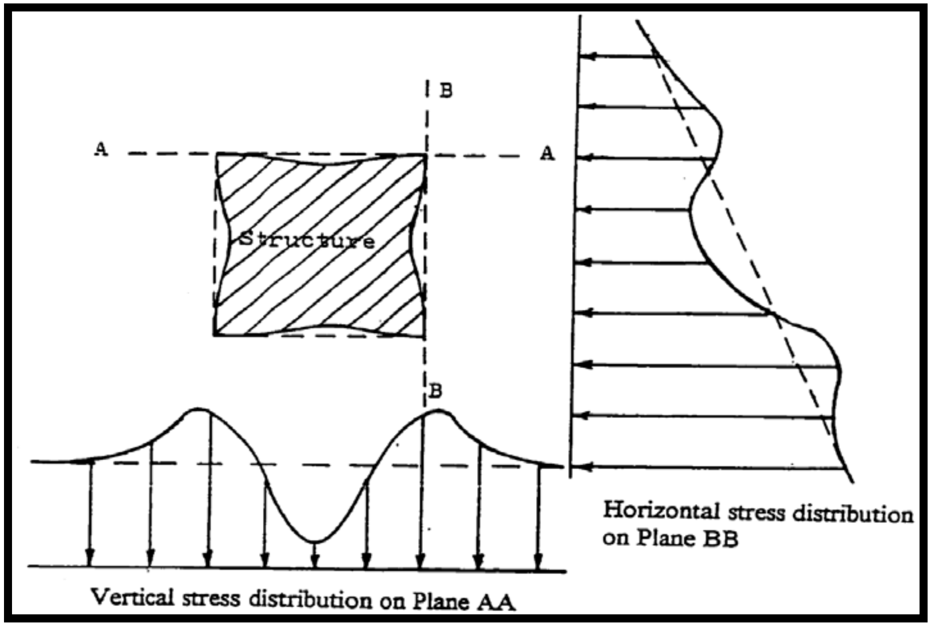
$\sigma_V \geq \gamma H$?

$\sigma_V \leq \gamma H$?

σ_V for design ?



Soil Arching Effect



Evan (1984)

$$\text{Soil Culvert Interaction Factors } (F_e) = \frac{\text{Actual Soil Pressure } (\sigma_v \text{ measured})}{\text{Theoretical Soil Pressure } (\sigma_v = \gamma H)}$$

Seismic Analysis of Box Culverts

General Effects of Earthquakes: (Anderson, 2008)

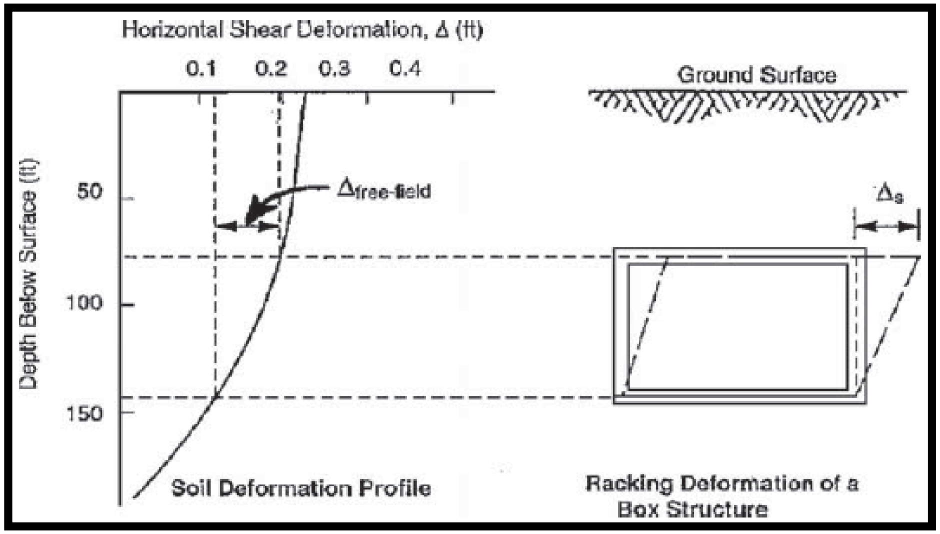
Ground Shaking (Seismic waves) leads to transit ground deformations (TGD)
Ground Failures (lateral spreading, liquefaction, etc) leads to permanent ground deformation (PGD)

Racking deformations of box culverts (Wang, 1993)

Δ_s is the differential sideways movements between the top and bottom slabs
Imposing Δ_s in simple frame analysis can give the required internal forces

AASHTO: No seismic analysis provided

CHBDC: The seismic bending moment is equal to the static bending moment times the vertical component of the PGA (Av)



Wang (1993), Anderson et al. (2008)

Soil-Box Culvert Interaction: Research Objectives

1. Evaluate arching effect of soil around box culverts under static and seismic loading (soil density, surface foundation and culvert thickness).

2. Investigate seismic response of box culverts considering earthquake amplitudes and frequency content.

3. Compare effects of static and seismic loading on box culverts.

4. Develop a numerical model to simulate the effect of soil arching by performing static and seismic parametric studies.

5. Develop static and seismic design guidelines for the soil pressures and bending moments on box culverts.

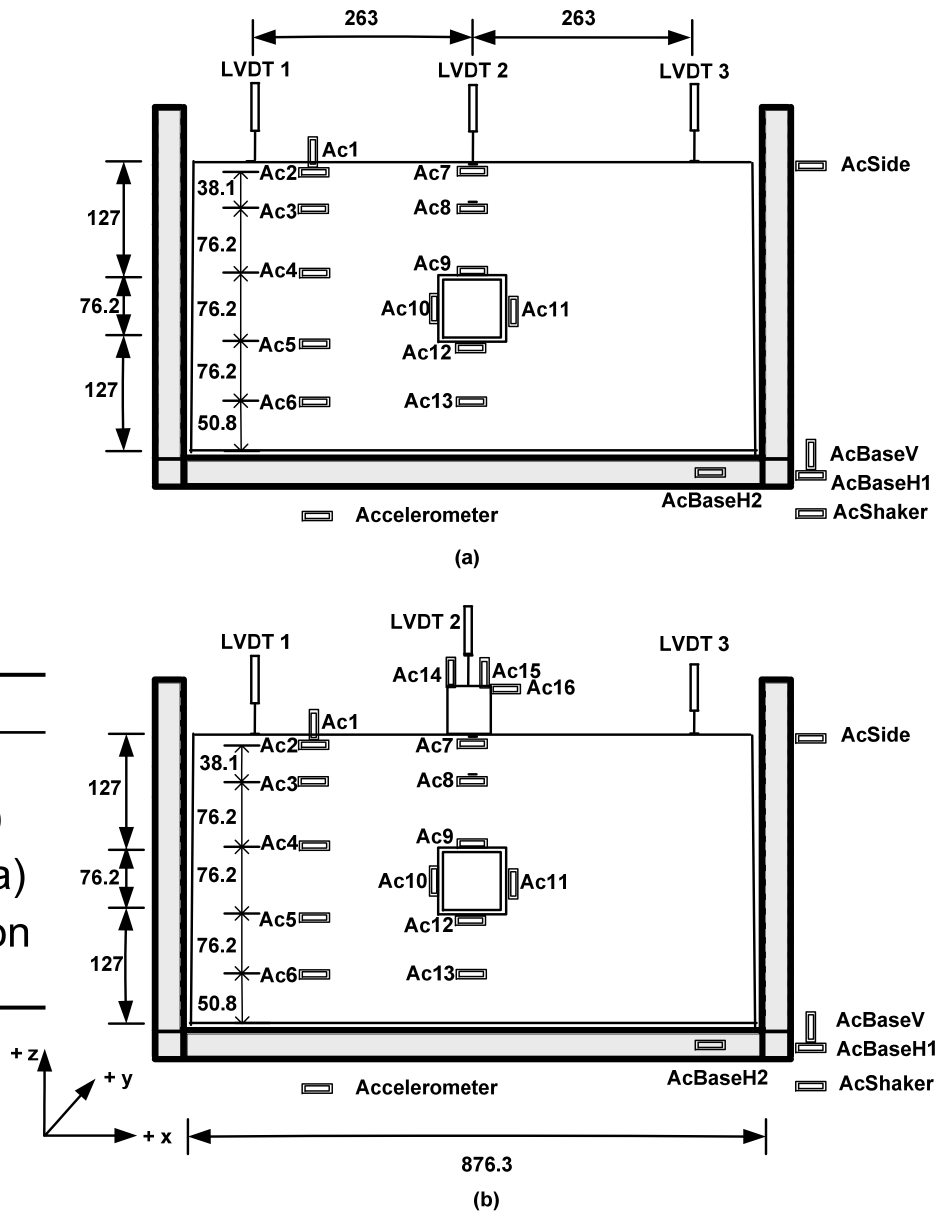
Centrifuge Modeling

Free Field (FF) vs Structural Field (SF)

Test No.	Culvert	Dr(%)
Test 1 (T1)	Thick	90
Test 2 (T2)	Thick	50
Test 3 (T3)	Thin	50
Test 4 (T4)	Thin	90

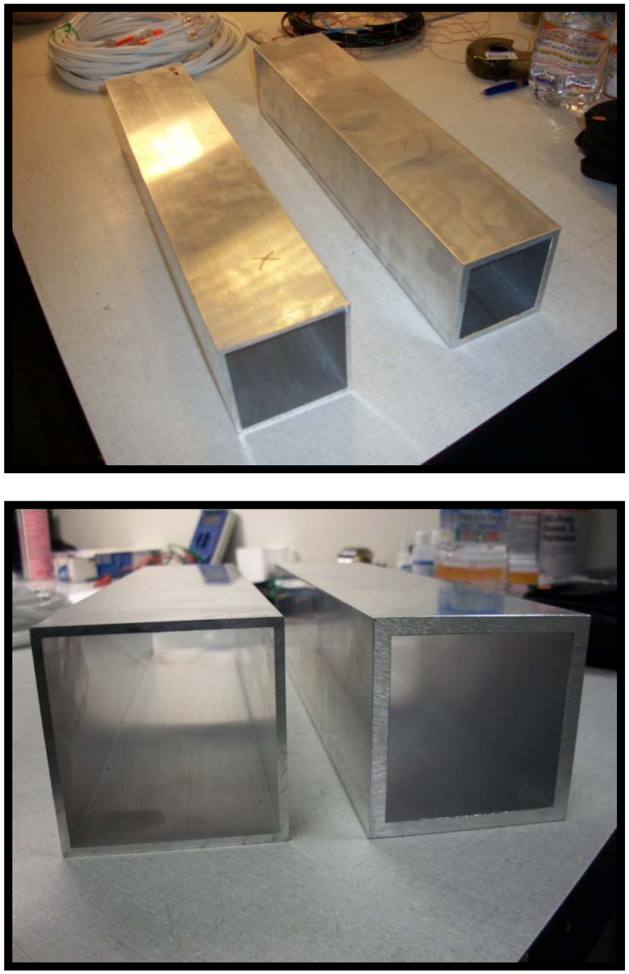
Test No.	Test Case
T1A, T2A, T3A, T4A	Sand surface alone
T1B, T2B, T3B, T4B	Strip foundation (50 kPa)
T1C, T2C, T3C, T4C	Strip foundation (100 kPa)
T3D, T4D	Rectangular foundation (100 kPa)

Soil model is 120-Nevada sand

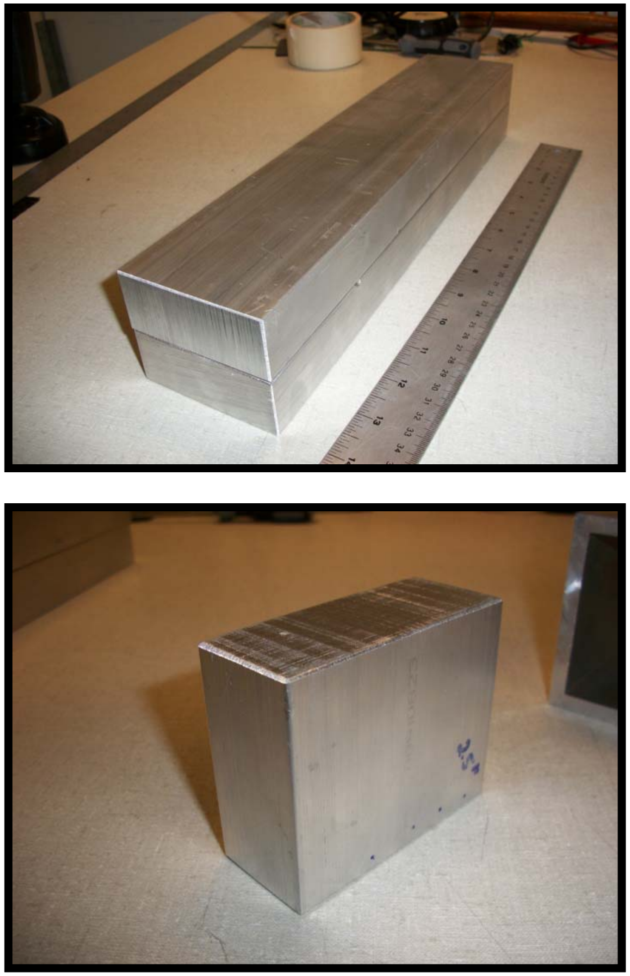


Centrifuge Modeling

Box Culvert model



Strip and Rectangular Foundation models



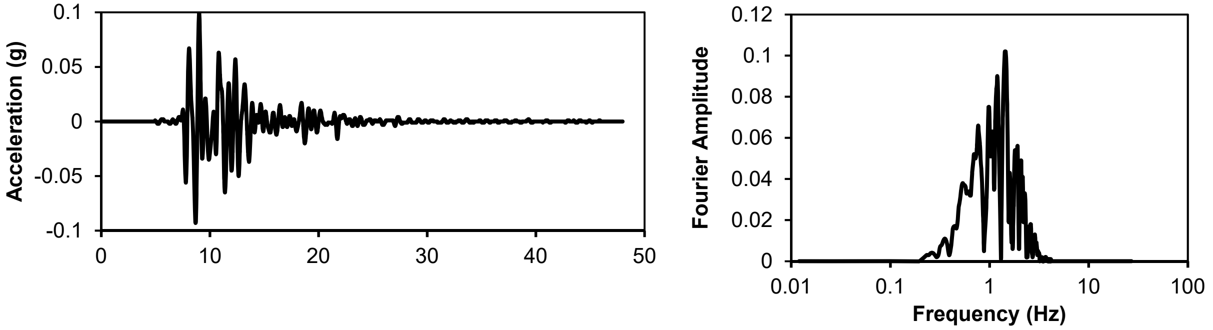
Centrifuge Modeling

Final model test cases

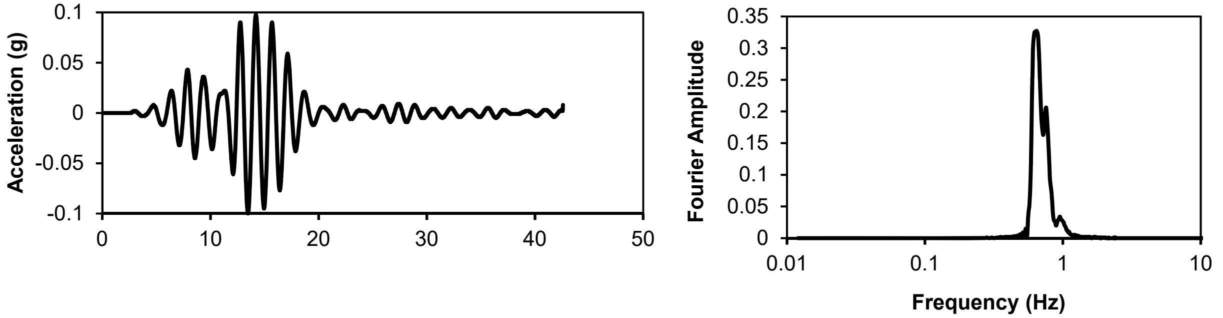
Centrifuge Modeling

Earthquakes

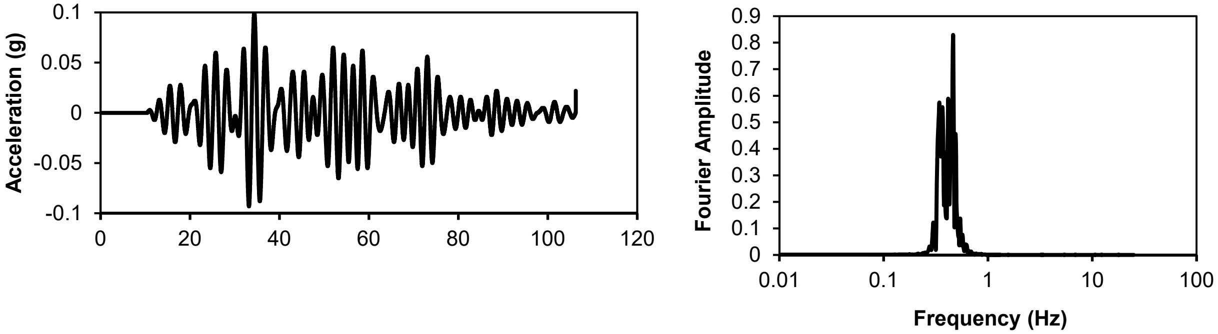
Kobe earthquake,
1.453 Hz



Western Canada,
0.647 Hz



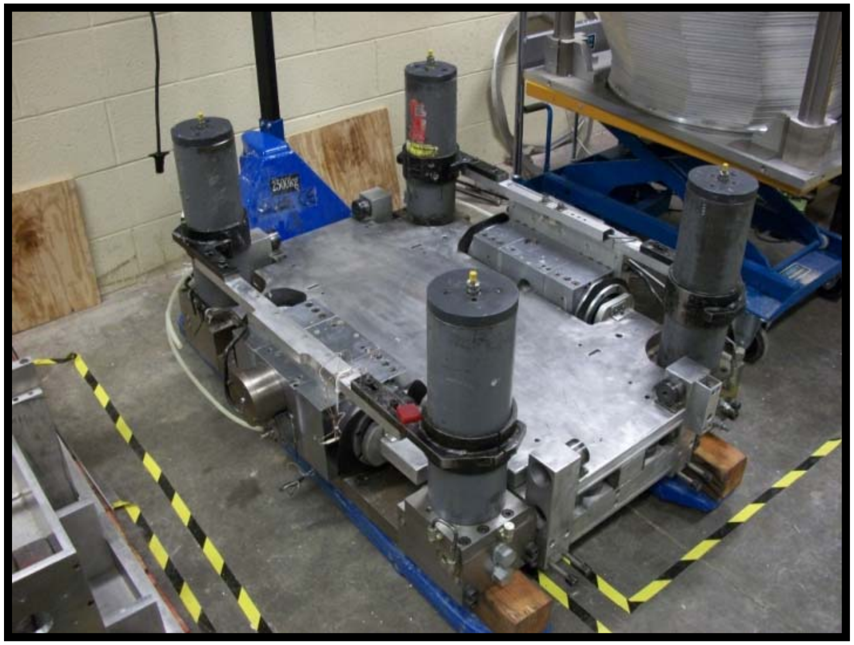
Vancouver Cascadian,
0.464 Hz



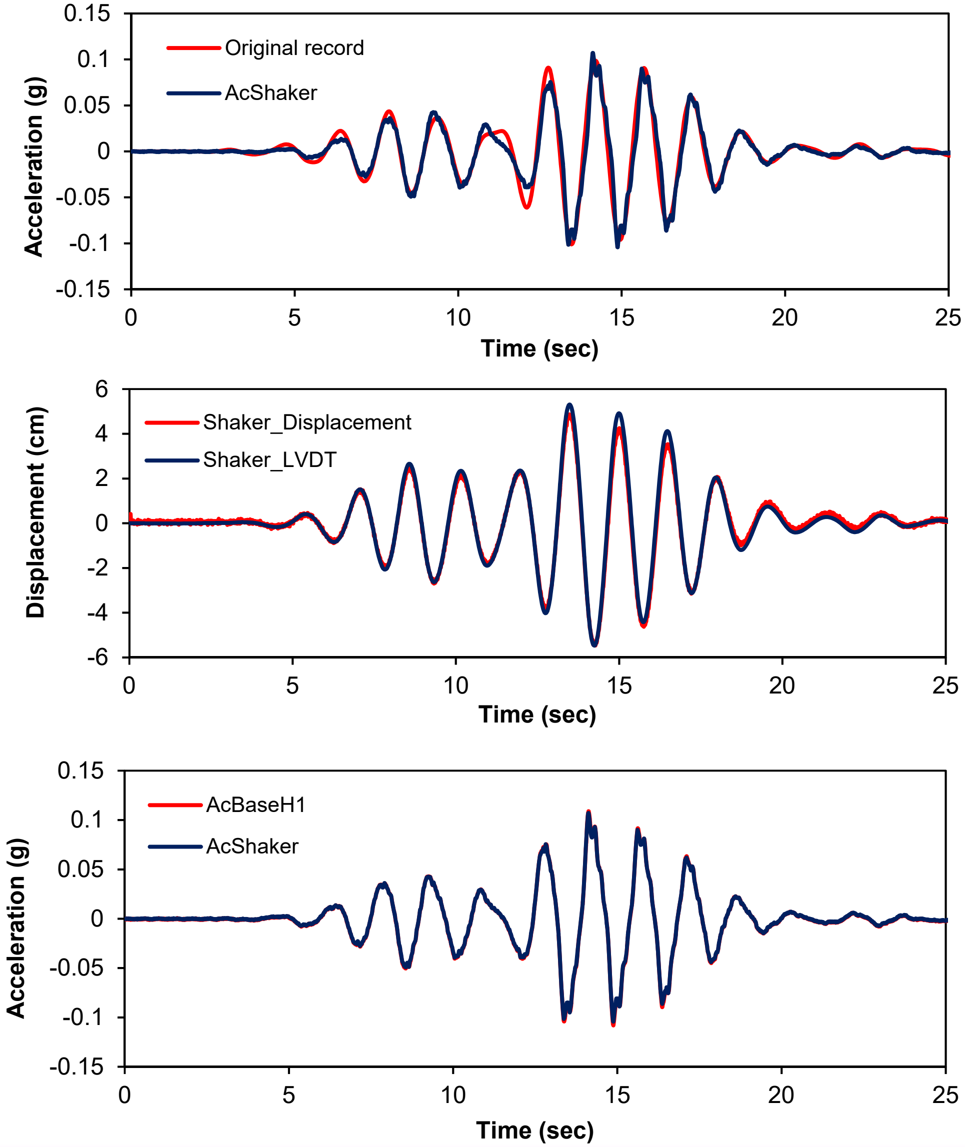
Shakings applied to the Cases A, C, and D of each test

Centrifuge Modeling

Earthquake Simulation

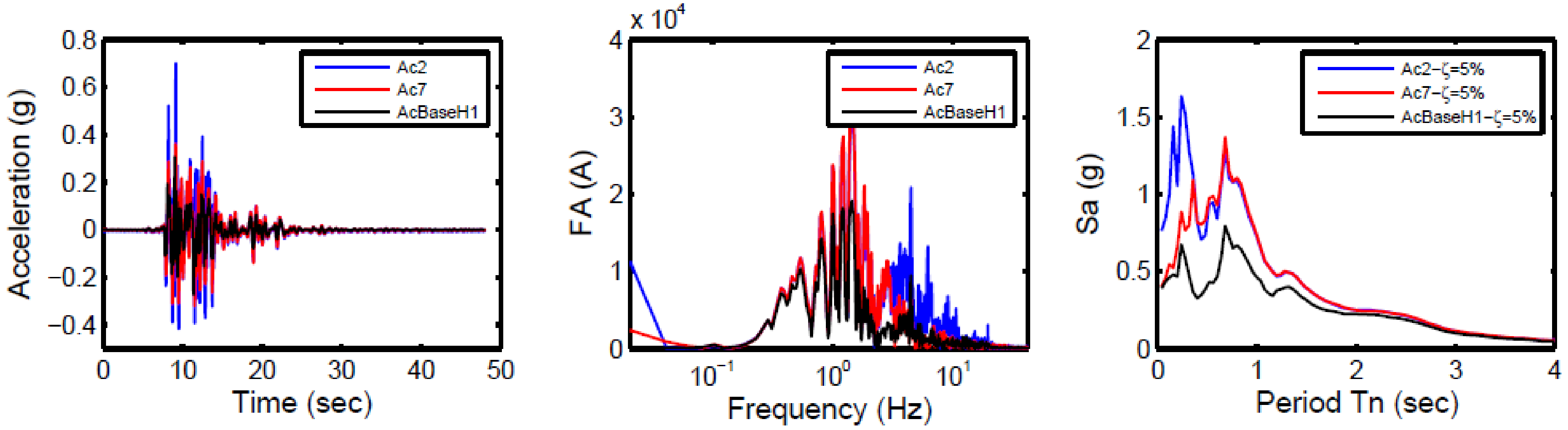


One – Dimensional Shaker

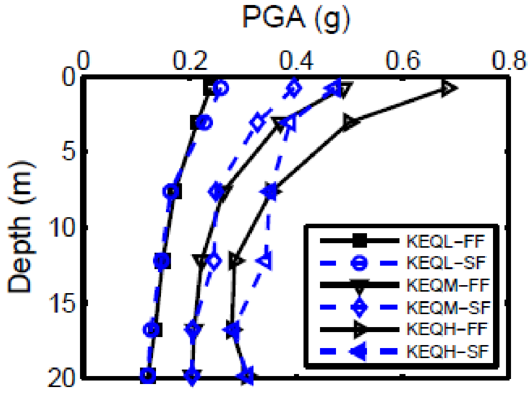


Centrifuge Results

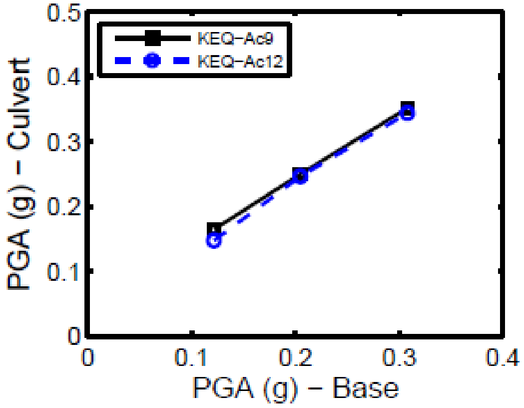
FF vs SF: Acceleration time history, Frequency and Response Spectrum



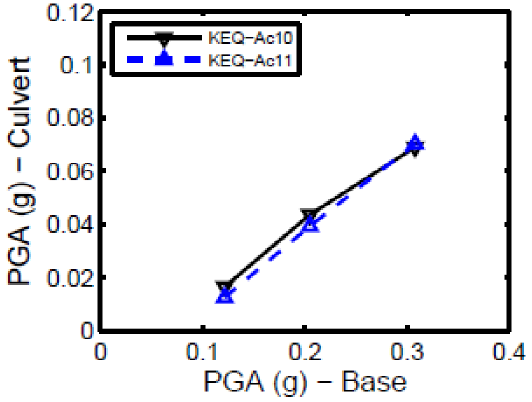
FF vs SF: Amplification



PGA (H) Culvert



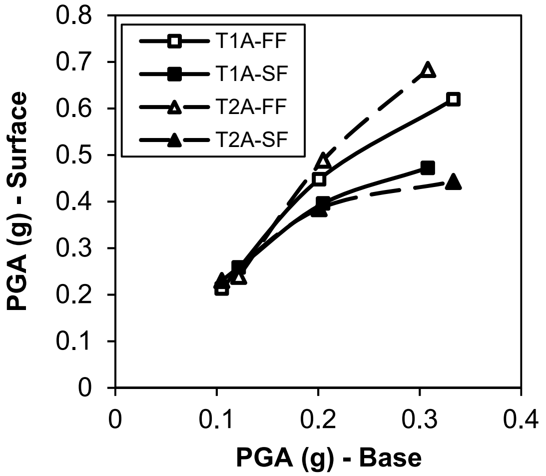
PGA (V) Culvert

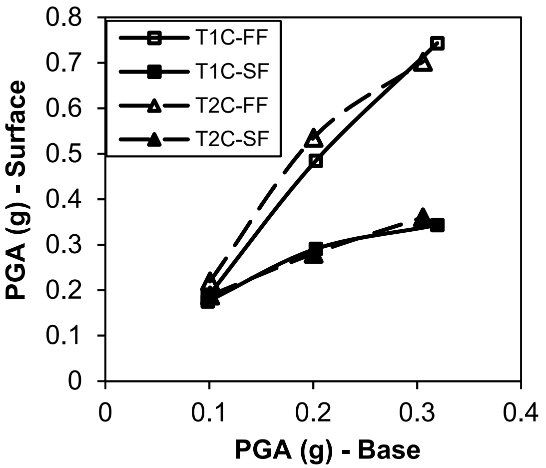


Centrifuge Results

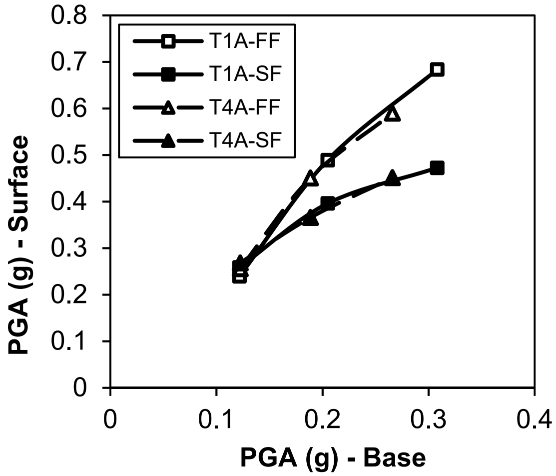
Kinematic Soil Culvert Interaction

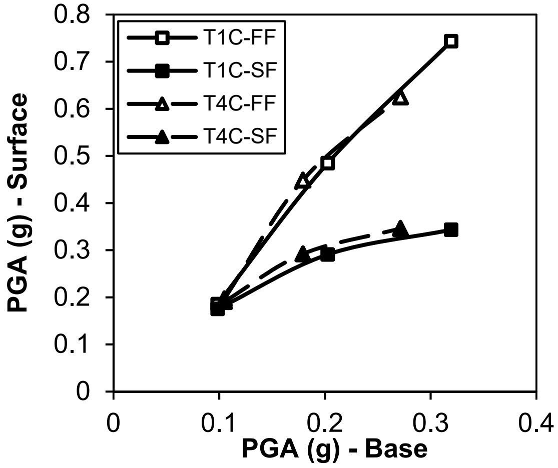
Effect of Soil Density





Effect of Culvert Thickness





Centrifuge Results

Soil Culvert Interaction Parameters: Soil Culvert Interaction Factors (F_e)

Test	Top Slab (F_e)	Side Wall (F_e) at rest
T1A	Edge 1.21	Top 1.11
	Center 1.04	Bottom 1.12
T2A	Edge 1.15	Top 1.09
	Center 1.09	Bottom 1.10
T3A	Edge 1.29	Top 1.13
	Center 0.90	Bottom 1.18
T4A	Edge 1.53	Top 1.14
	Center 0.68	Bottom 1.22

Rocking of Structures

Rocking of Box Culvert

	PGA (g) Base	Rocking Angle
Shaking Type	Test 1 Case C	
KEQL	0.098	0.0088
KEQM	0.203	0.0088
KEQH	0.319	0.0120

Rocking of Foundation

	PGA (g) Base	Rocking Angle
Shaking Type	Test 1 Case C	
KEQL	0.098	0.0112
KEQM	0.203	0.0104
KEQH	0.319	0.0192

Centrifuge Results

Racking of Box Culverts

Test	Shaking Type	*PGA* (Base) (g)	$\frac{\Delta PGD_{SF}}{\Delta PGD_{FF}}$
T1A	KEQL	0.122	0.76
T1C	KEQL	0.098	0.78
T1A	KEQM	0.205	0.77
T1C	KEQM	0.203	0.76
T1A	KEQH	0.308	0.46
T1C	KEQH	0.319	0.31

Test	Shaking Type	*PGA* (Base) (g)	$\frac{\Delta PGD_{SF}}{\Delta PGD_{FF}}$
T2A	KEQL	0.105	1.45
T2C	KEQL	0.101	1.59
T2A	KEQM	0.201	1.73
T2C	KEQM	0.200	1.74
T2A	KEQH	0.333	1.69
T2C	KEQH	0.306	1.72

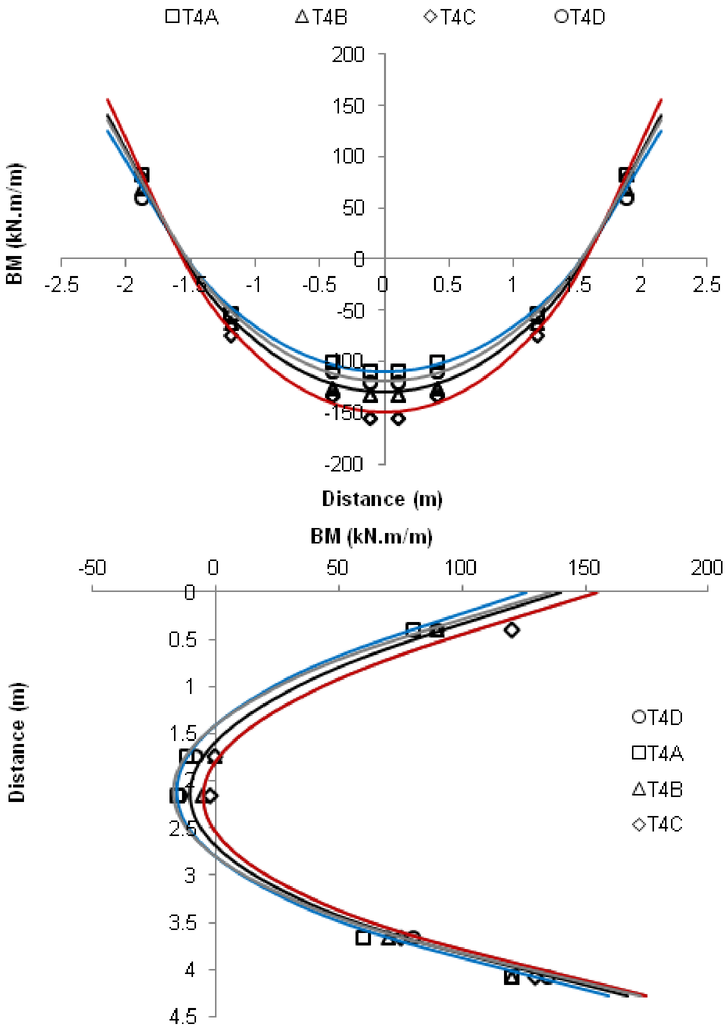
Test	Shaking Type	*PGA* (Base) (g)	$\frac{\Delta PGD_{SF}}{\Delta PGD_{FF}}$
T3A	KEQL	0.113	6.11
T3C	KEQL	0.098	14.52
T3D	KEQL	0.104	11.24
T3A	KEQM	0.214	15.92
T3C	KEQM	0.201	-45.60
T3D	KEQM	0.203	-158.50
T3A	KEQH	0.313	48.13
T3C	KEQH	0.298	-91.82
T3D	KEQH	0.301	156.03

Test	Shaking Type	*PGA* (Base) (g)	$\frac{\Delta PGD_{SF}}{\Delta PGD_{FF}}$
T4A	KEQL	0.122	4.77
T4C	KEQL	0.105	4.20
T4D	KEQL	0.104	4.86
T4A	KEQM	0.189	6.27
T4C	KEQM	0.179	7.23
T4D	KEQM	0.186	7.32
T4A	KEQH	0.266	7.07
T4C	KEQH	0.271	6.81
T4D	KEQH	0.270	6.34

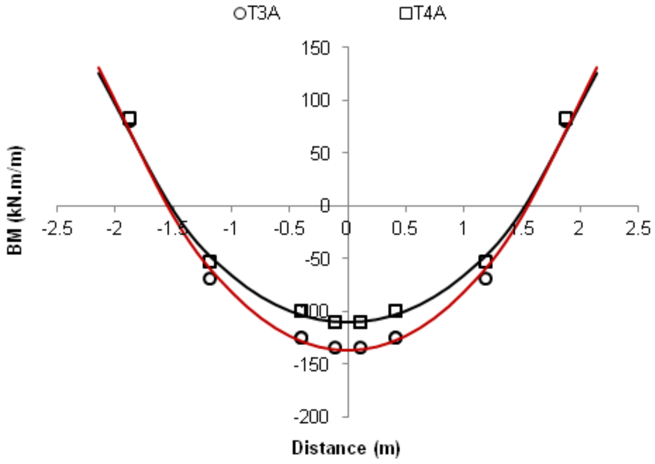
Centrifuge Results

Soil Culvert Interaction Parameters: Static Bending Moment

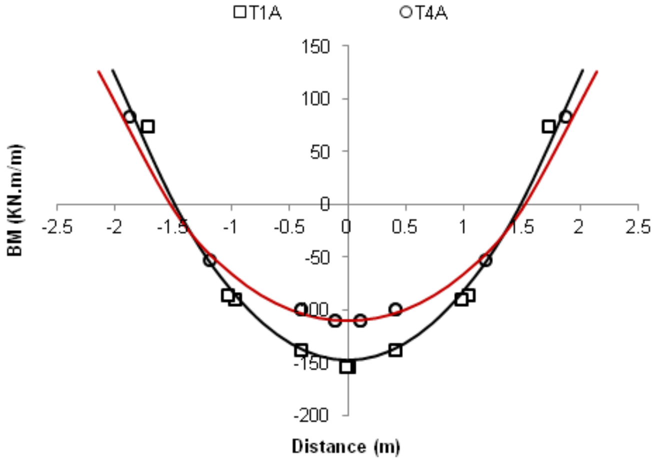
Effect of Surface Foundation



Effect of Soil Density

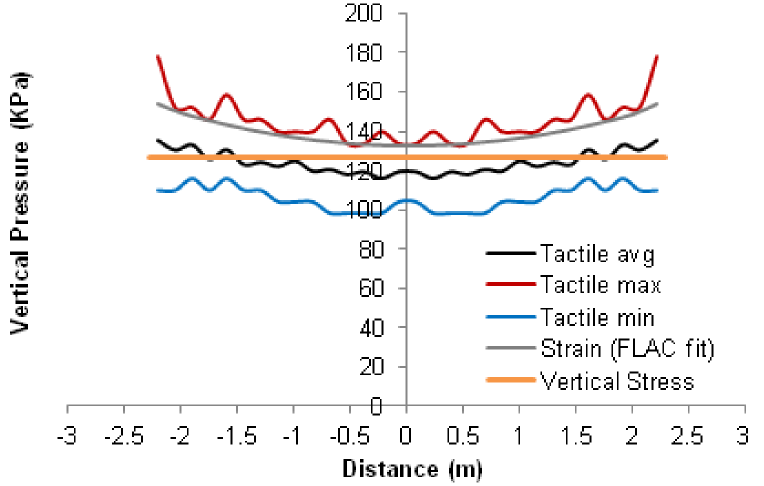


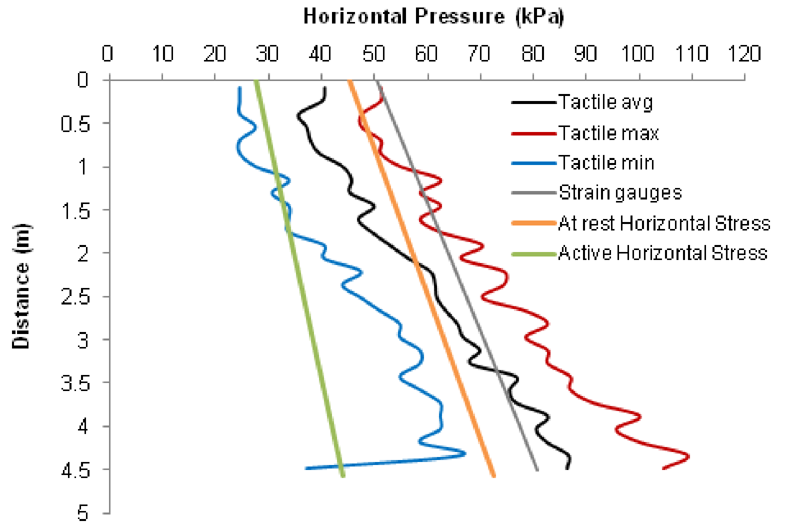
Effect of Culvert Thickness



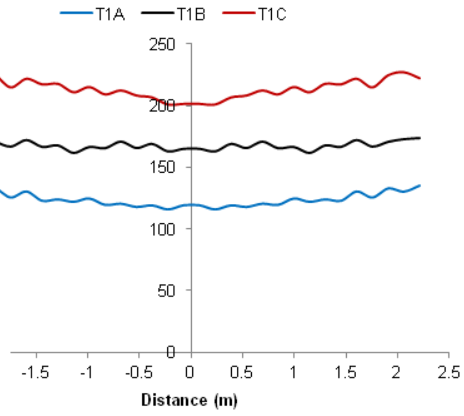
Centrifuge Results

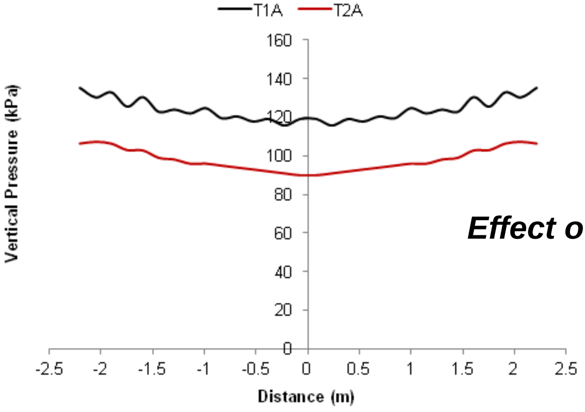
Soil Culvert Interaction Parameters: Static Soil Pressure



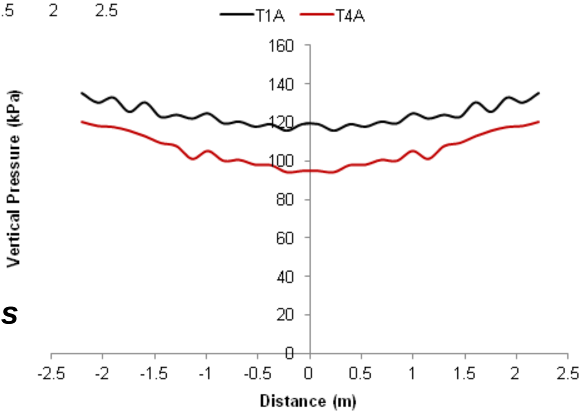


Effect of Surface Foundation





Effect of Soil Density



Effect of Culvert Thickness

Numerical Modeling

FLAC 2D

Structure – Linear Elastic Model
- *Box Culvert – Liner Element*
- *Foundation – Beam Element*

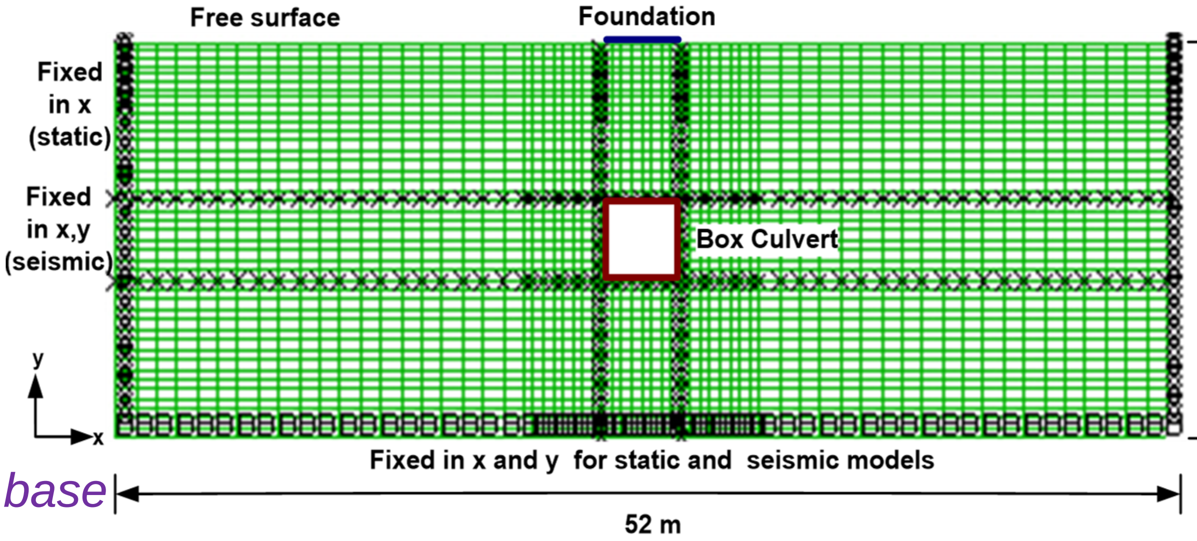
Sand – Elastic-Plastic Model
- *(Mohr Coulomb)*

Interface Elements – Glued interface (No slippage or gap opening)

In Dynamic analysis:

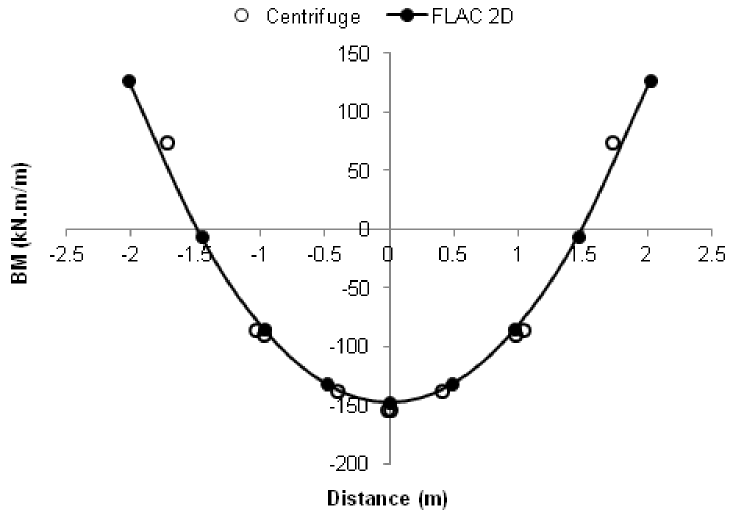
Vs and G obtained from centrifuge results used.
Hysteretic damping
Acceleration applied at the base

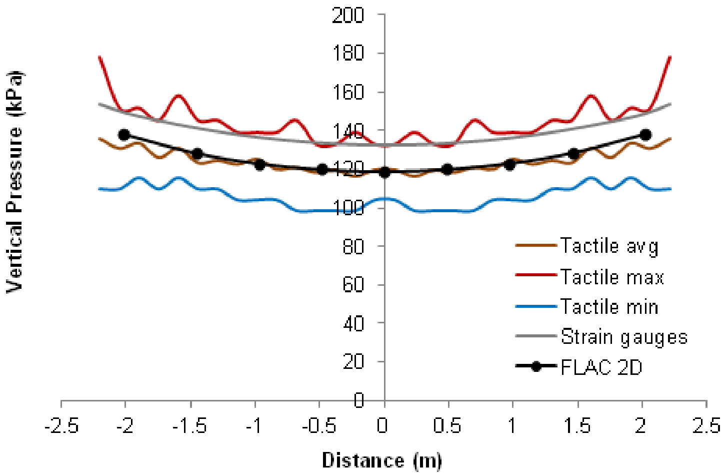
Model parameters	Medium dense	Dense
Relative density *Dr* (%)	50	90
Mass density ρ (kg/m^3)	1605.7	1687.7
Elastic modulus *Es* (MPa)	10	30
Shear modulus *G* (MPa)	3.91	11.7
Bulk modulus *K* (MPa)	7.58	22.7
Poisson's ratio ν	0.28	0.28
Friction angle ϕ (°)	40	40
Dilation angle ψ (°)	5	5
Cohesion c (kPa)	1	1



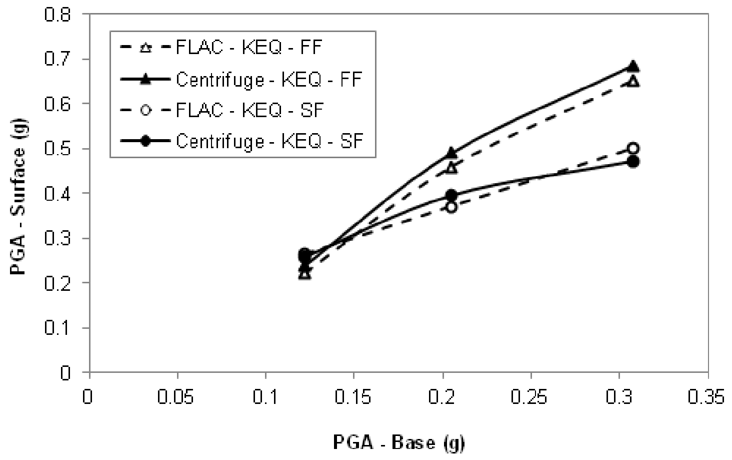
Numerical Modeling

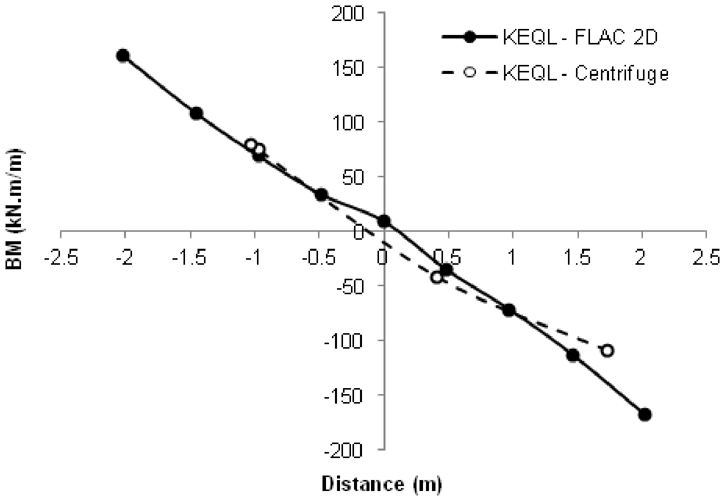
Verification of Static results





Verification of Seismic results





Static and Seismic Design Guidelines

1. Soil Pressure Distribution:
    a. Parabolic shape on the top slab
    b. Increase linearly on the side wall

2. Soil Culvert Interaction Factors:
    a. Obtain the F_e values using the *H/Bc - t/Bc* relations
    b. Consider the effect of soil *Es* and ν

3. Soil Pressure Values:
    a. Vertical Soil Pressure on top slabs

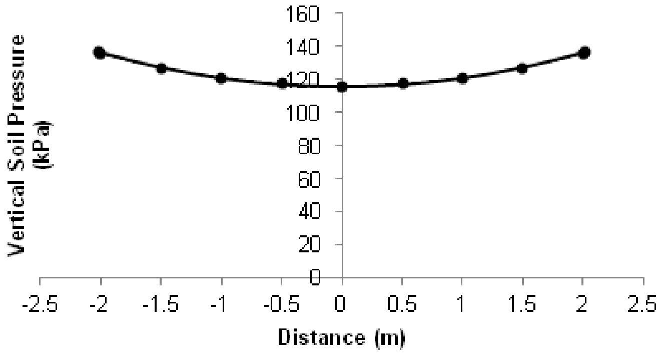
$$(\sigma_v)_{Actual} = F_e \,.\, (\sigma_v)_{Theoretical} = F_e \,.\, \gamma_s \,.\, H$$

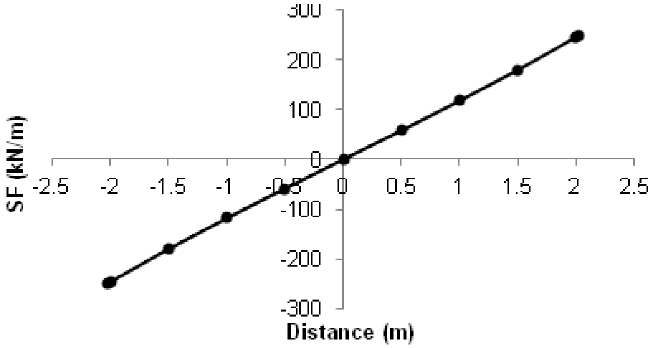
    b. Horizontal Soil Pressure on side wall

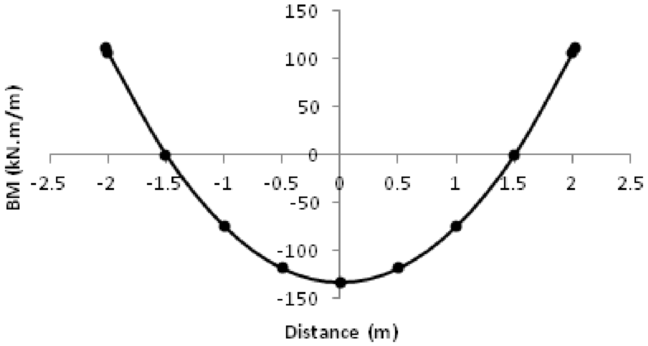
$$(\sigma_h)_{Actual} = F_e \,.\, (\sigma_h)_{Theoretical} = F_e \,.K.\, \gamma_s \,.\, H$$

4. Fit the top slab pressure with 2nd order polynomial and the side wall pressure with linear function, then double integrate them to get the SF and BM diagrams.

5. Obtain the ratio BM_{dy}/BM_{st} *using H/Bc - t/Bc* relations, the seismic bending moment can be obtained.







Seismic Design Guidelines

1. *Shape of Seismic Bending Moment:*
   1. No single shape for the seismic bending moment
   2. Recommended to run nonlinear dynamic numerical analysis
   3. Several factors affecting such as PGA and Frequency of EQ

2. *The BM_{dy}/BM_{st} ratio:*
   1. Useful in defining the seismic bending moment
   2. Can be used for specific *H/Bc* and *t/Bc*
   3. Can help in define the right value of *H of soil fill* and *t of the culvert*

3. *Total Bending Moment:*
   1. Not recommended to use
   2. Design separately for static BM and seismic BM and then combine.

4. *Kinematic Soil Culvert Interaction:*
   1. Should be considered for buried structures
   2. Leads to large reduction in PGA values at SF vs FF

5. *Racking of Box Culvert method:*
   1. Combined effect for the soil density and culvert thickness
   2. Should be used with caution

Interesting Findings

1. Kinematic soil culvert interaction can reduce the PGA (SF) at the surface by considerable amount comparing to the PGA (FF). The results show about a 50% reduction at 0.3g KEQ and a minimum effect below 0.1g. Soil density effect appears more for the (FF) condition.

2. The rocking angles of structures are very small, the values obtained for the box culvert is less than the surface foundation because of the soil confinement.

3. The PGD and PGA at the top of the foundation increase as the PGA at the model base increase.

4. The racking ratio of box culvert is soil density and culvert thickness dependent. For thick culvert and dense sand, the ratio is < 1.0, while for all the other cases > 1.0 and even higher with some negative values. Therefore this method should be used with caution.

References

1. Abuhajar, O., El Naggar, M.H. and Newson, T. 2016. Numerical modeling of soil and surface foundation pressure effects on buried box culvert behaviour. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 142 (12).
2. Abuhajar, O., El Naggar, M.H. and Newson, T. 2015. Static soil culvert interaction the effect of box culvert geometric configurations and soil properties. Computers and Geotechnics. Vol., 69, pp. 219-235.
3. Abuhajar, O., El Naggar, M.H. and Newson, T. 2015. Experimental and numerical investigations of the effect of buried box culverts on earthquake excitation. Soil Dynamics and Earthquake Engineering, Vol. 79, pp. 130-148.
4. Abuhajar, O., El Naggar, M.H. and Newson, T. 2015. Seismic soil culvert interaction. Canadian Geotechnical Journal, 52(11): 1649–1667.
5. Abuhajar, O., Newson, T and El Naggar, M.H. 2015. Scaled physical and numerical modelling of static soil pressures on box culverts. Canadian Geotechnical Journal, 52(11): 1637:1648.

Performance Assessment and Design Guidelines for Three-Sided Precast Concrete Culverts

(Funding by MTO and Canadian Concrete and Precast Pipe Association)

- Three-sided Culvert Shapes

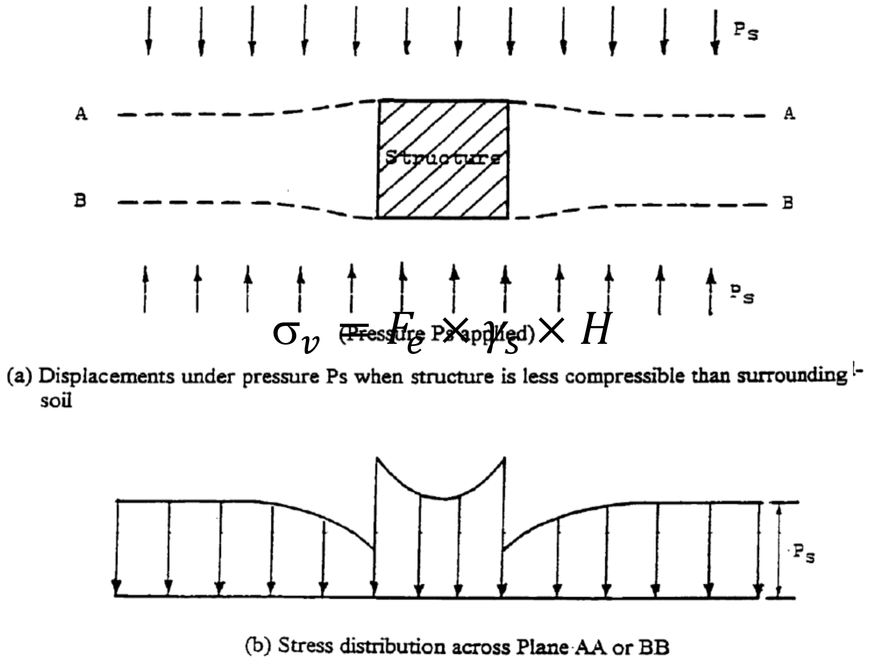
https://www.westsidepioneer.com/Articles/082217/Chestnut.html

http://www.bargerandsons.com/precast/box-culvert/box-culvert/three-sided-box-culvert.html

https://www.fortmiller.com/wp-content/gallery/box-culverts/boxculverts_2.jpg

Literature Review

- Soil arching
- Active vs Passive arching



$\sigma_v = F_e \times \gamma_s \times H$

(Pressure Ps applied)

(a) Displacements under pressure Ps when structure is less compressible than surrounding soil

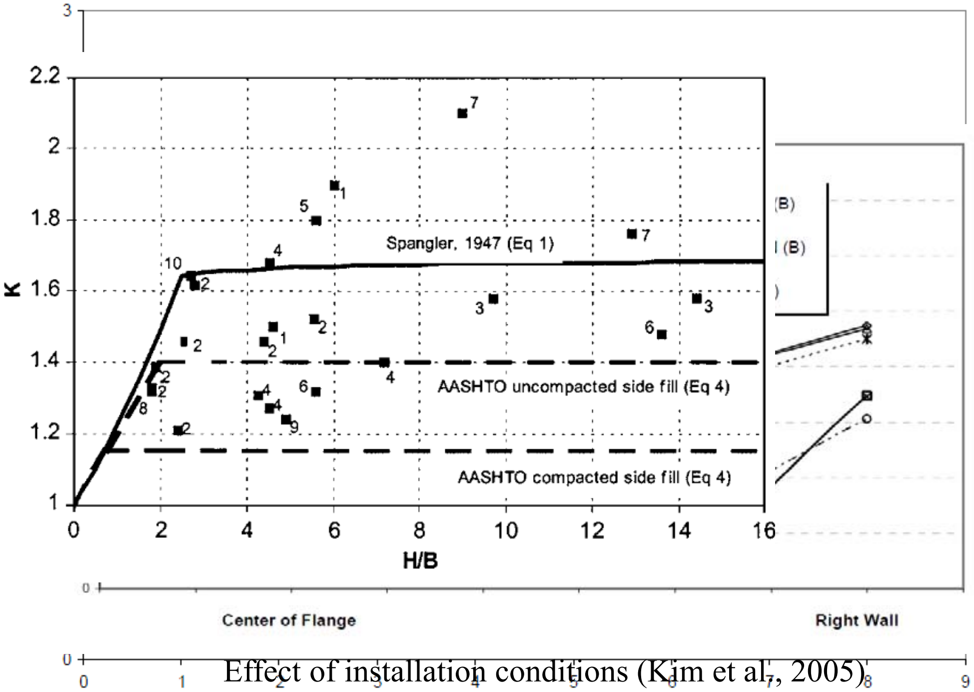
(b) Stress distribution across Plane AA or BB

Passive arching (Evans, 1984)

Active arching (Evans, 1984)

Literature Review

- Factors affecting the applied stresses on culverts
  - Geometry and stiffness of the structure
  - Soil properties
  - Backfill height
  - Installation conditio



Effect of installation conditions (Kim et al., 2005)

Effect of soil conditions (Kim et al., 2005)

Literature Review

- Current design of practice (CHBDC S6-14)

Arching factors for box sections in standard installations. (CHBDC, 2014)

Installation Type	Vertical Arching Factor, λ_v	Horizontal Arching Factor, λ_h	
		Minimum	Maximum
B1	1.20	0.30	0.50
B2	1.35	0.25	0.50

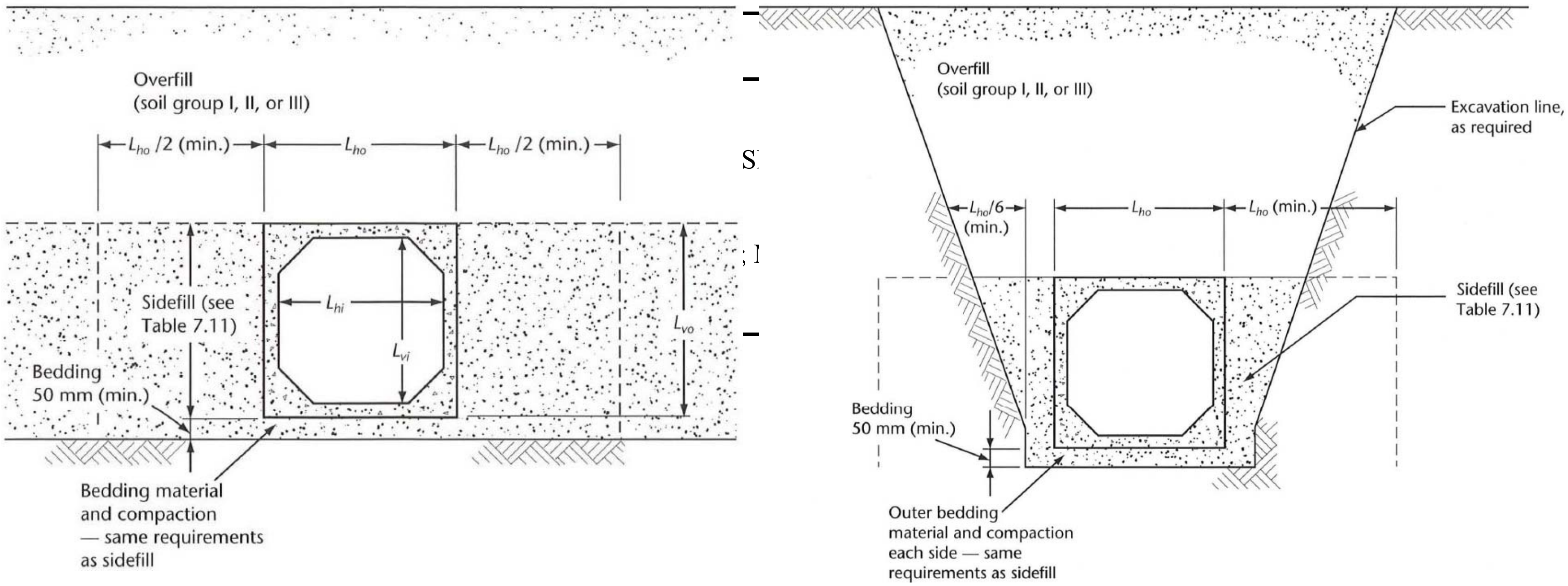
Soils and compaction requirements for standard installations. (CHBDC, 2014)

Installation type	Soil group	Equivalent minimum Standard Proctor compaction in sidefill and outer bedding zones
B1	I	90%
	II	95%
	III	Not permitted
B2	I	80%
	II	85%
	III	95%

Literature Review

- Current design of practice (CHBDC S6-14)

Classification of placed soils. (CHBDC, 2014)



CHBDC, 2014

Literature Review

- Current design of practice (AASHTO LRFD 2014)

$$W_E = F_{e\ or\ t} \times \gamma_s \times B_c \times H$$

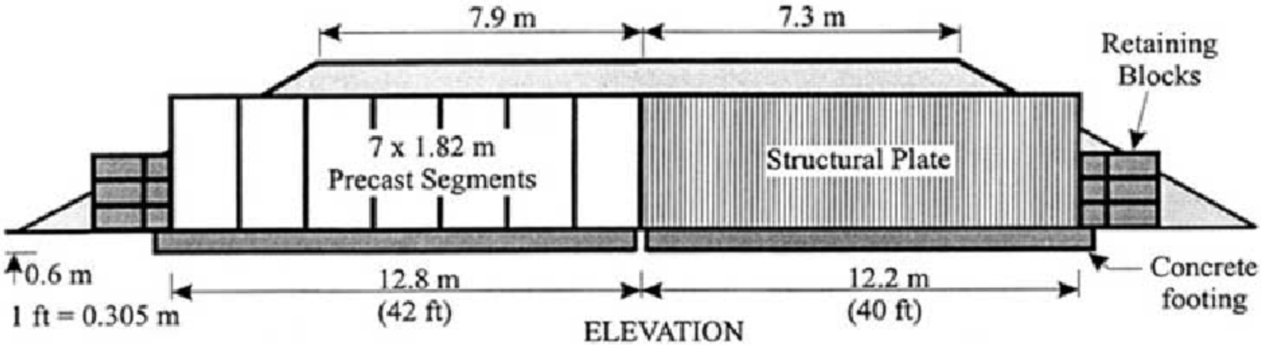
$$F_e = 1 + 0.20\frac{H}{B_c}$$

For installations with compacted and un-compacted fill along the sides of box culverts, F_e shall not exceed 1.15 and 1.40, respectively.

$$F_t = \frac{C_d\ {B_d}^2}{H\ B_c} \leq F_e$$

Literature Review

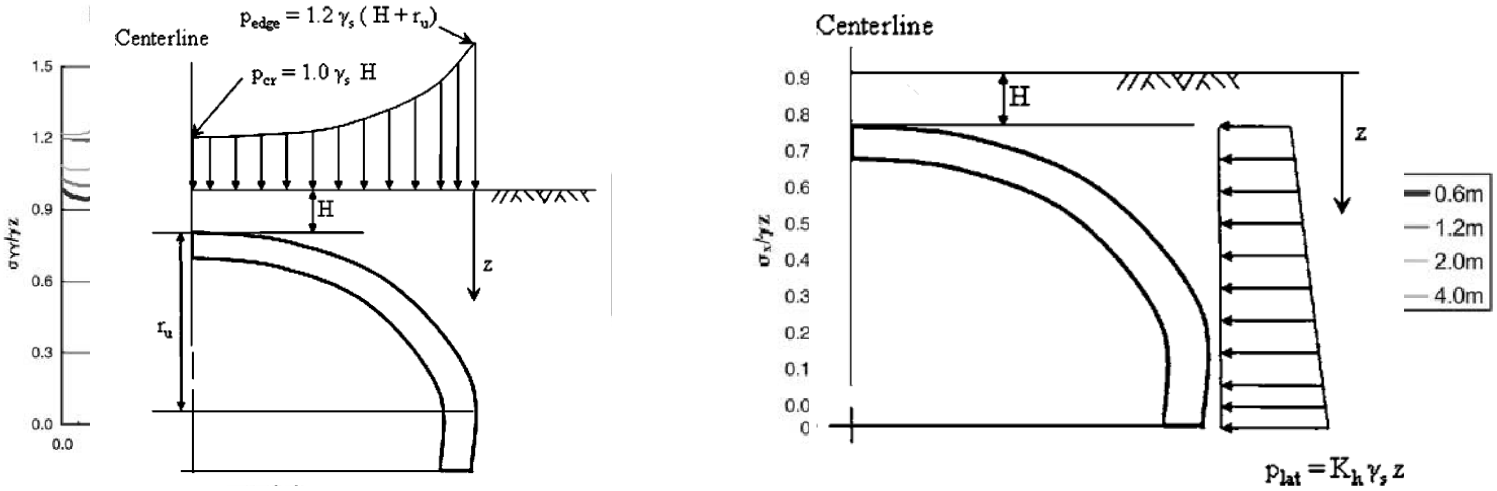
- Arch culverts (McGrath et al., 2002)
  - Two full-scale field tests on a 9.1 m span arch culvert
  - Different backfill depths
  - Different compaction (92% and 85% relative compaction)



Arrangement of tested culverts (McGrath et al., 2002)

Literature Review

- Arch culverts (McGrath et al., 2002)



(McGrath et al., 2002)

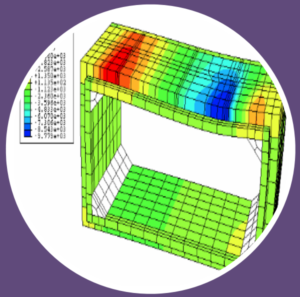
Research Motivation

- Large span three-sided concrete culverts are gaining popularity and are currently used to provide an economical alternative for short span bridges replacement.
- Design practice, as currently specified in Codes and Standards, does not distinguish between box and three-sided culverts.
- Stress distribution on large span arch culverts is different than that used for box culverts.
- No research on the structural performance and soil-structure interaction of three-sided culverts.
- Results of our preliminary numerical analyses.

Objectives

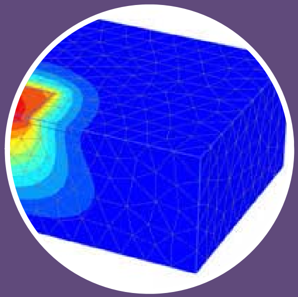
- Develop design methodologies, specific to precast concrete three-sided structures, for Section 7, Buried Structures for CSA S6-CHBDC.
- Enhance knowledge on behaviour of three-sided culverts and the influencing parameters.
- Investigate actual loading conditions due to soil pressures and vehicle live loads.
- Investigate long term performance of three sided culverts.
- Assess the applicability and limitations of the current state of practice design methods for three-sided culverts.
- Provide recommendations for representative numerical modelling of three-sided culvert-soil systems.

Methodology



Preliminary Analysis

Full-Scale Field Tests

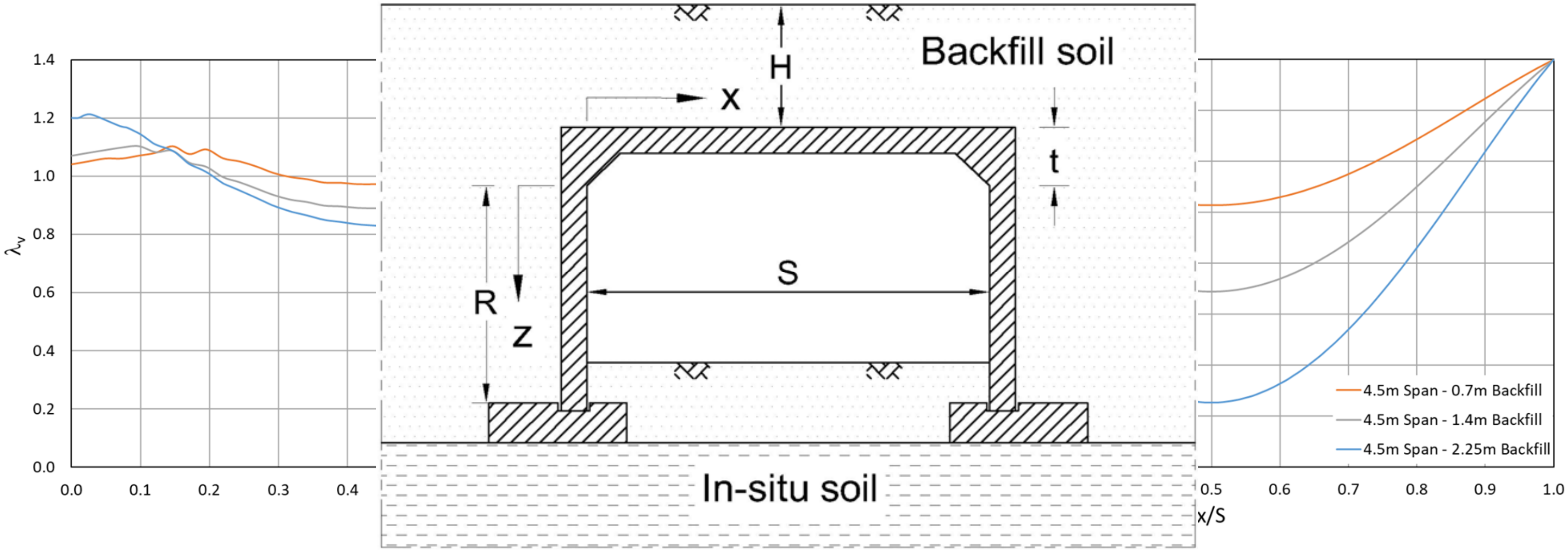


Numerical Modelling

Centrifuge Modelling

Preliminary Analysis

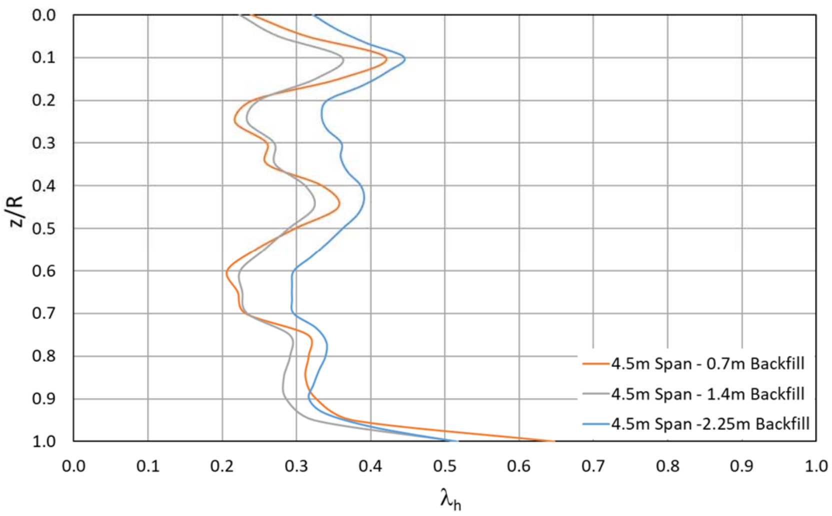
- Preliminary Analysis (results for 4.5 m span culvert)

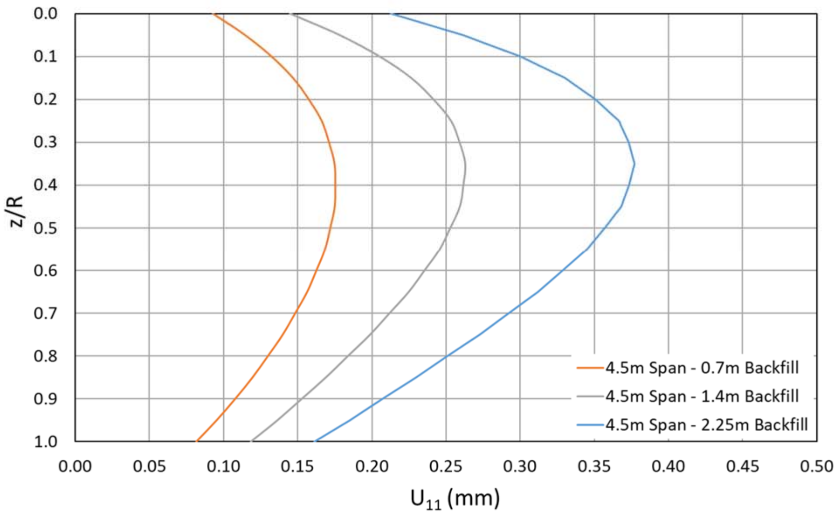


$$\lambda_v = \frac{\sigma_{33}}{\gamma H}$$

Preliminary Analysis

- Preliminary Analysis (results for 4.5 m span culvert)

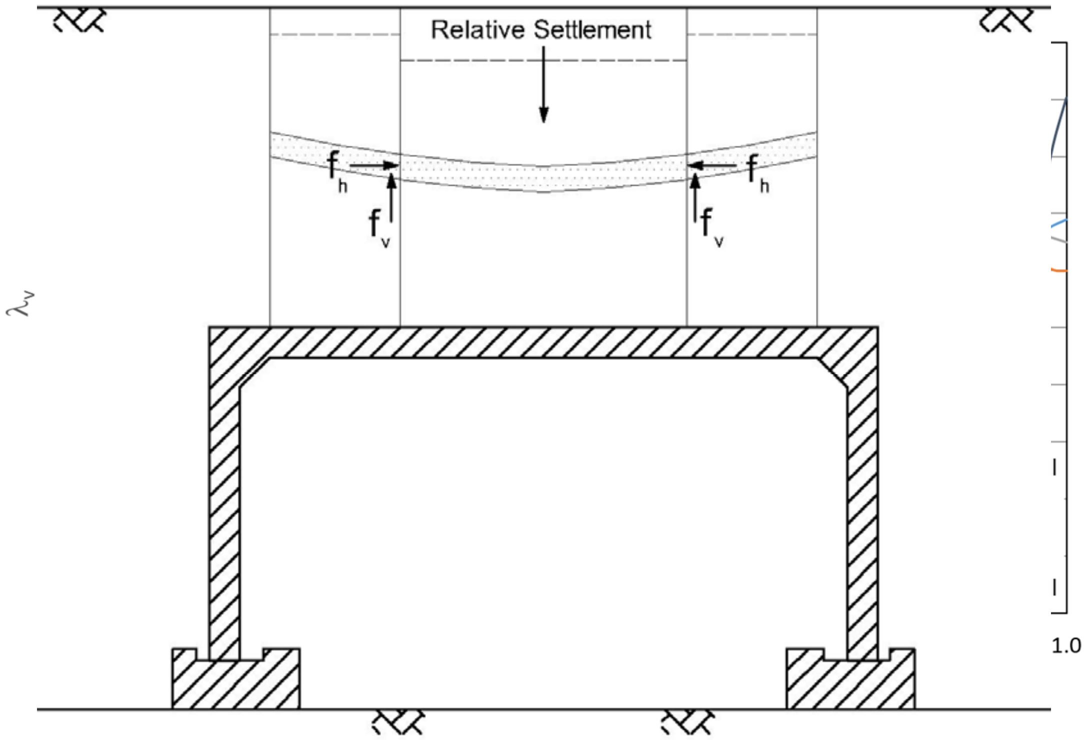




$$\lambda_h = \frac{\sigma_{11}}{\gamma(H+t+z)}$$

Preliminary Analysis

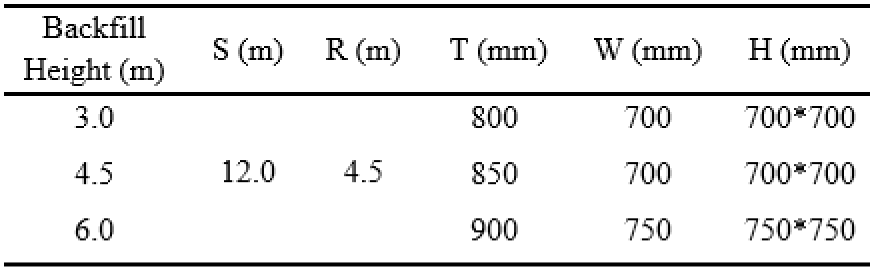
- Preliminary Analysis (results for different spans)



Preliminary Analysis

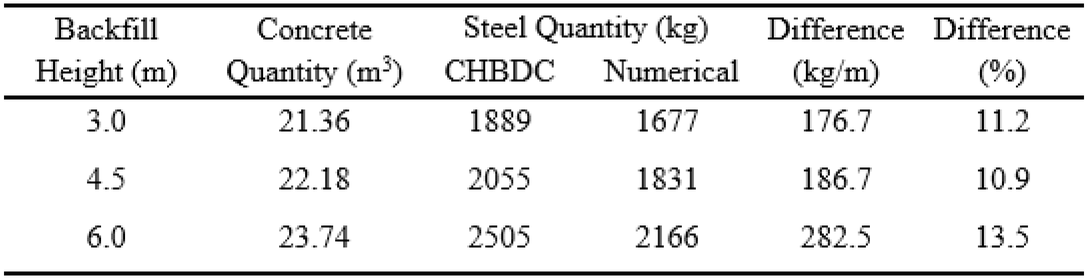
- Preliminary Analysis (Reinforcement)

12.0m span culvert dimensions



Backfill Height (m)	S (m)	R (m)	T (mm)	W (mm)	H (mm)
3.0	12.0	4.5	800	700	700*700
4.5			850	700	700*700
6.0			900	750	750*750

12.0m span culvert quantities



Backfill Height (m)	Concrete Quantity (m^3)	Steel Quantity (kg)		Difference (kg/m)	Difference (%)
		CHBDC	Numerical		
3.0	21.36	1889	1677	176.7	11.2
4.5	22.18	2055	1831	186.7	10.9
6.0	23.74	2505	2166	282.5	13.5

Full-Scale Field Tests

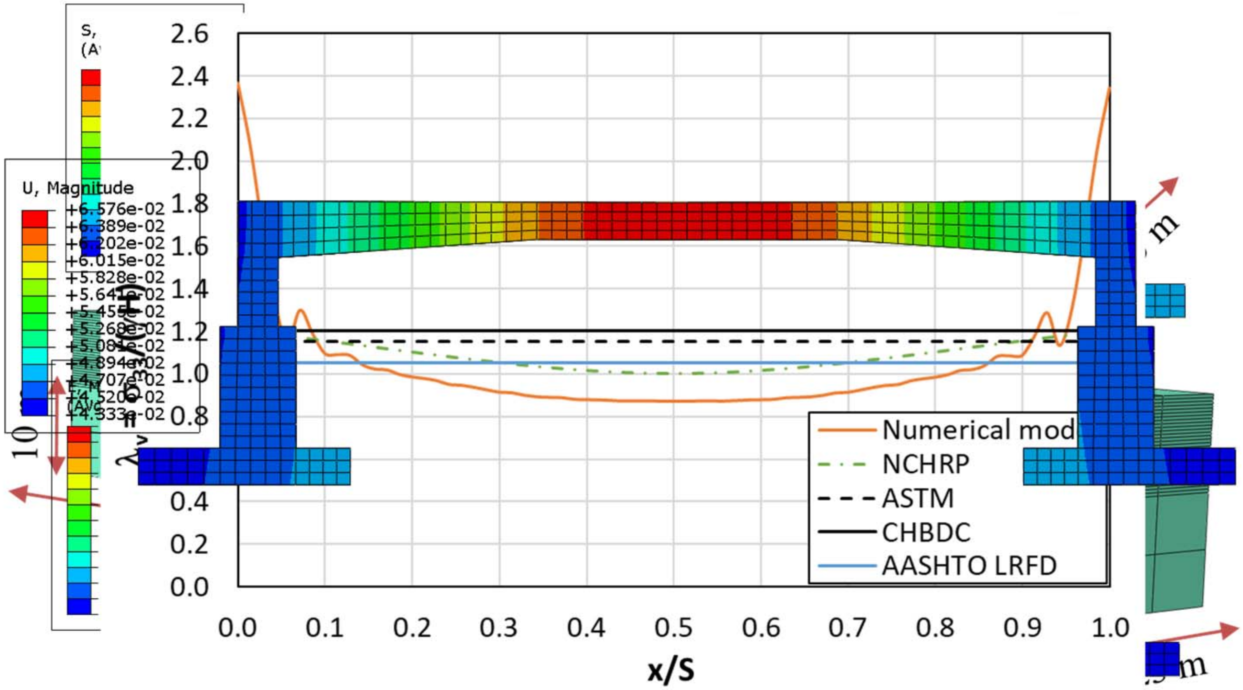
- Full-scale field tests are funded by MTO and CCPPA
- Geometry of the approved projects

Details of the approved field tests

Culvert ID	Culvert type	Span (m)	Backfill height (m)	Foundation type
Culvert 18	Three-sided	16.1	1.2	Shallow
Pickering Culvert	Three-sided	13.5	3.0	Shallow
Oshawa Culvert	Three-sided	10.3	3.8	Shallow
Barbut Creek	Three-sided	10.0	1.4	Shallow
Locha Creek	Three-sided	7.3	6.5	Deep
Muskrat Creek	Box or three-sided	4.0	0.7	To be determined
Meloche Creek	Box	3.6	5.0	Shallow

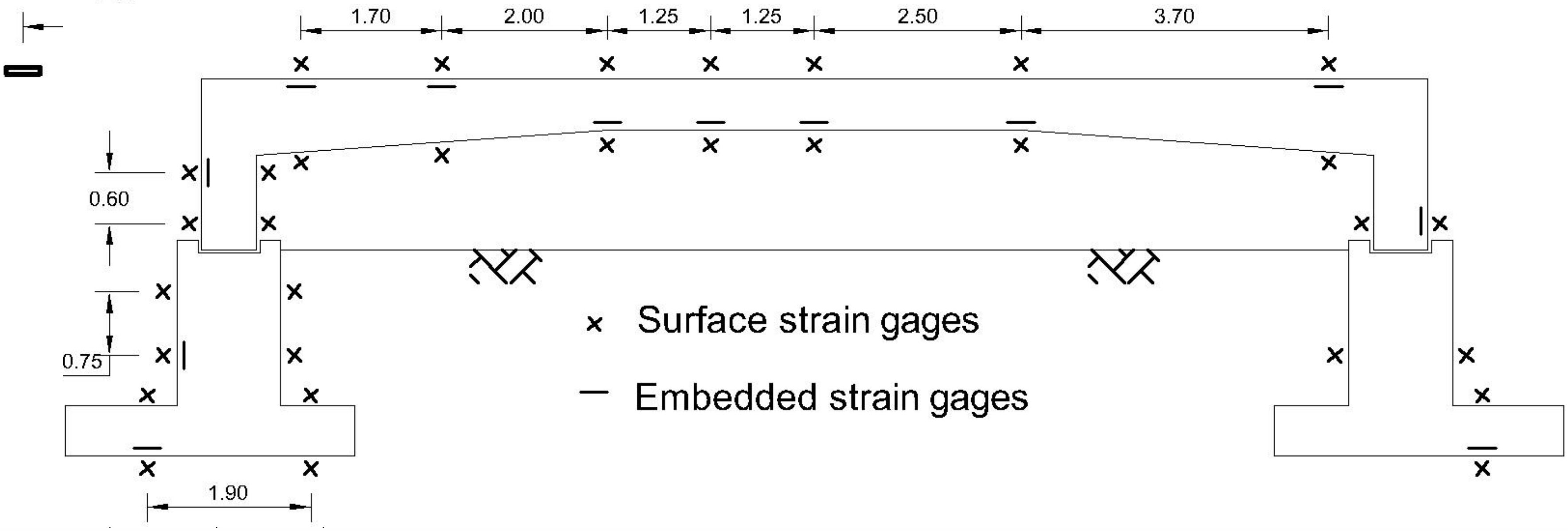
Full-Scale Field Tests

- Field tests design (Oshawa Culvert)
  - 3-D Preliminary numerical analyses have been conducted for the approved projects.



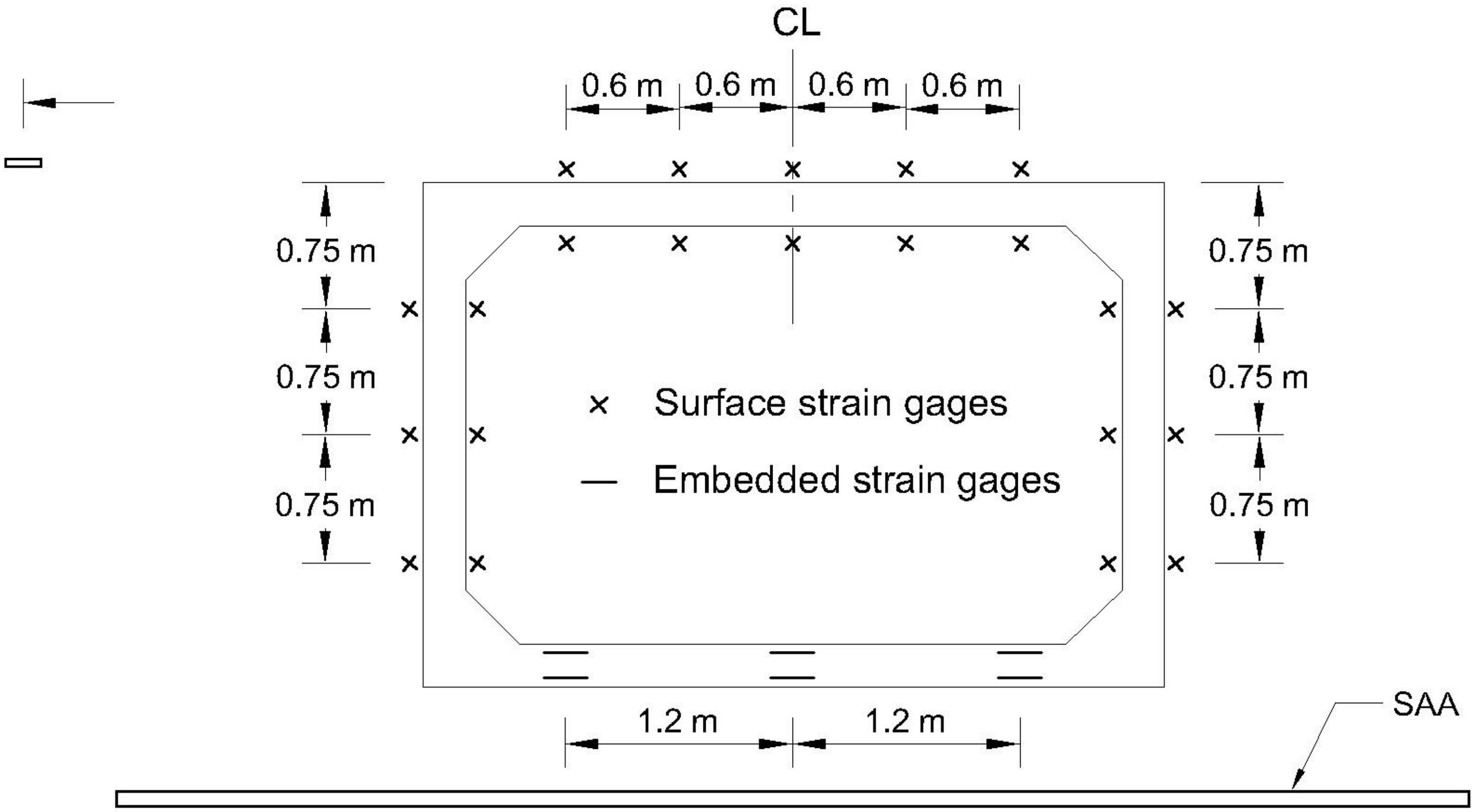
Full-Scale Field Tests

- Field tests design (Oshawa Culvert)
  - Measurements will include interface pressures, culvert internal strains, and soil settlement.
  - Instruments: pressure cells, strain gauges (surface and embedded), **SAA**, and multi-point borehole extensometers.



Full-Scale Field Tests

- Field tests design (Meloche Creek Culvert)



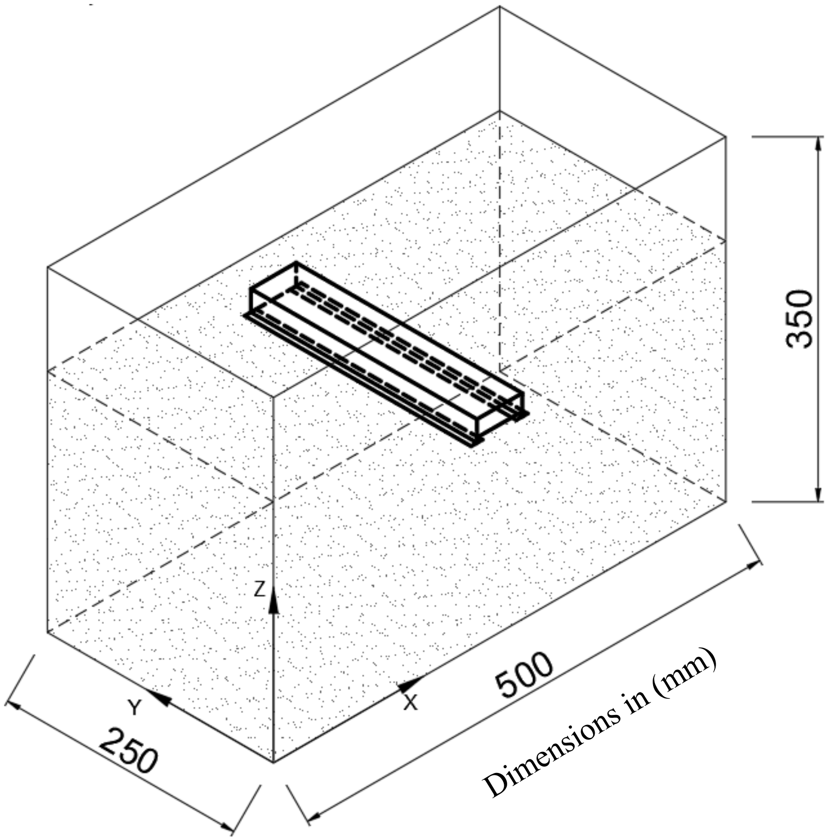
Centrifuge Modeling

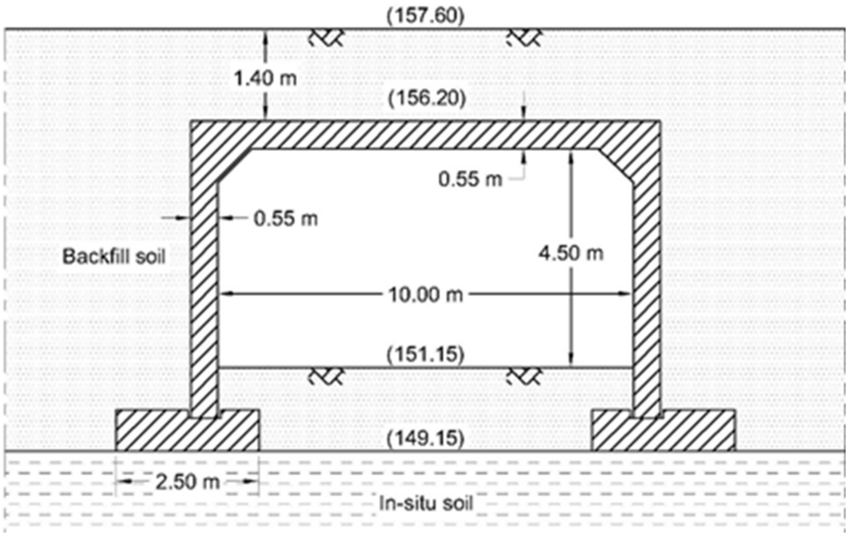
- Expanding the full-scale field testing to cover other important parameters, e.g. installation method and foundation soil conditions.
- Centrifuge testing on long span culverts with arched top slab.

Centrifuge facility at RPI, Troy, NY, USA

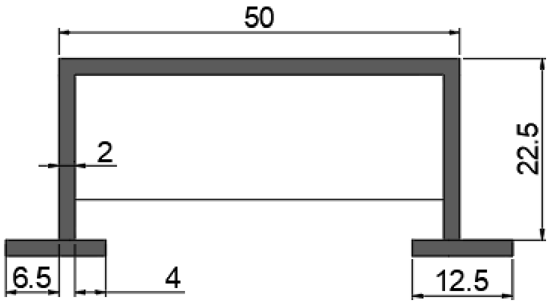
Centrifuge Modelling of Three Sided Culverts

- Model Scaling (N=200)





Dimensions in (m)



Dimensions in (mm)

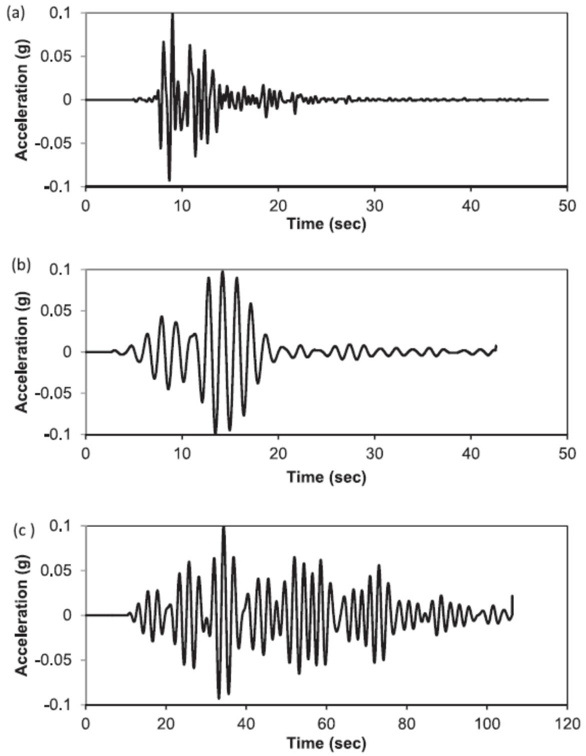
Centrifuge Modelling under Static and Seismic Loading

- Static Loading
- Seismic Loading

Summary of the proposed input motions

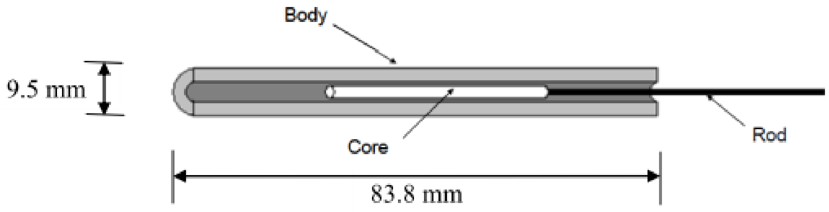
Input acceleration	Prototype PGA (g)	Prototype Predominant frequency (Hz)	Centrifuge test (scale 1:200) PGA (g)	Centrifuge test (scale 1:200) Predominant frequency (Hz)
VCL	0.10	0.464	20.0	92.8
VCM	0.16	0.464	32.0	92.8
WCL	0.09	0.647	18.0	129.4
WCM	0.24	0.647	48.0	129.4
KEQL	0.12	1.453	24.0	290.6
KEQM	0.20	1.453	40.0	290.6
KEQH	0.31	1.453	62.0	290.6

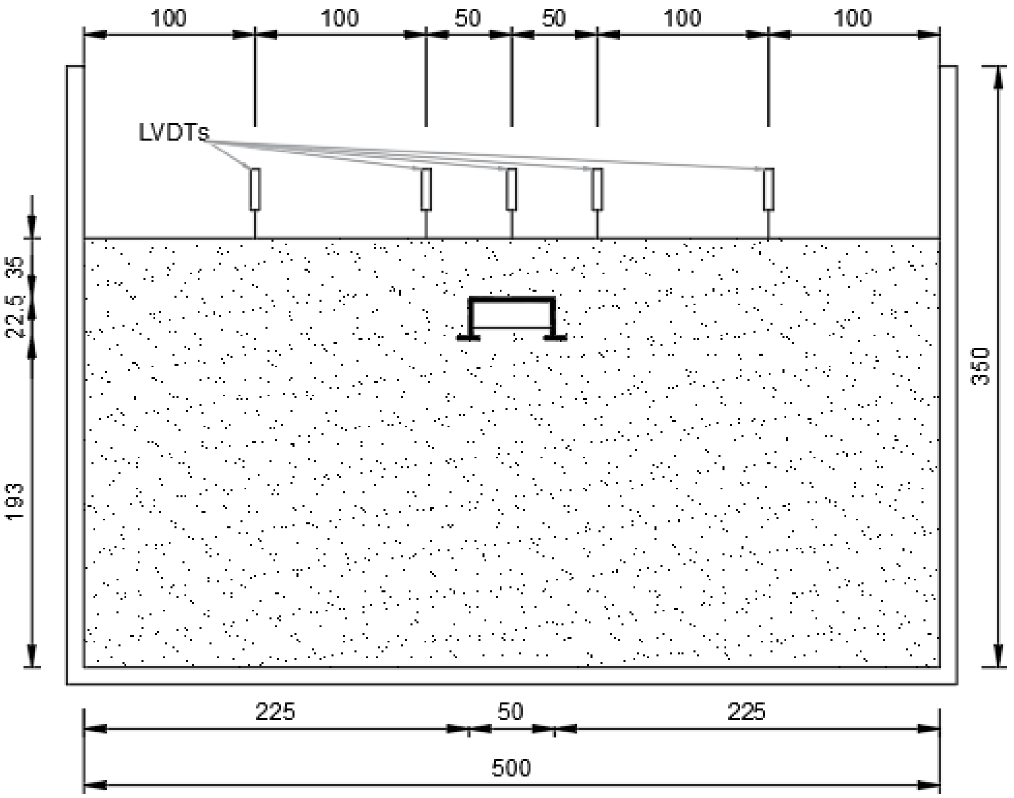
Note: VC, Vancouver Cascadia; WC, Western Canada; KEQ, Kobe Earthquake; L, low; M, medium; H, high



Test Instrumentation

1- Linear Variable Differential Transducers (LVDT)

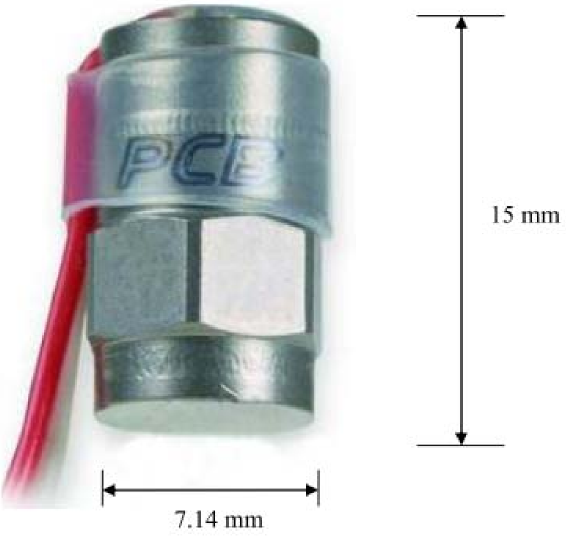


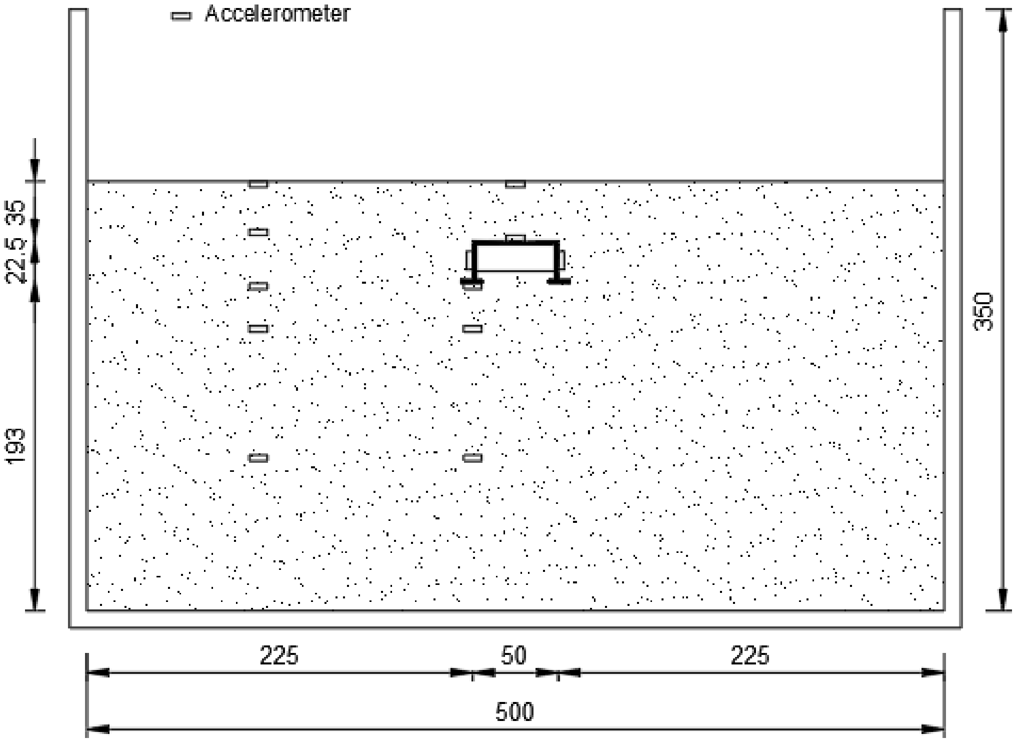


Dimensions in (mm)

Test Instrumentation

2- Accelerometers

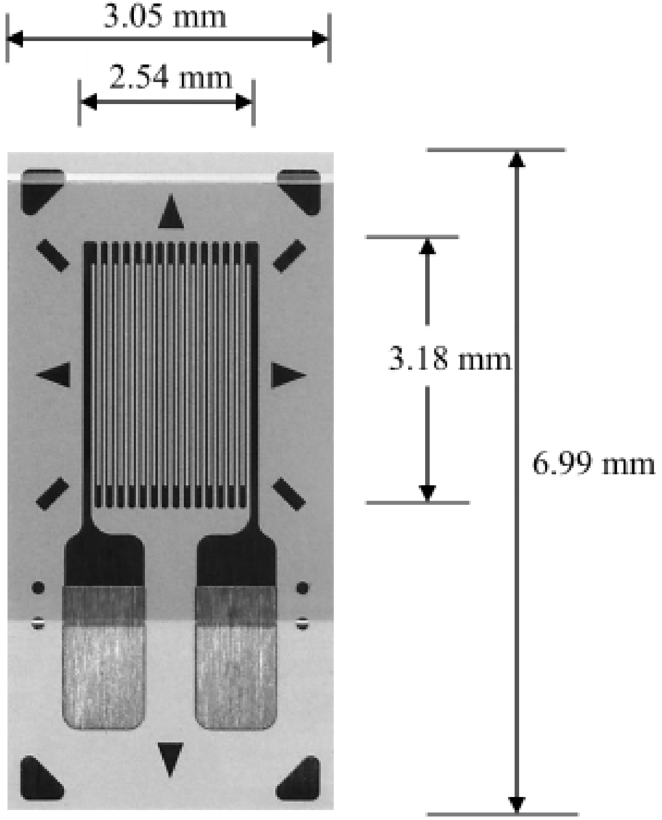




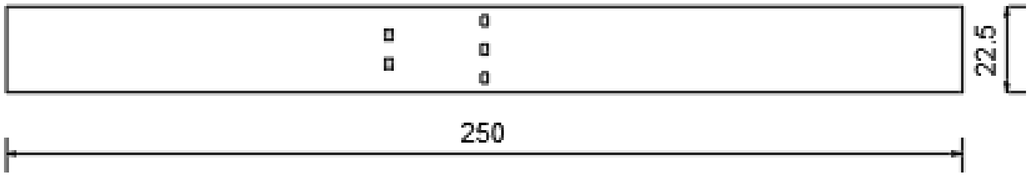
Dimensions in (mm)

Test Instrumentation

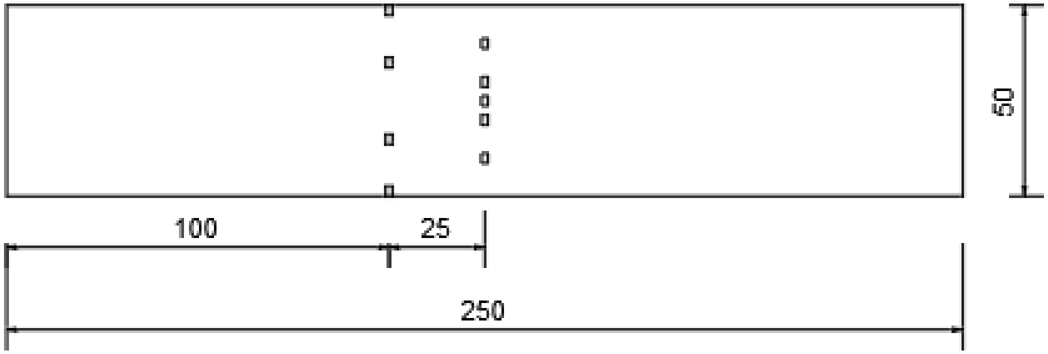
3- Strain Gauges



Strain Gauges



Side View of Culvert Wall



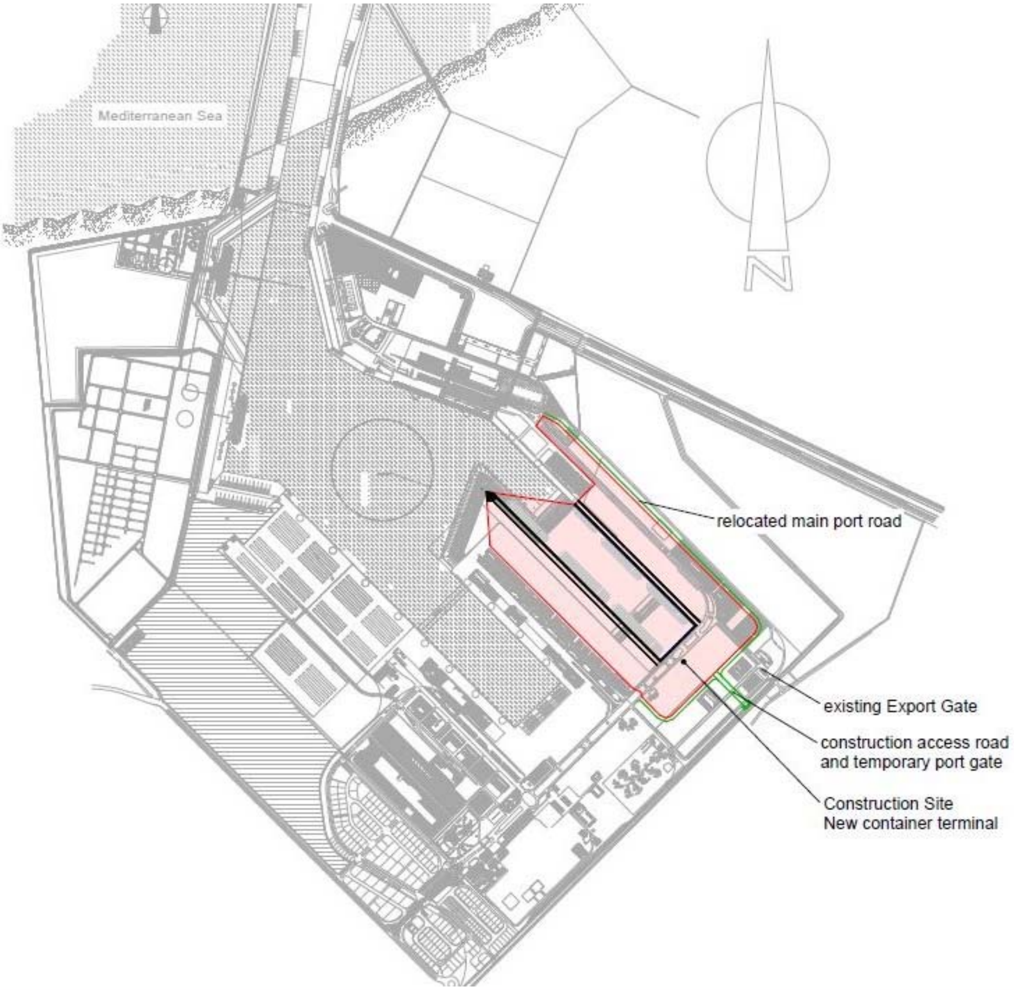
Plan of Culvert Slab

Dimensions in (mm)

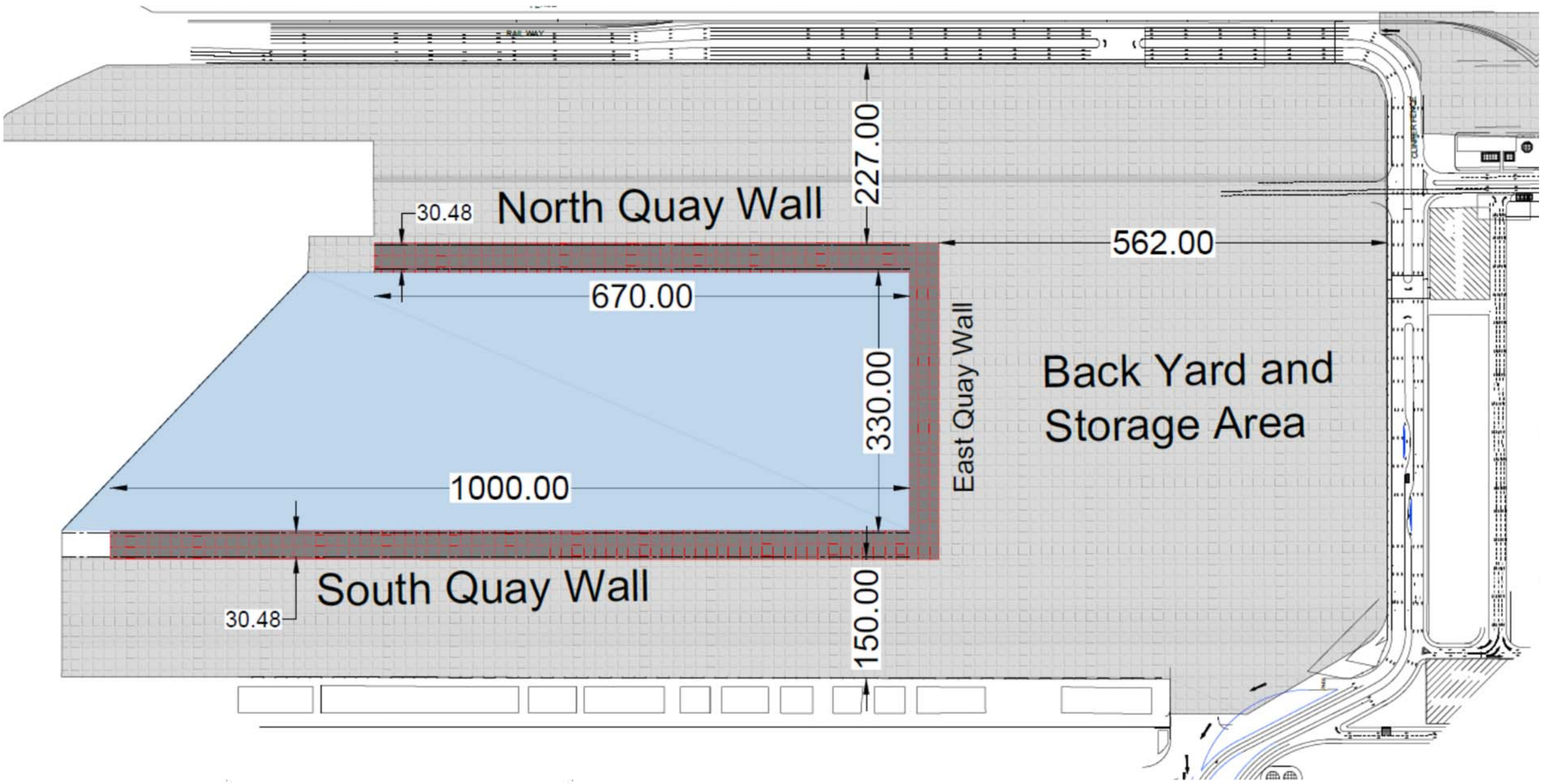
Design Alternatives for Damietta International Port - Second Container Terminal

Funded by Research Contract – Geotechnical Research Centre

- The por ptian
  Mediterra port
  is approxi west
  of Port Sa
- The total
- Damietta
  container
  Terminal).



Project Description

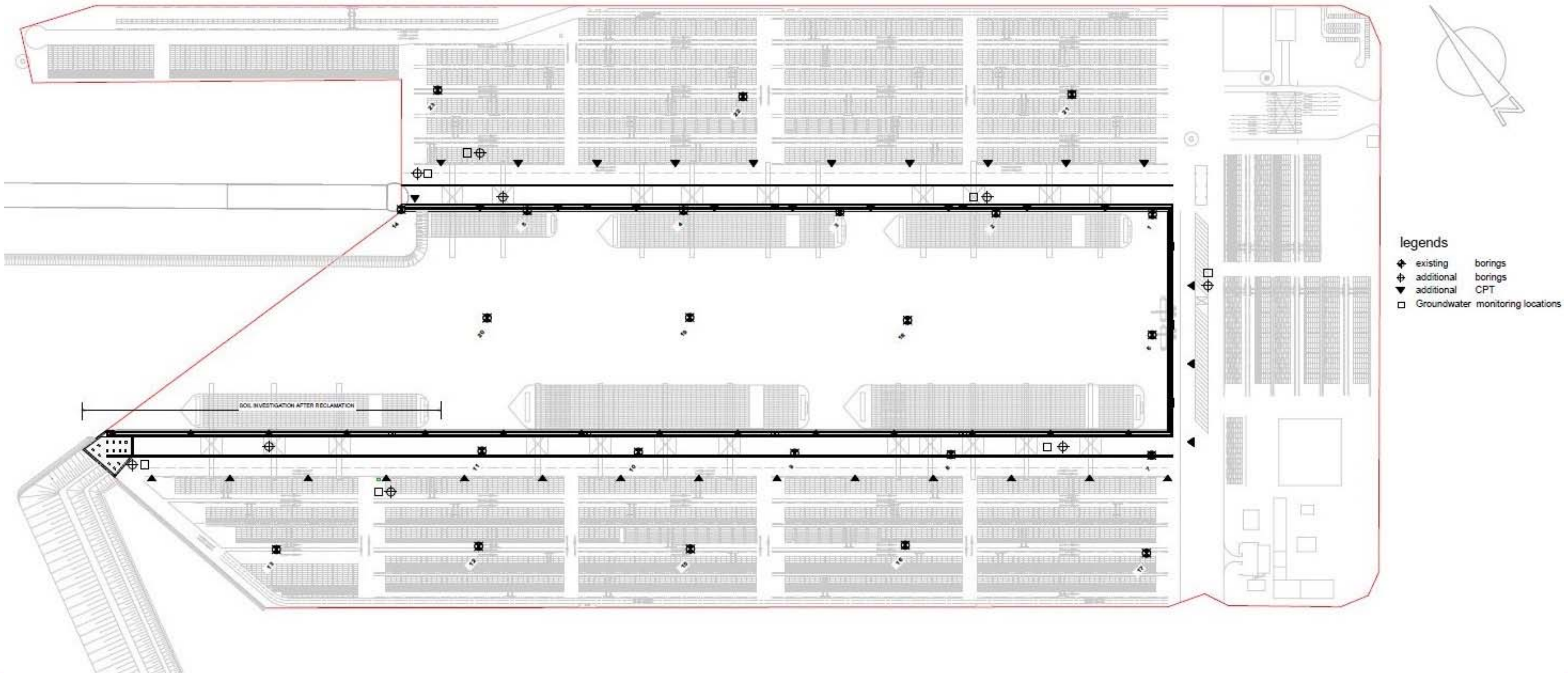


South Terminal Area : 226,400 M^2
Total Terminal Spaces : 768,232 M^2

South Terminal Area : 4,220
Total Terminal Spaces : 16,465

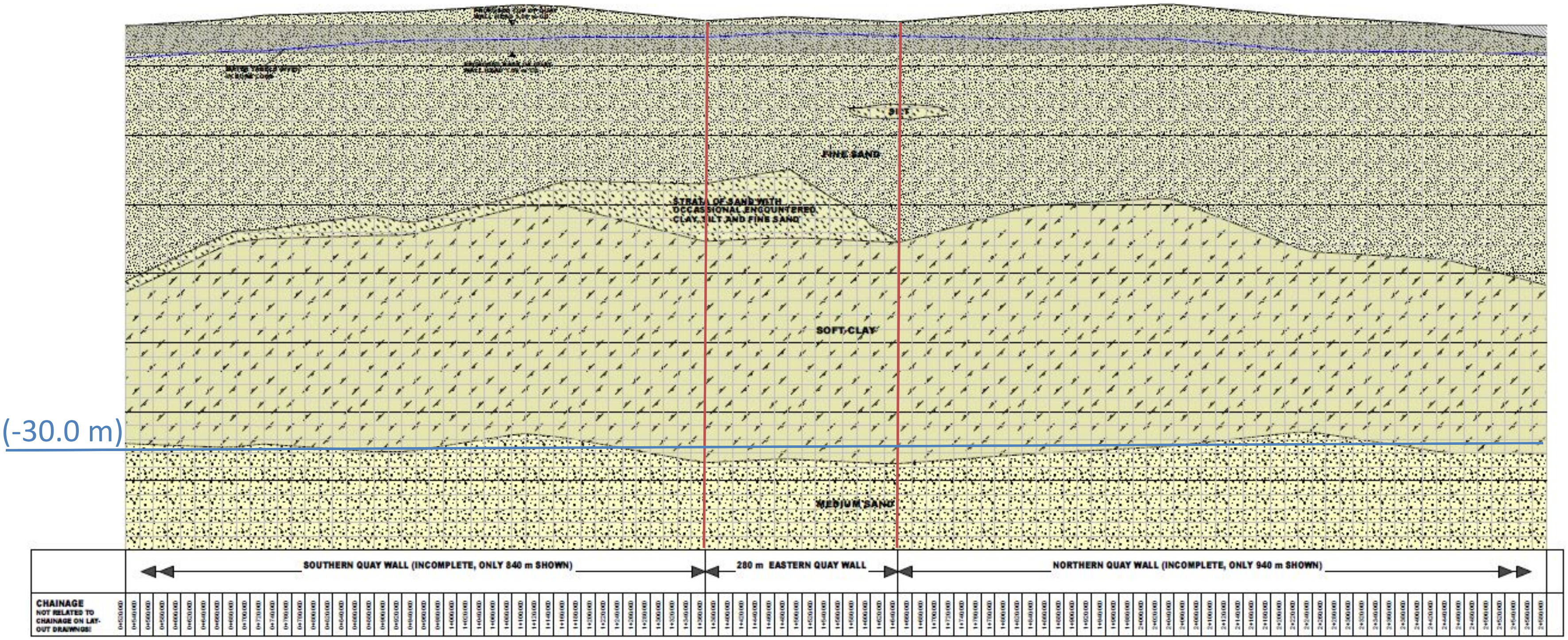
Design Challenges

1. Soil stratigraphy (Soft Clay)
   - Regional geology
   - Site investigation and soil profile

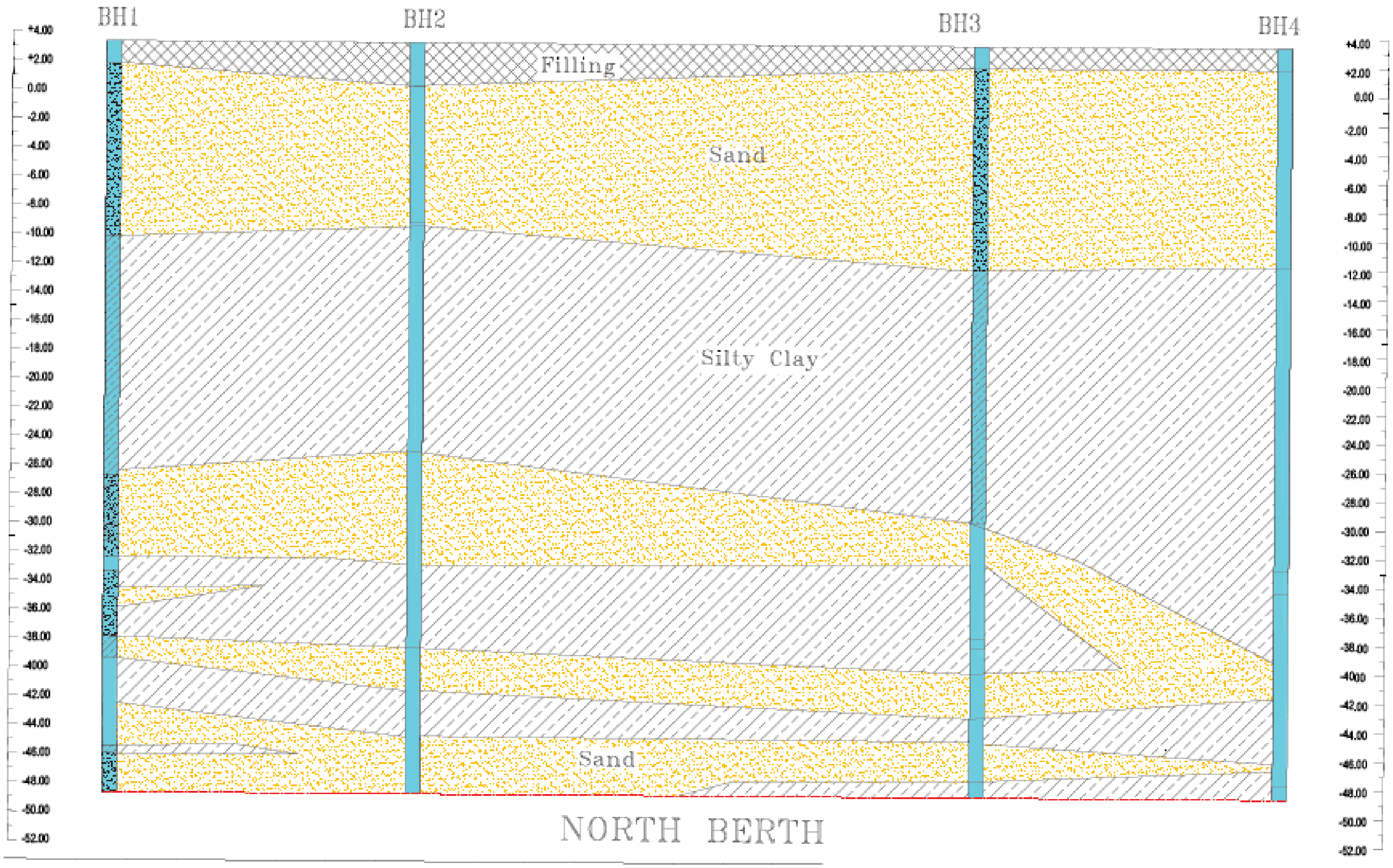


Design Challenges

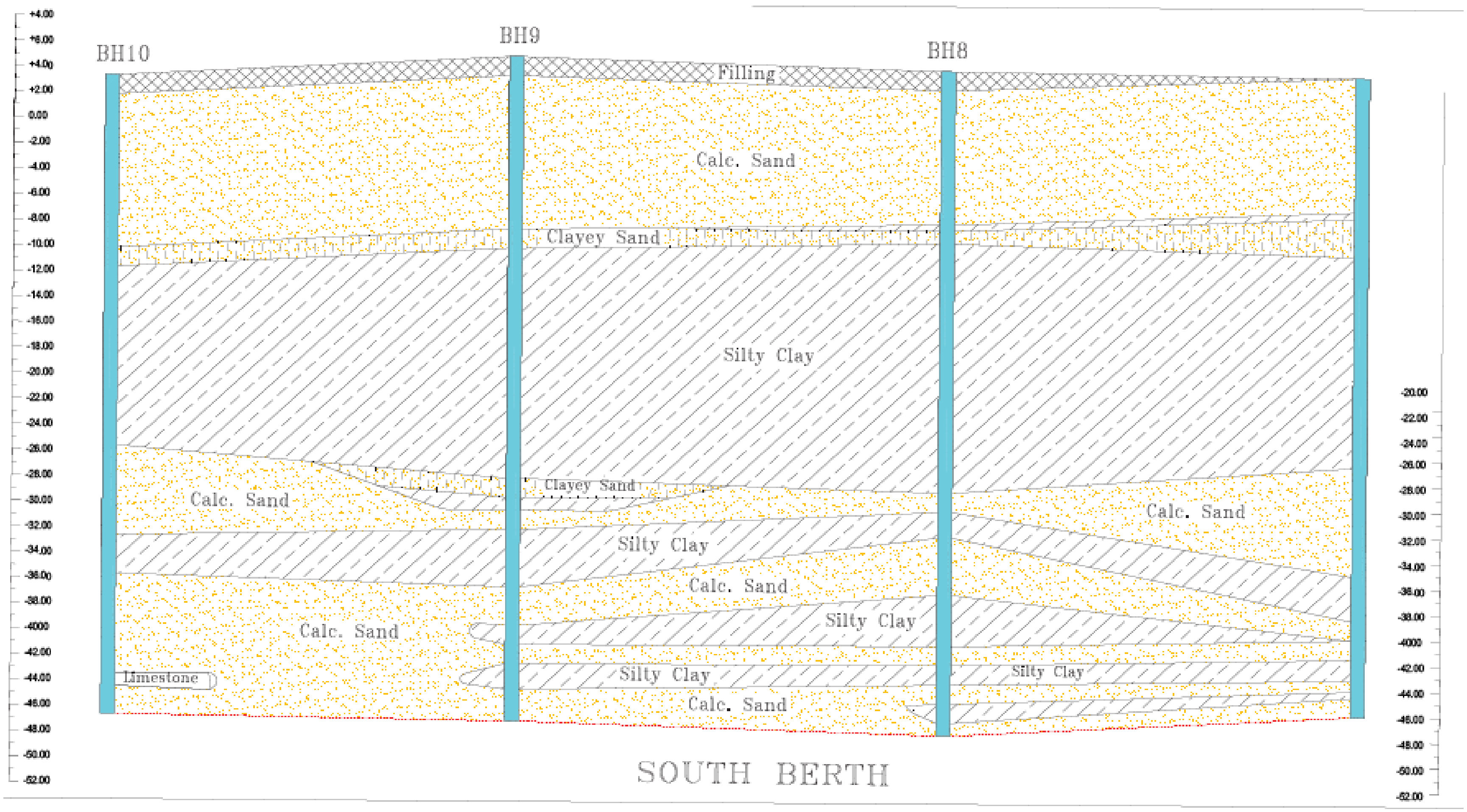
1. Soil stratigraphy (Soft Clay)
    - Regional geology
    - Site investigation and soil profile



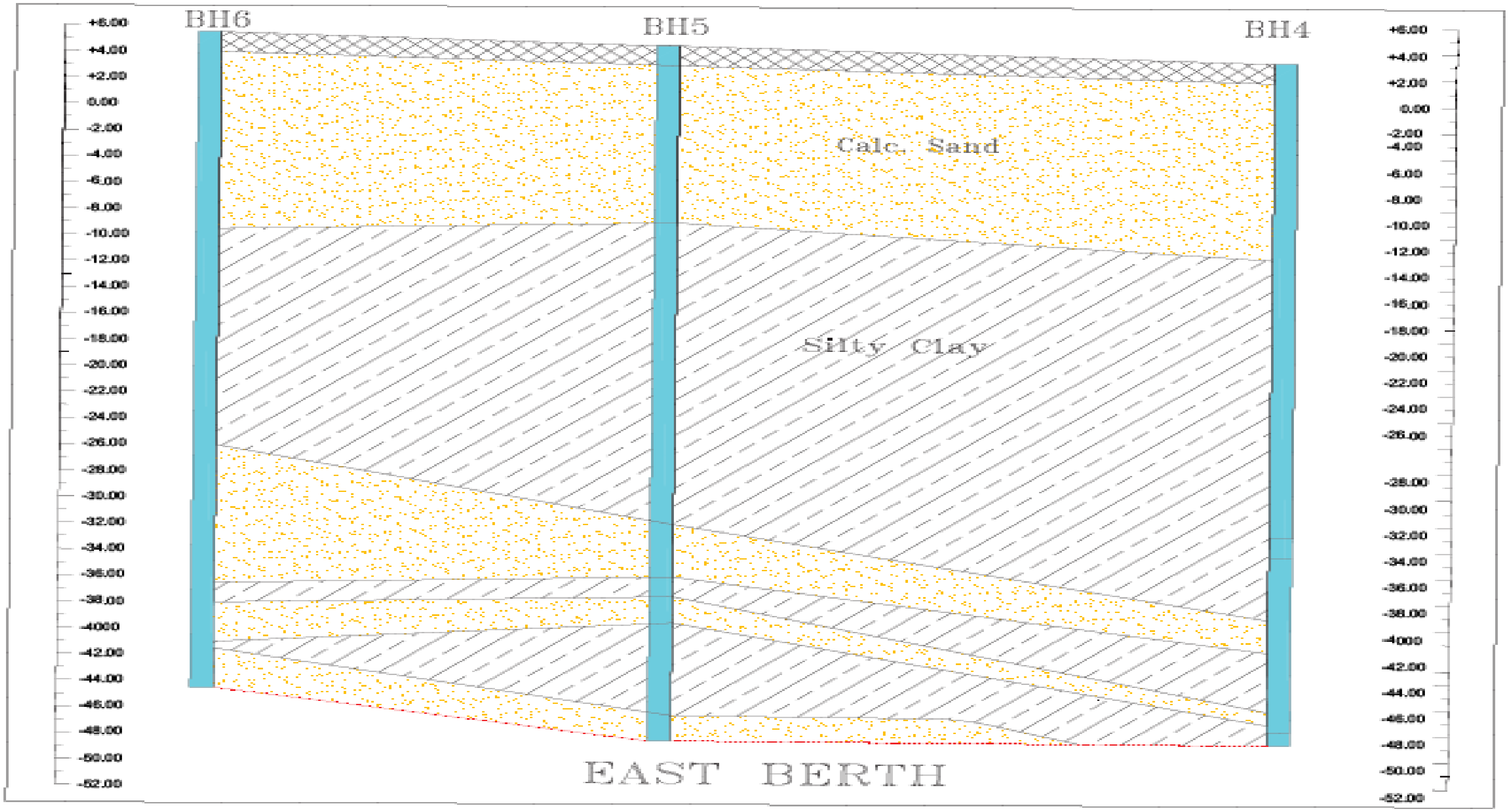
Soil Profile along North Quay Wall



Soil Profile along South Quay Wall

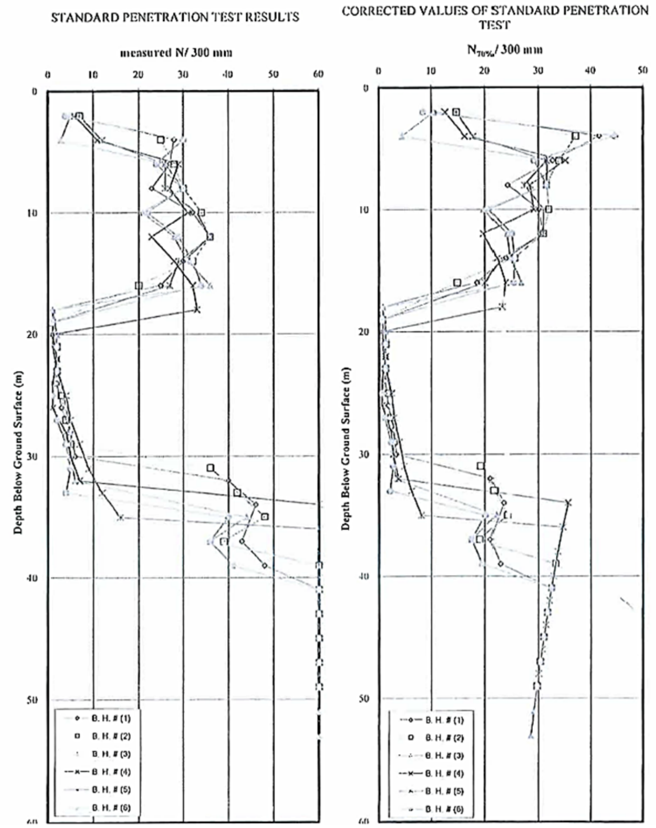


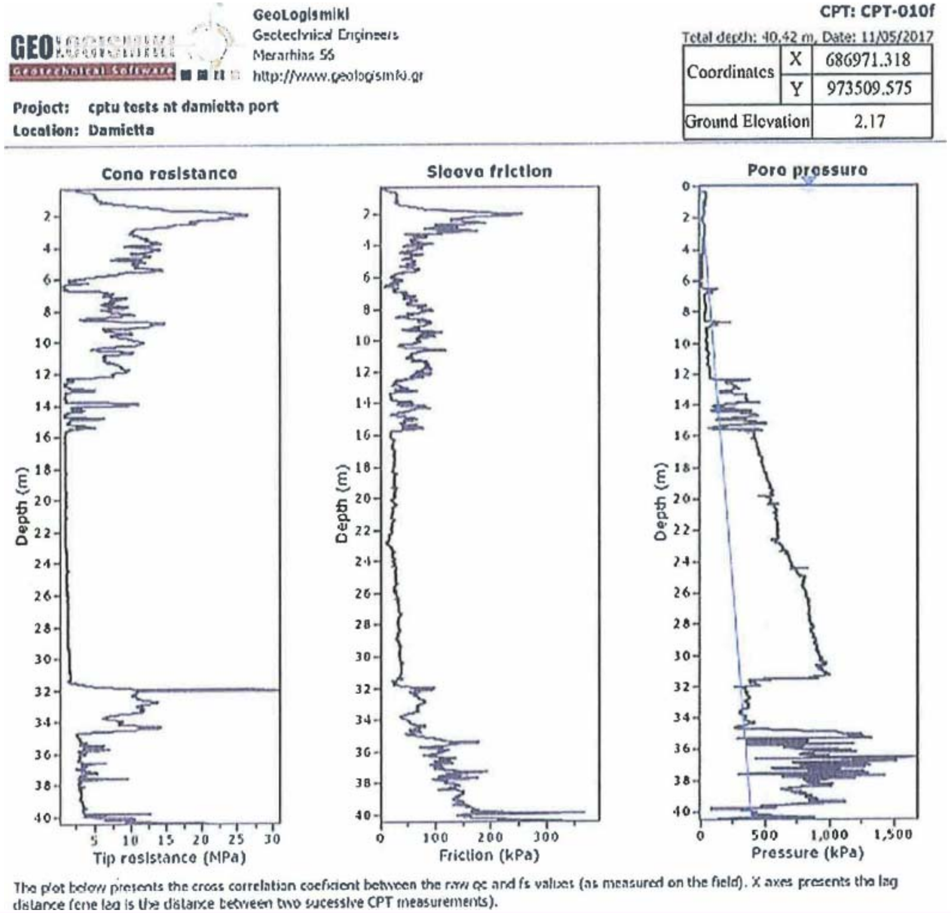
Soil Profile along East Quay Wall



Design Challenges

1. Soil stratigraphy (Soft Clay)
   - SPT and CPT Data





GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

CPT: CPT-010f

Total depth: 40.42 m, Date: 11/05/2017

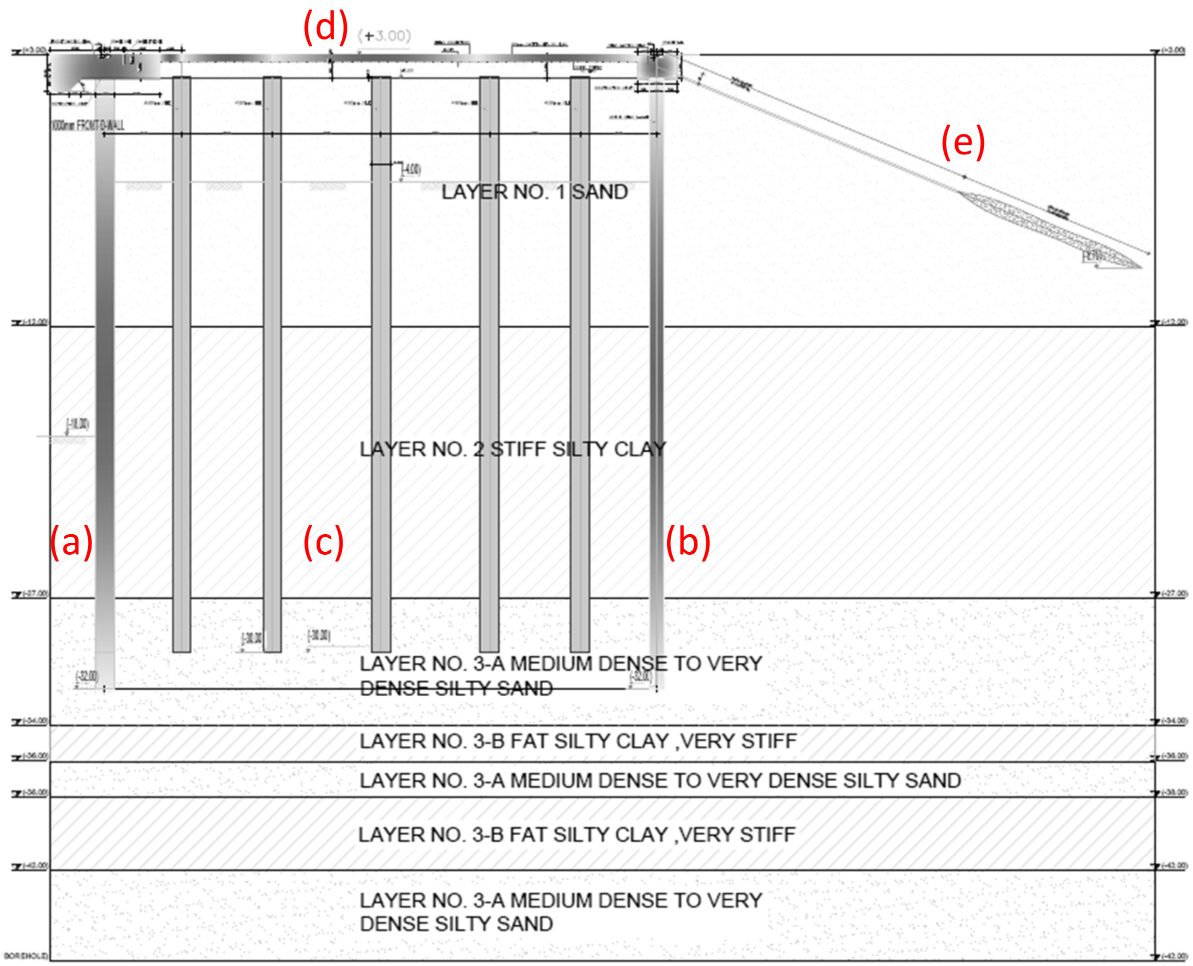
Coordinates	X	686971.318
	Y	973509.575
Ground Elevation		2.17

Project: cptu tests at damietta port
Location: Damietta

The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

Design Challenges

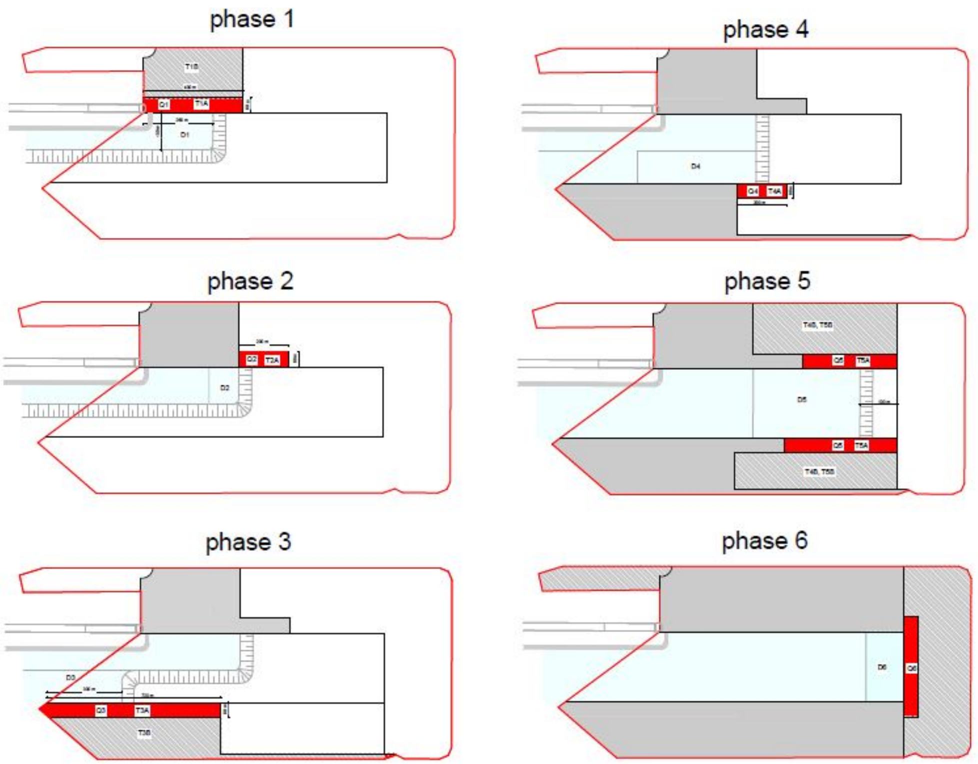
2. Old Design
   - Components
     a) Front DW
     b) Rear DW
     c) Piles
     d) Top Concrete slab
     e) Ground Anchors



Design Challenges

2. Old Design

   - Construction phases

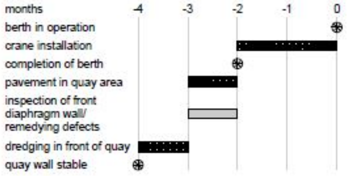


works phase	to be completed ...... days after commencement date
phase 1	420 days
phase 2	480 days
phase 3	510 days
phase 4	540 days
phase 5	600 days
phase 6	630 days

Q: Quay wall section ready for taking over
T: Terminal infrastructure section ready for use
D: Dreding completed

It has to be observed

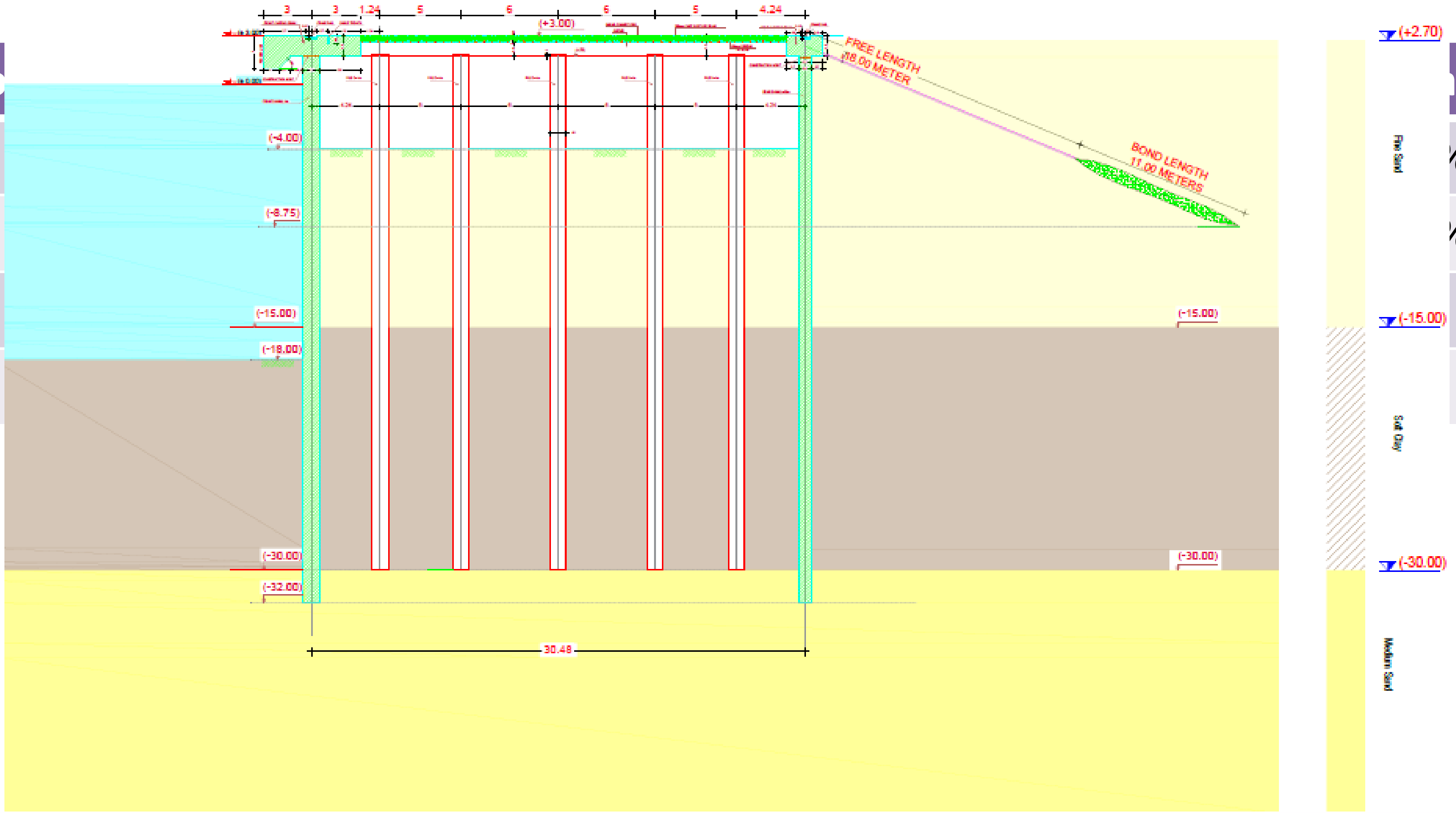
- that the quay wall has to be completed (stable) to carry the full horizontal load 60 days before completion date,
- that the terminal infrastructure contractor has to commence pavement and drainage works between front and back crane beams 30 days before completion date, and
- that the dredging contractor has to commence dredging in front of quay wall 60 days before completion date,
- that inspection of wall on water side and remedying works can only commence after dredging 30 days before completion date.



Design Challenges

2. Old Design

   - Constructed work



Co

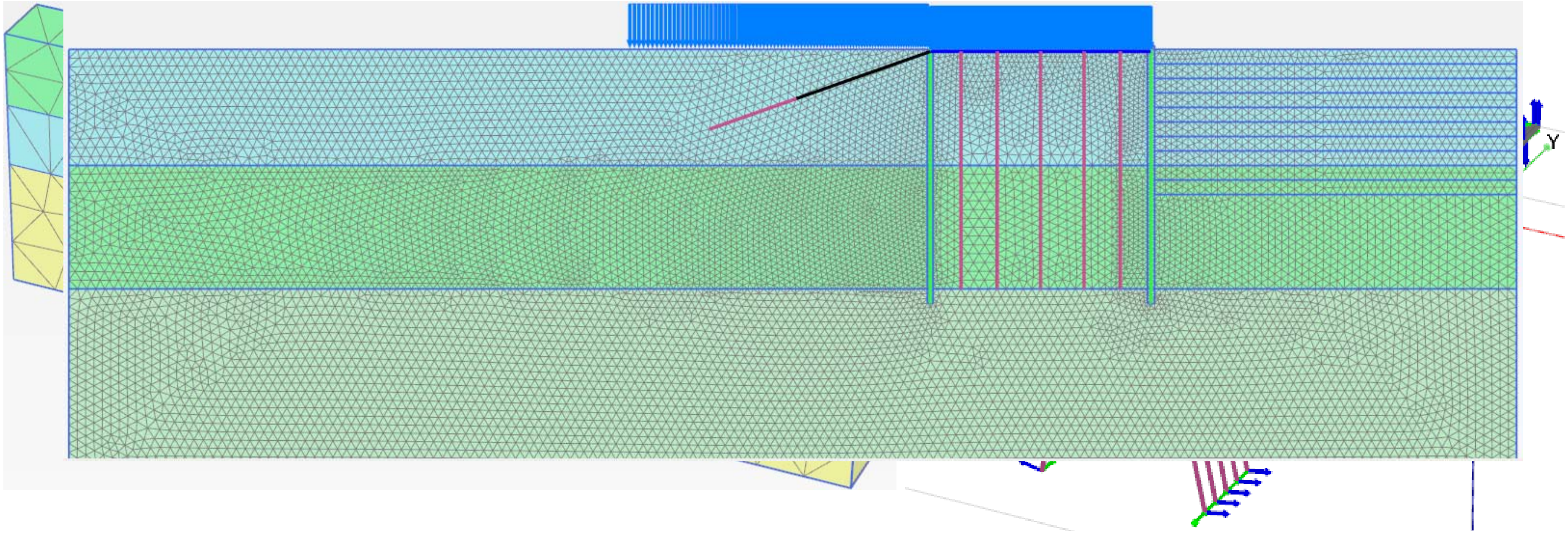
ll

%)

%)

Analysis of Old Design (Existing Construction)

- 2D and 3D FEA analyses
  - Undrained and drained conditions



Analysis of Existing Construction

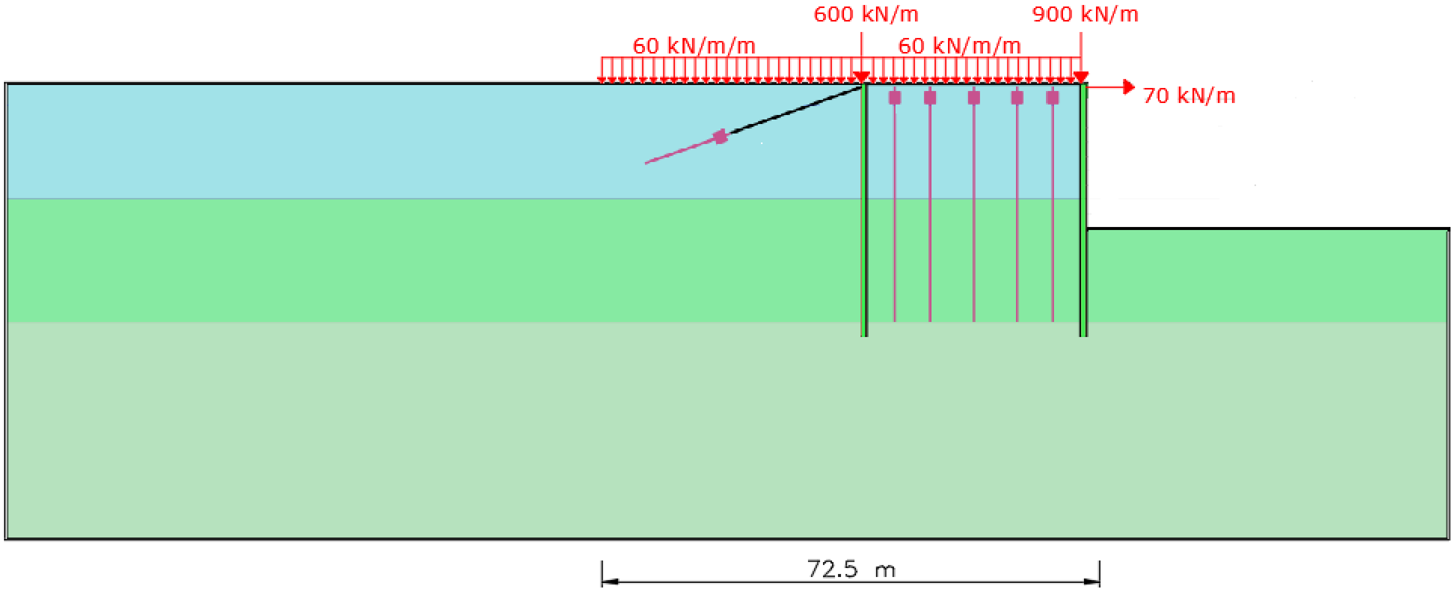
- Construction Stages

Phase title	Phase description
Initial equilibrium	Inserting the diaphragm walls and the piles
1st excavation*	Excavate from the ground surface (3.0 m) to level (1.0 m)
2nd excavation	Excavate to level (-1.0 m)
3rd excavation	Excavate to level (-3.0 m)
4th excavation	Excavate to level (-5.0 m)
5th excavation	Excavate to level (-7.0 m)
6th excavation	Excavate to level (-9.0 m)
7th excavation	Excavate to level (-11.0 m)
8th excavation	Excavate to level (-13.0 m)
9th excavation	Excavate to level (-15.0 m)
10th excavation	Excavate to level (-17.0 m)
Live load	Apply the live loads
Safety factor	Determine the global factor of safety of the problem

Analysis of Existing Construction

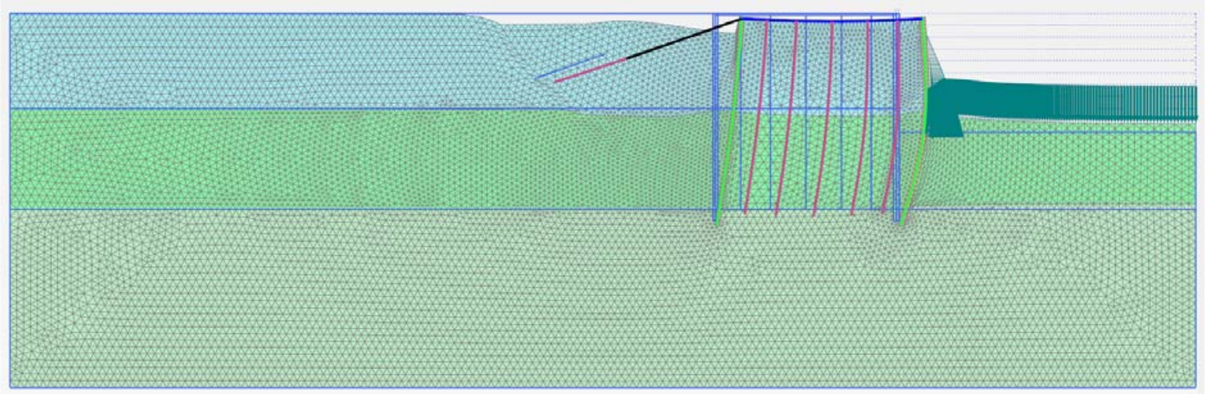
- 2D and 3D FEA analyses
  - Considered loads

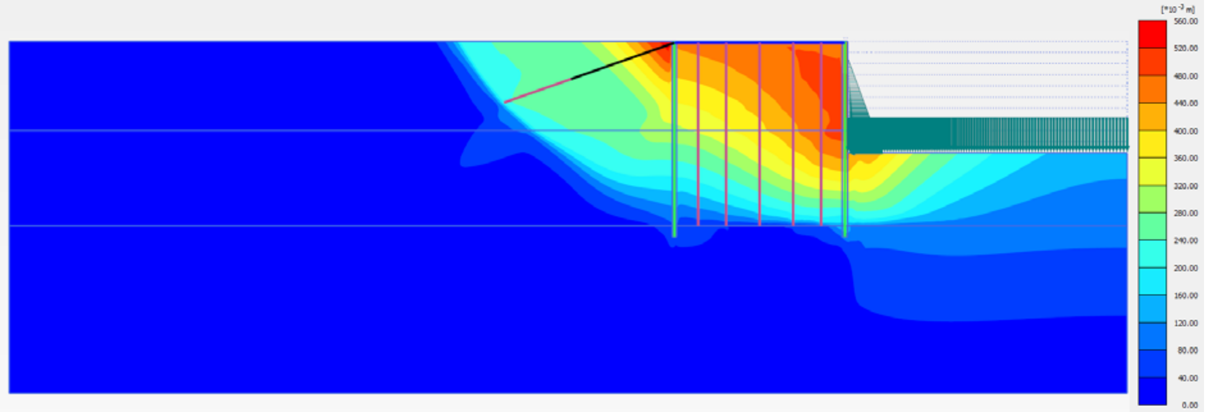
Load	Location	Value
Dead	-	Own self weight
Live	All berth	60 kN/m/m
Crane	Sea side	$P_V = 900$ kN/m
		$P_H = 70$ kN/m
	Land side	$P_V = 600$ kN/m



Analysis of Existing Construction)

- Results
  - Undrained analysis





Analysis of Existing Construction)

- Results

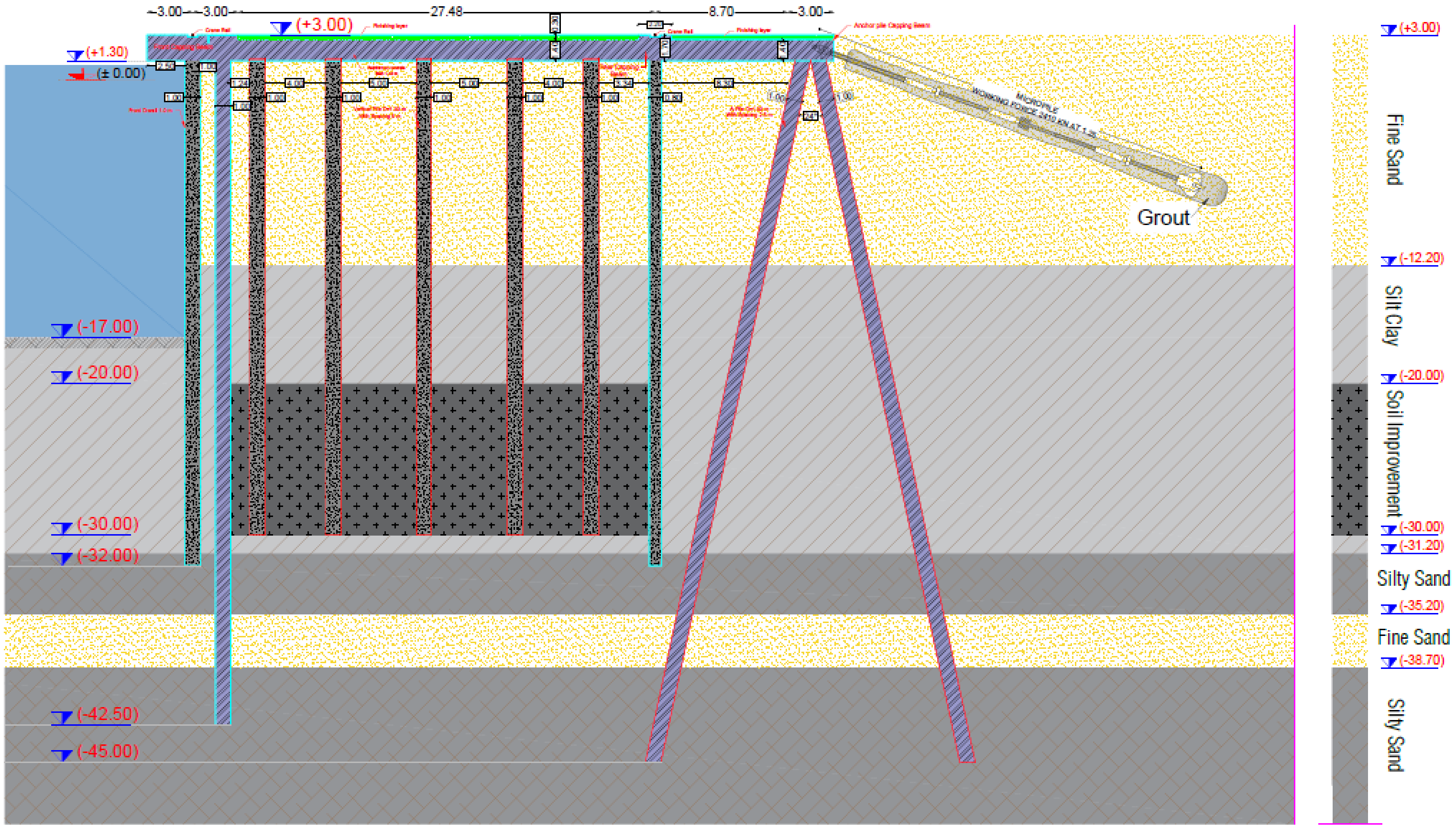
2D Analysis			
Structural element	Output	Undrained condition*	Drained condition
Front wall	Total displacement	**50.46 cm**	**31.62 cm**
	Max. axial force	-1277.0 kN/m	-2092.0 kN/m
	Max. shear force	688.5 kN/m	423.3 kN/m
	Max. bending moment	3918.0 kN.m/m	3474.0 kN.m/m
Back wall	Total displacement	44.43 cm	26.97 cm
	Max. axial force	-1189.0 kN/m	-1828.0 kN/m
	Max. shear force	162.3 kN/m	106.1 kN/m
	Max. bending moment	1004.0 kN.m/m	481.1 kN.m/m
Pile 1 (seaside)	Max. axial force	-964.5 kN	-1604.0 kN
	Max. shear force	211.2 kN	127.1 kN
	Max. bending moment	2037.0 kN.m	1259.5 kN.m
Pile 2	Max. axial force	-911.5 kN	-1986.0 kN
	Max. shear force	171.9 kN	94.6 kN
	Max. bending moment	1455.0 kN.m	901.5 kN.m
Pile 3	Max. axial force	-921.5 kN	-2209.5 kN
	Max. shear force	210.2 kN	157.2 kN
	Max. bending moment	1707.0 kN.m	949.0 kN.m
Pile 4	Max. axial force	-935.5 kN	-1918.0 kN
	Max. shear force	258.4 kN	121.2 kN
	Max. bending moment	1798.0 kN.m	652.0 kN.m
Pile 5 (landside)	Max. axial force	-864.5 kN	-1192.0 kN
	Max. shear force	144.0 kN	62.0 kN
	Max. bending moment	1321.5 kN.m	451.7 kN.m
Ground anchor	Max. axial force	680.3 kN	972.9 kN
-	Global F.S	**1.50**	**1.58**

Main Findings of Analysis of Existing Construction

- The analysis results for the undrained condition (i.e. immediately after excavation) indicate that the front wall would exhibit a maximum deformation of 50.5 cm due to the 20.0 m excavation and the model failed in the stage of applying the live loads (FS < 1 in short term).
- This means D-Walls will fail upon applying the live loads. The total pile loads would exceed the pile ultimate capacity, resulting in failure of all piles and leading to the failure of the entire structural system.
- The total tension axial force in the ground anchors would exceed 900 kN and failure would occur. Likewise, unacceptable performance was detected from the drained analysis.
- Thus, retrofitting solutions are proposed and analyzed.

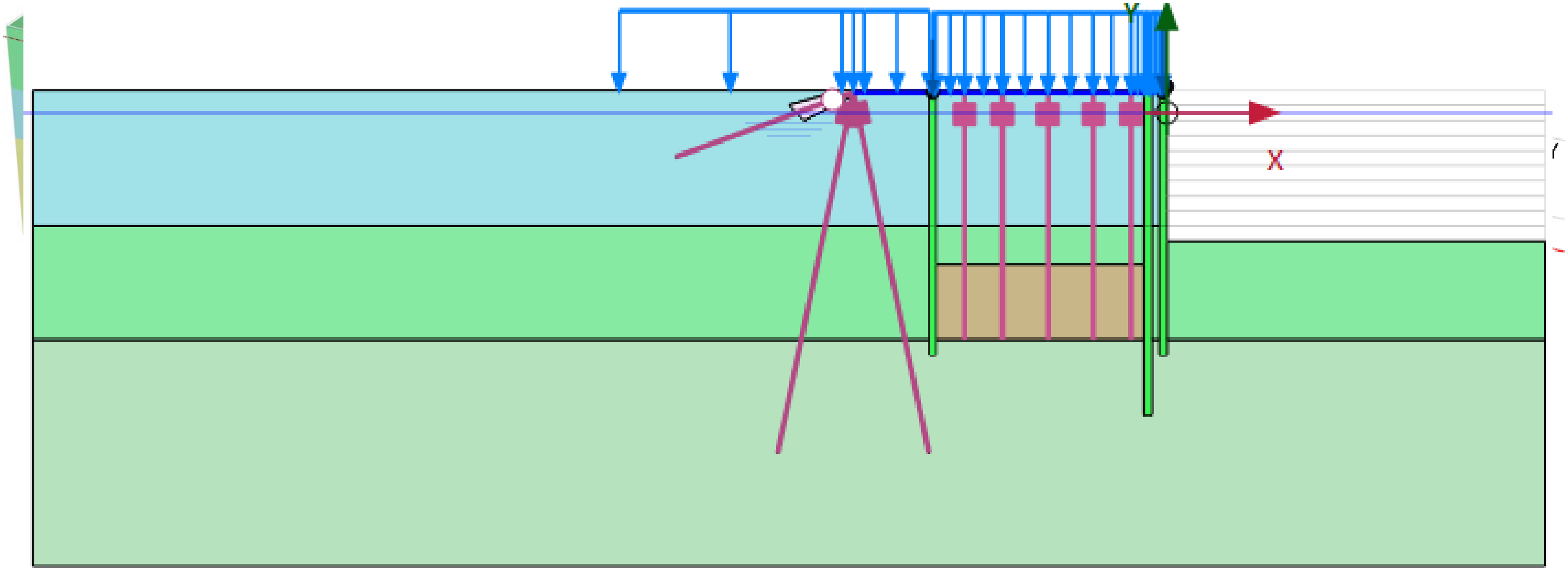
Proposed Designs (Retrofit)

- Design Alternative 1 (Design 1)



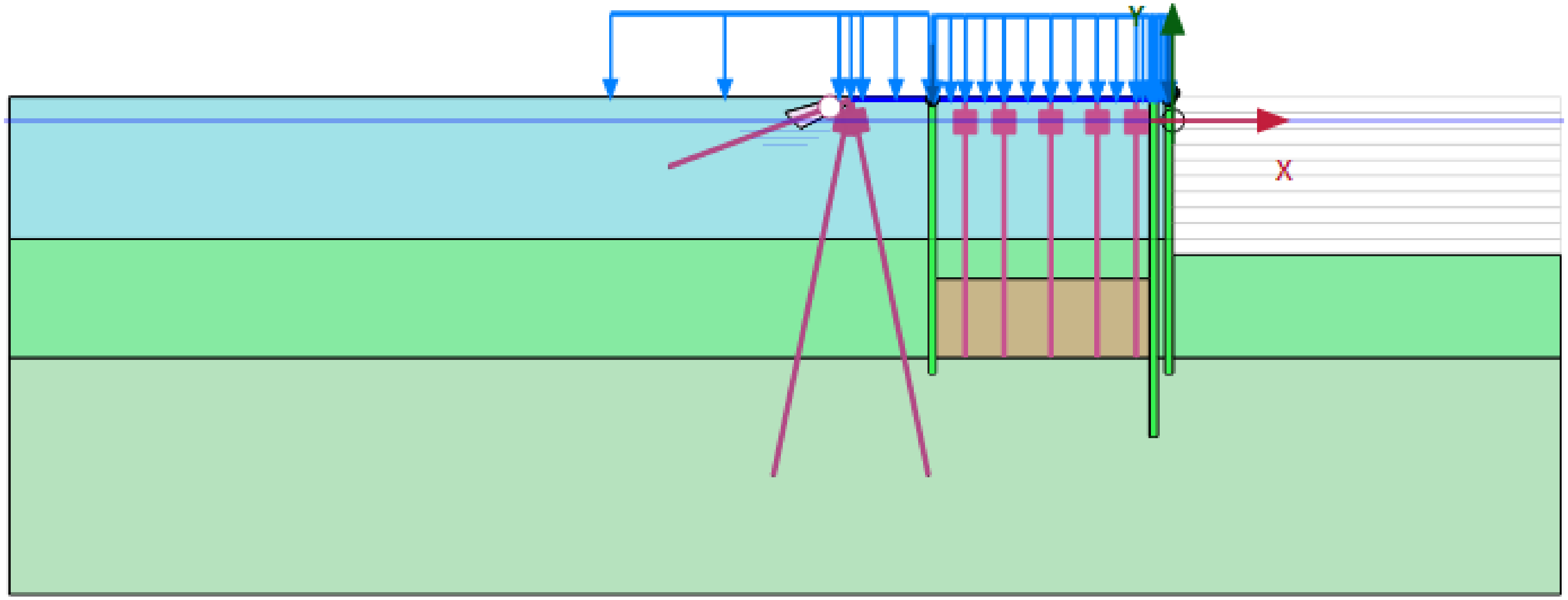
Proposed Designs (Retrofit)

- Design 1 analysis
  - 2D and 3D analyses (Drained and Undrained conditions)



Proposed Designs (Retrofit)

- Design 1 analysis
  - Construction stages



Clay fill [illegible]
Initial phase [illegible]

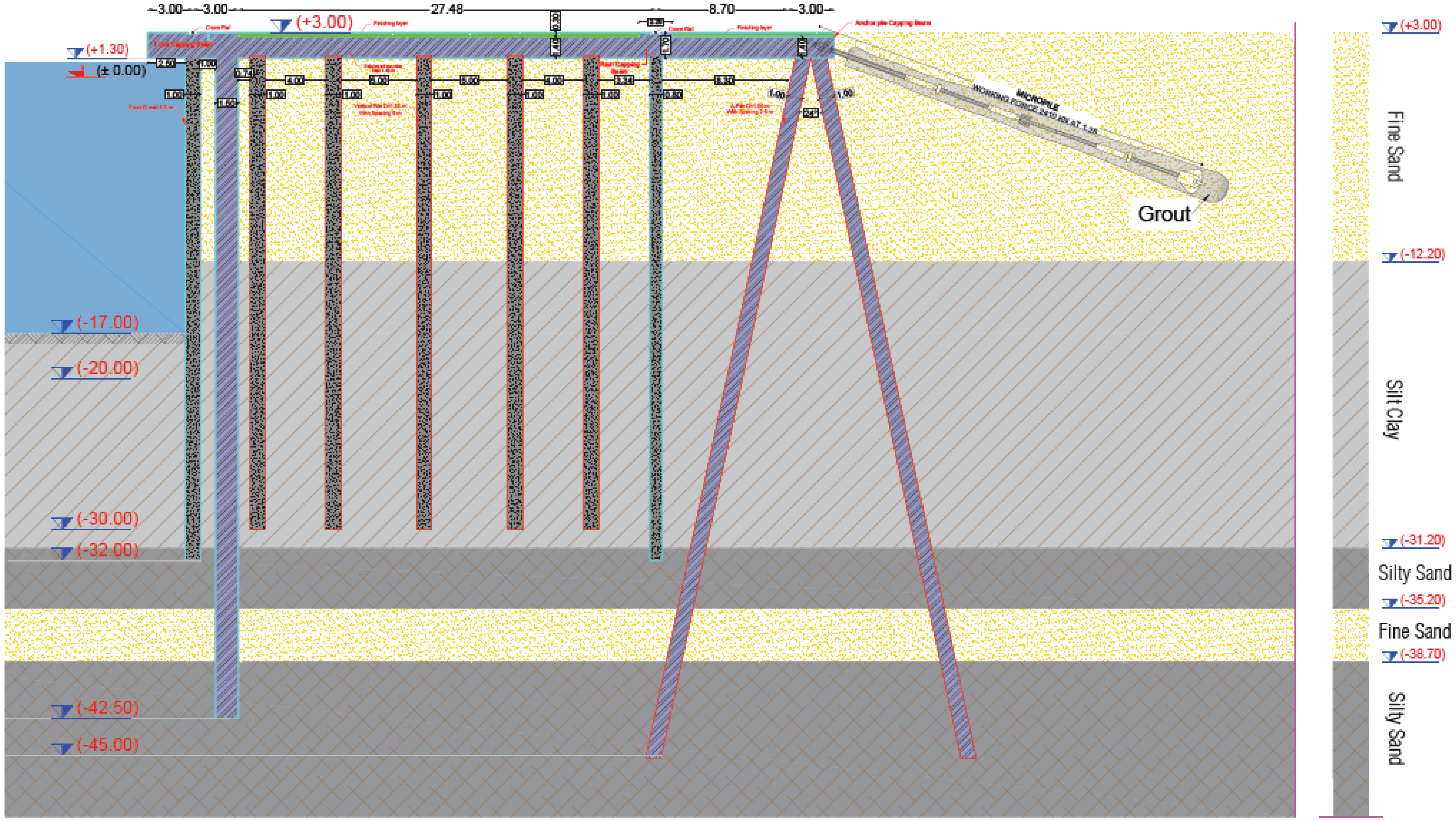
Proposed Designs (Retrofit)

- Design 1 analysis
  - Results

2D Analysis			
Structural element	Output	Undrained condition	Drained condition
Front wall 1	Total displacement	**20.18 cm**	**15.67 cm**
	Displacement due to excavation	**12.44 cm**	**9.43 cm**
Front wall 2	Total displacement	20.15 cm	15.69 cm
	Displacement due to excavation	12.48 cm	9.19 cm
Back wall	Total displacement	17.70 cm	14.43 cm
	Displacement due to excavation	8.90 cm	7.78 cm
Pile 1 (seaside)	Max. axial force	-1790.5 kN	-1037.5 kN
	Max. shear force	113.4 kN	120.8 kN
	Max. bending moment	780.5 kN.m	641.5 kN.m
Pile 5 (landside)	Max. axial force	-964.5 kN	-990.0 kN
	Max. shear force	99.9 kN	64.4 kN
	Max. bending moment	443.4 kN.m	281.2 kN.m
A-frame pile 1	Max. axial force	-3447.5 kN	-3547.5 kN
	Max. shear force	61.1 kN	101.2 kN
	Max. bending moment	395.3 kN.m	602.3 kN.m
A-frame pile 2	Max. axial force	+1079.8 kN	-1534.3 kN
	Max. shear force	106.4 kN	144.1 kN
	Max. bending moment	575.5 kN.m	753.3 kN.m
Micropile	Max. axial force	+770.8 kN	+782.0 kN
	Max. shear force	211.7 kN	188.5 kN
	Max. bending moment	203.3 kN.m	160.4 kN.m
-	Global F.S	**2.632**	**2.669**

Proposed Designs (Retrofit)

- Design Alternative 2 (Design 2)

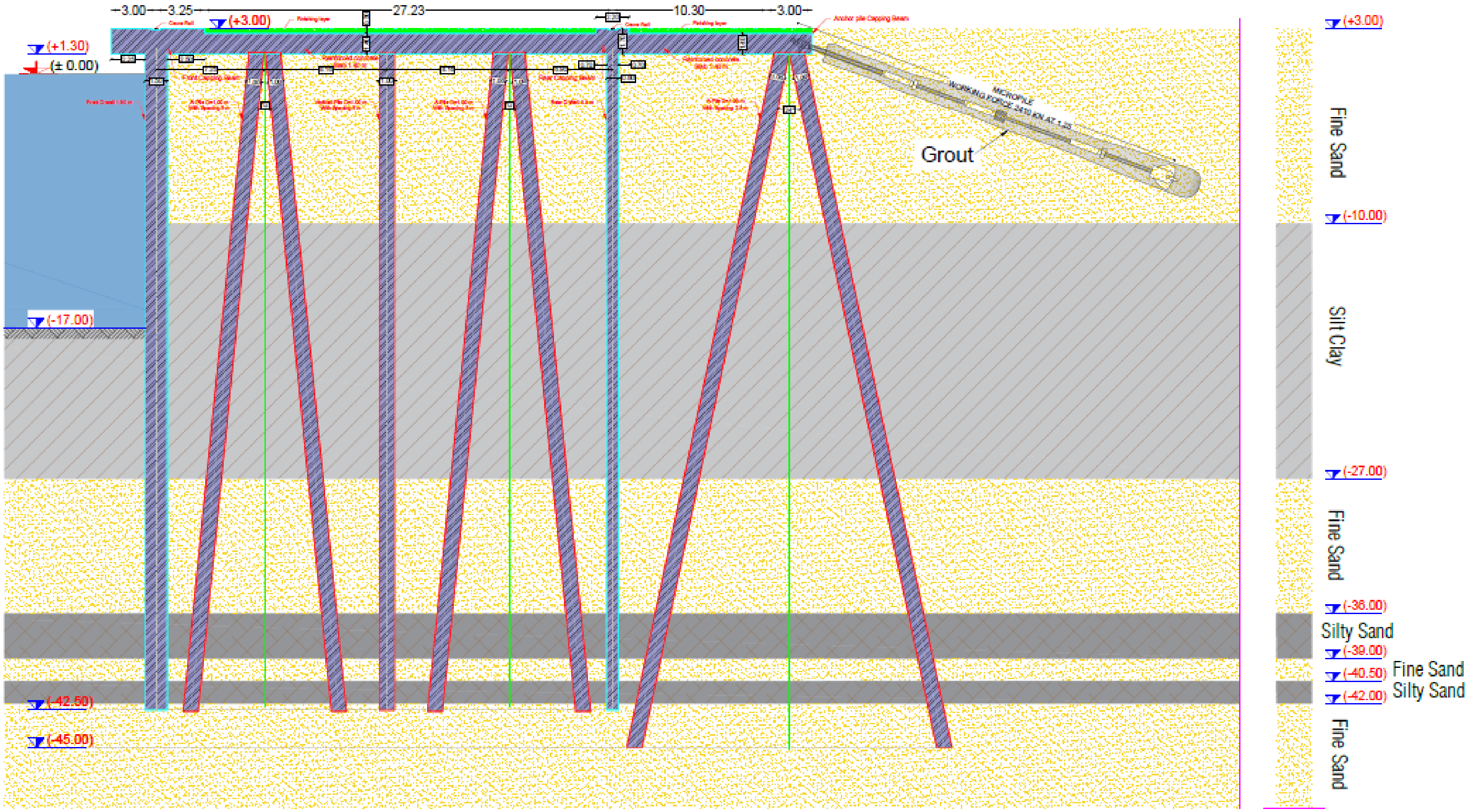


Proposed Designs (Retrofit)

- Design 2 Analysis
  - Results

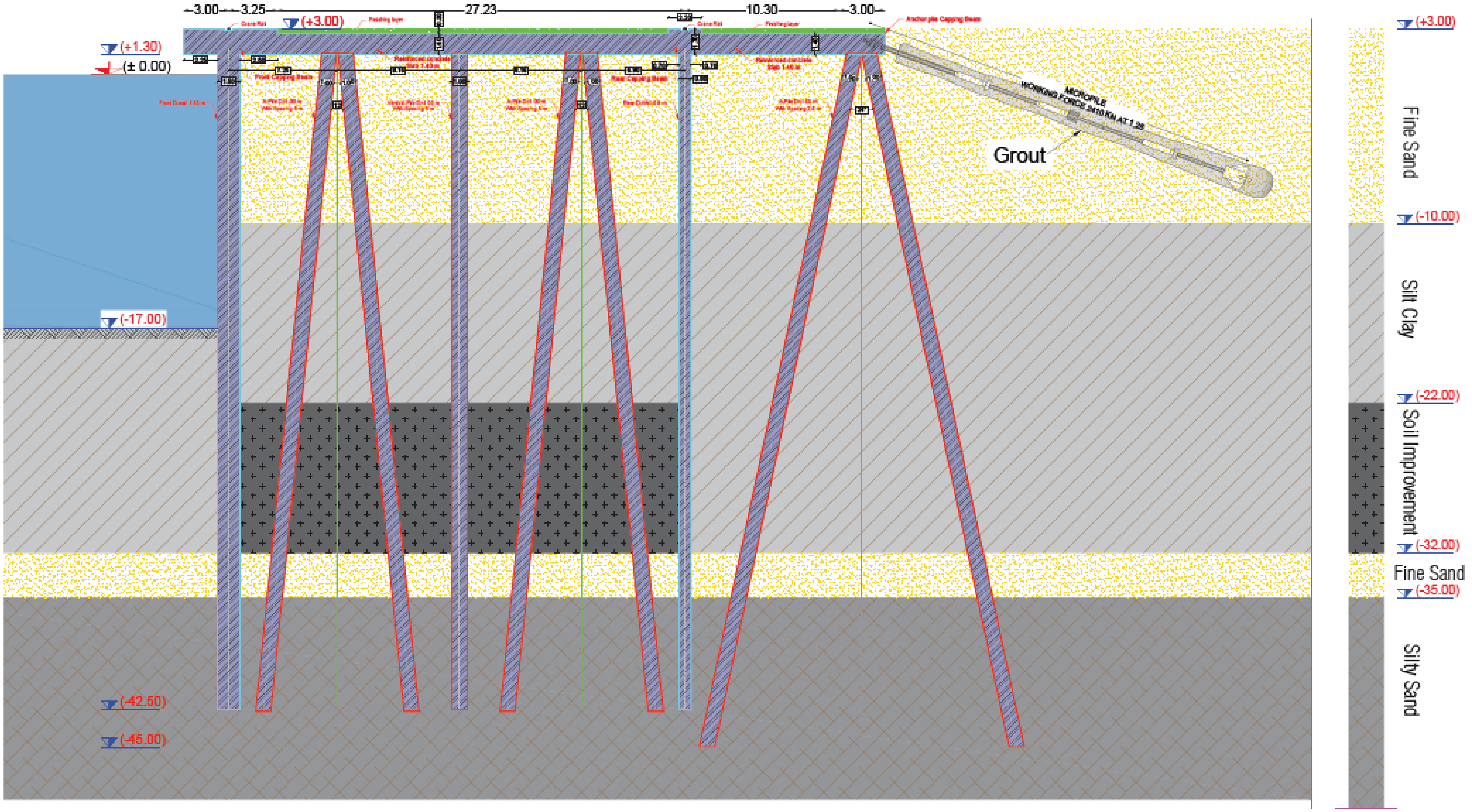
2D Analysis			
Structural element	Output	Undrained condition	Drained condition
Front wall 1	Total displacement	**27.88 cm**	**17.67 cm**
	Displacement due to excavation	**14.86 cm**	**10.74 cm**
Front wall 2	Total displacement	27.90 cm	17.47 cm
	Displacement due to excavation	14.78 cm	10.18 cm
Back wall	Total displacement	26.91 cm	17.01 cm
	Displacement due to excavation	12.64 cm	8.40 cm
Pile 1 (seaside)	Max. axial force	-401.9 kN	-334.3 kN
	Max. shear force	84.5 kN	86.0 kN
	Max. bending moment	906.5 kN.m	593.0 kN.m
Pile 5 (landside)	Max. axial force	-1067.5 kN	-671.5 kN
	Max. shear force	60.5 kN	36.7 kN
	Max. bending moment	524.0 kN.m	271.2 kN.m
A-frame pile 1	Max. axial force	-4285.0 kN	-3837.5 kN
	Max. shear force	244.4 kN	144.2 kN
	Max. bending moment	1388.3 kN.m	784.0 kN.m
A-frame pile 2	Max. axial force	+1371.5 kN	-1274.5 kN
	Max. shear force	214.1 kN	181.2 kN
	Max. bending moment	1247.0 kN.m	917.5 kN.m
Micropile	Max. axial force	+775.3 kN	+783.3 kN
	Max. shear force	234.8 kN	196.5 kN
	Max. bending moment	248.0 kN.m	174.0 kN.m
-	Global F.S	**2.376**	**2.497**

Proposed Designs (New Construction)



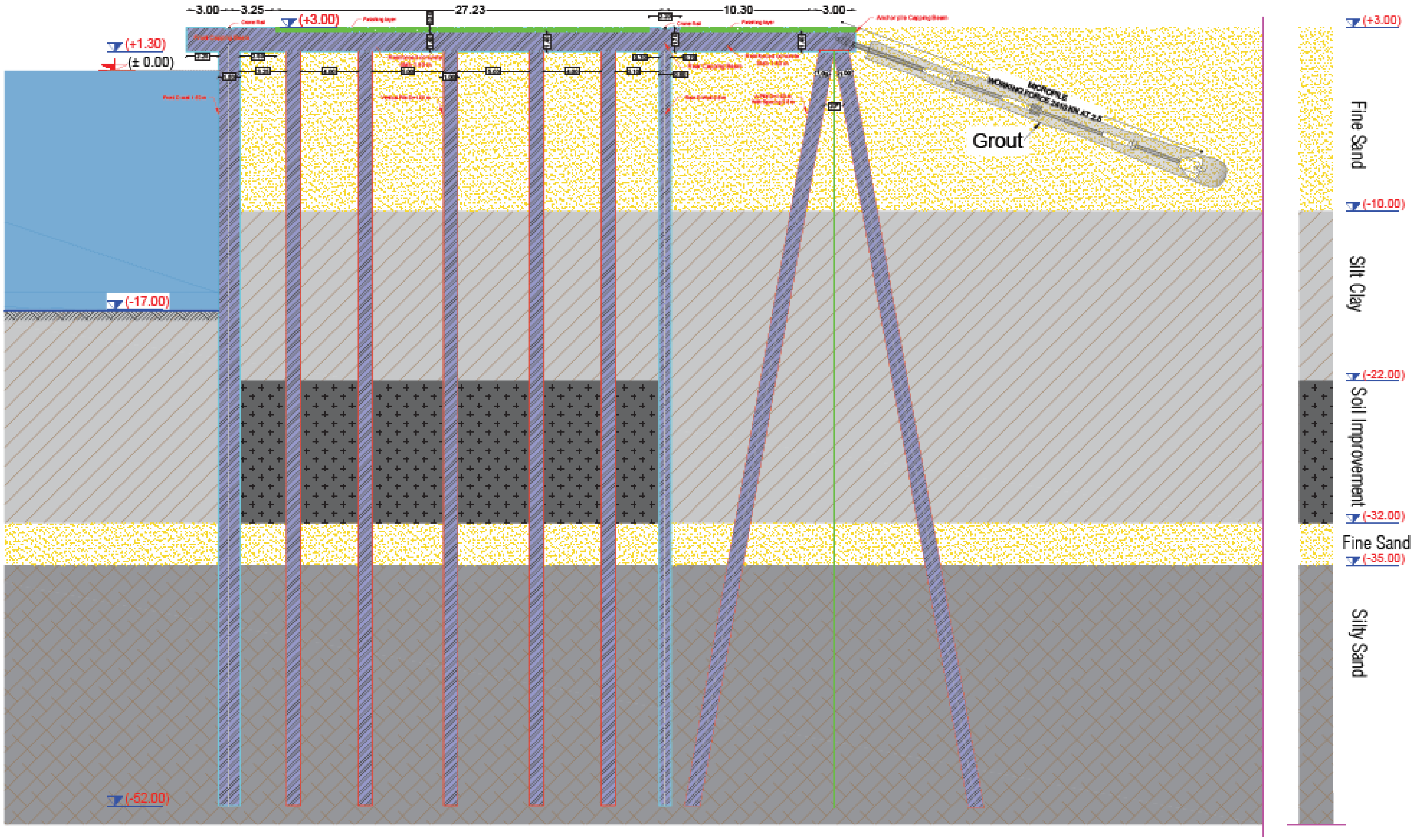
Schematic for New Construction 2 (clay layer 17.0 m thick)

Proposed Designs (New Construction)



Schematic for New Construction 2 (clay layer 22.0 m thick)

Proposed Designs (New Construction)



Schematic for New Construction 2 (clay layer 17.0 m thick)

Proposed Monitoring Plan

- Detailed monitoring plan measurements:
  1. Deformations of the diaphragm wall panels.
  2. Excavation-induced ground movements of the adjacent soils.
  3. Straining actions in different structural elements (e.g. A-frame piles and micropiles).
  4. Variations in the applied lateral earth pressures.
  5. Water and piezometric levels.

❖ Data is to be collected during the ongoing construction activities and throughout the lifetime of the project.

Proposed Monitoring Plan

- Detailed monitoring plan Instruments:
  1. Inclinometers.
  2. Shape accelerometer arrays.
  3. Probe extensometers
  4. Pressure cells
  5. Strain gauges
  6. Piezometers.
  7. Precision survey monitoring

Proposed Monitoring Plan

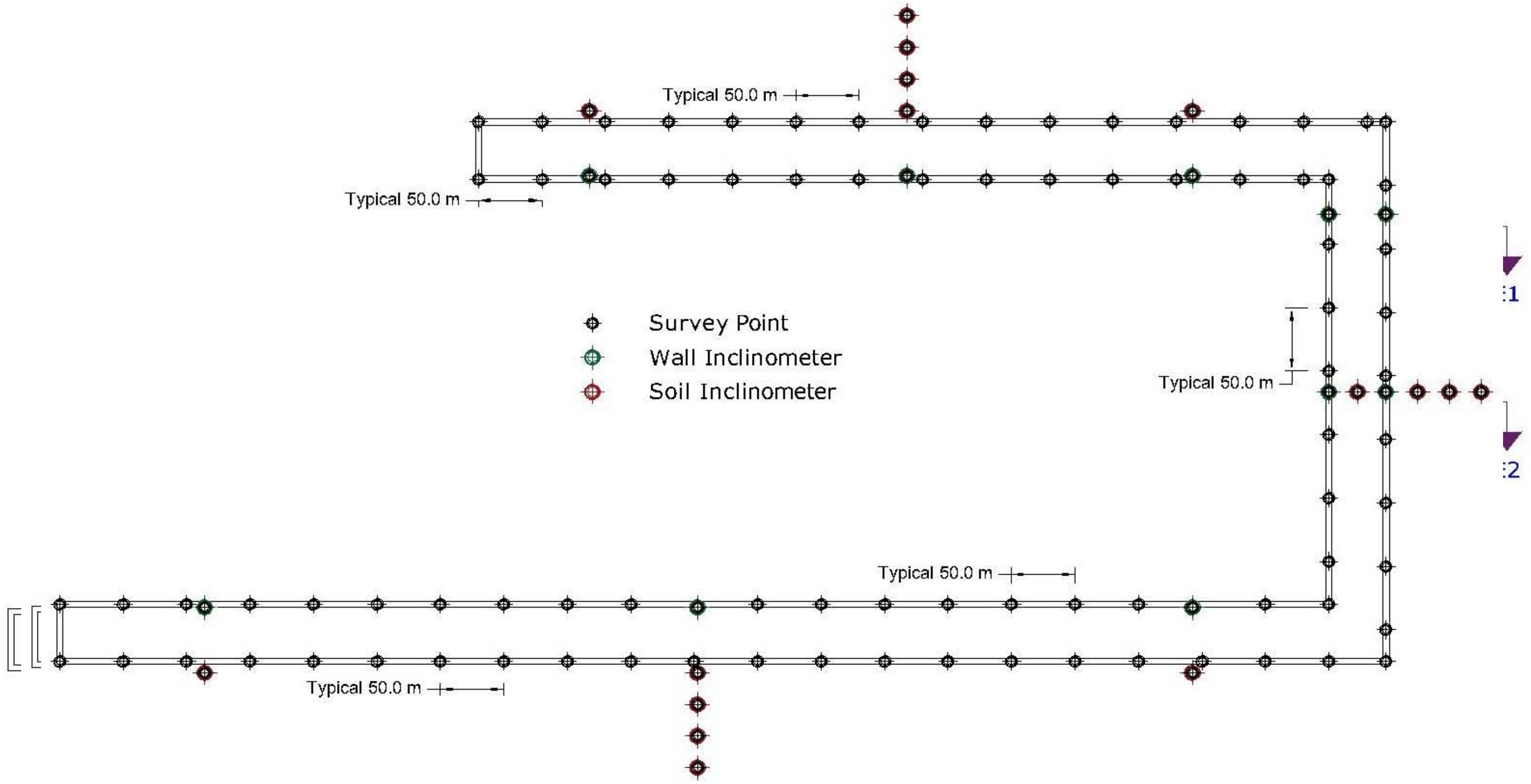
Inclinometers; Shape accelerometer arrays; Probe extensometers; Pressure cells; Strain gauges; Piezometers; Precision survey monitoring

Summary of the monitoring instruments.

Instrument	Total number	Comment
Wall inclinometer	10	-
Soil inclinometer	16	-
SAAF	16	-
Extensometer	3	Horizontal or inclined up to 7.5° with the horizontal.
Piezometer	13	-
Pressure cell	37	-
Sister bar strain gauge	924	3 groups of 7 A-frame piles.
Vibrating wire strain gauge	40	5 micropile anchors.
Survey points	272	Include survey points on D-Walls, inclinometers and 40 groups of A-frame piles.

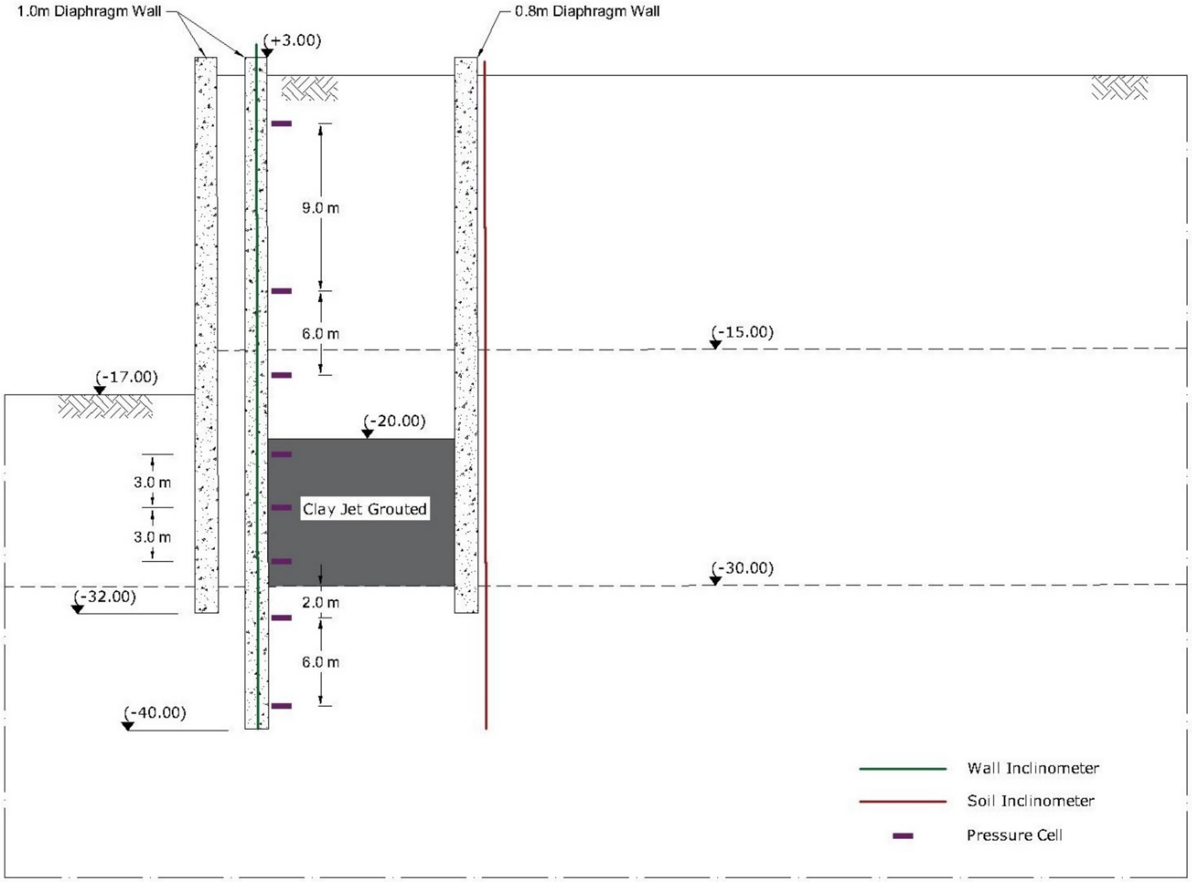
Proposed Monitoring Plan

- Detailed monitoring plan sections:



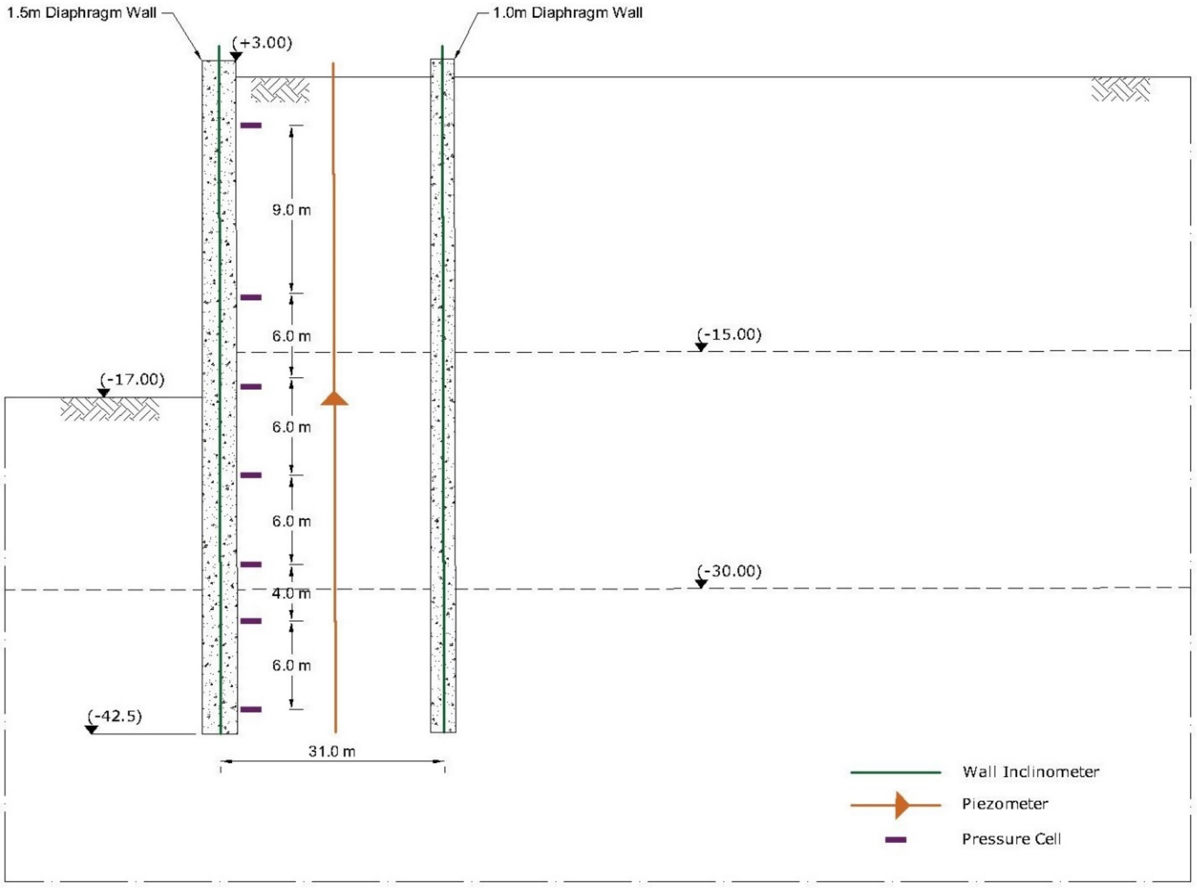
Proposed Monitoring Plan

- Detailed monitoring plan sections:



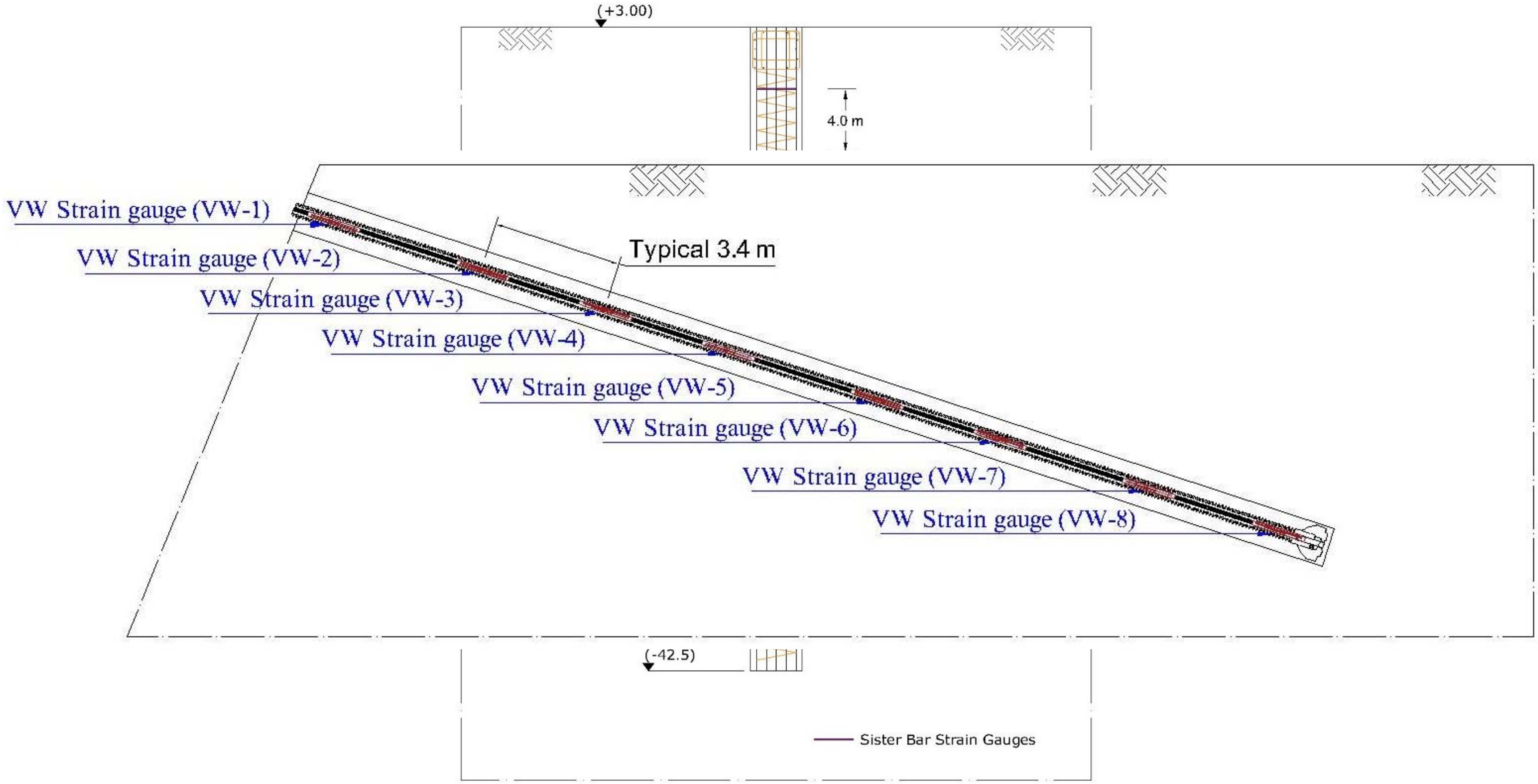
Proposed Monitoring Plan

- Detailed monitoring plan sections:



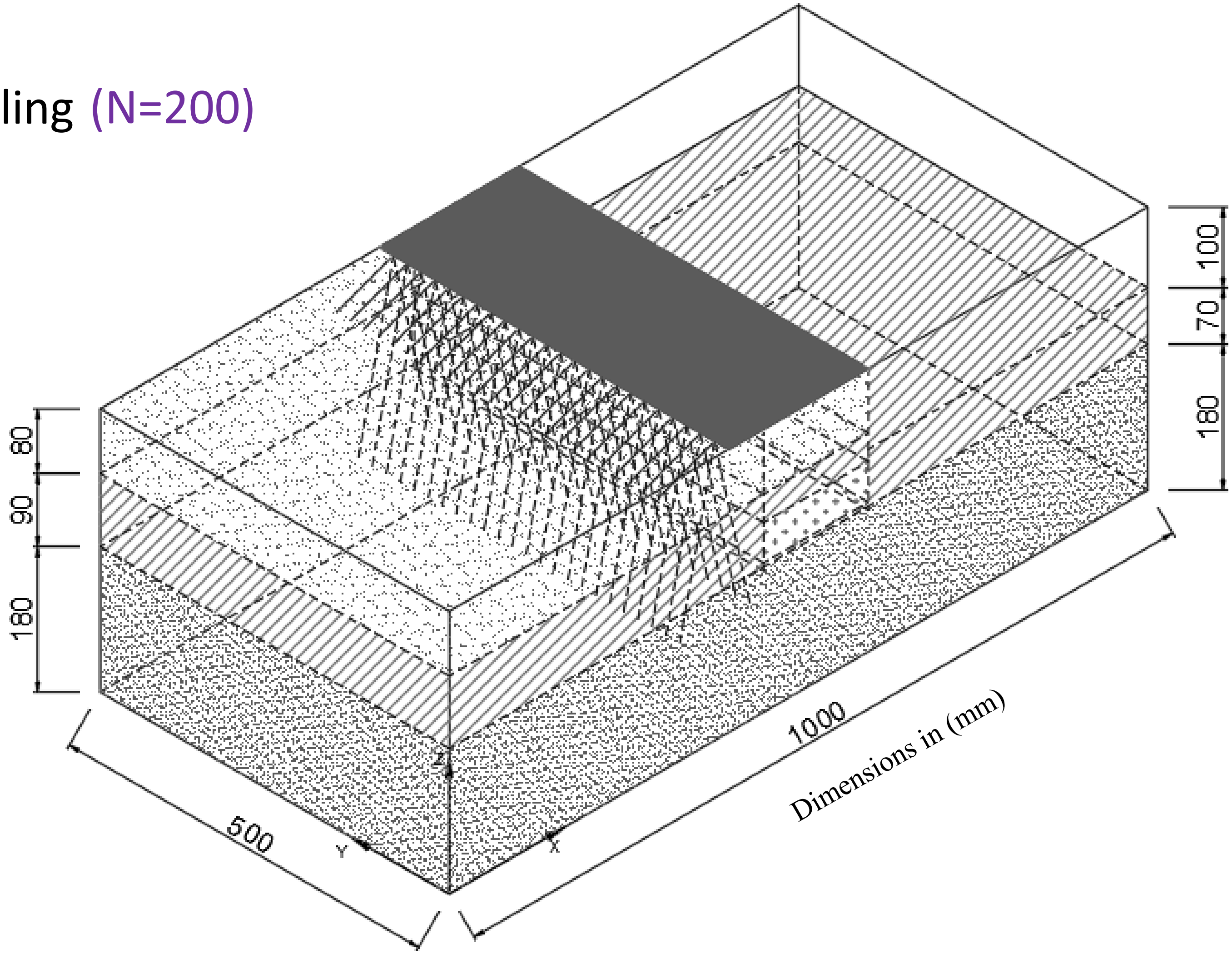
Proposed Monitoring Plan

- Detailed monitoring plan sections:



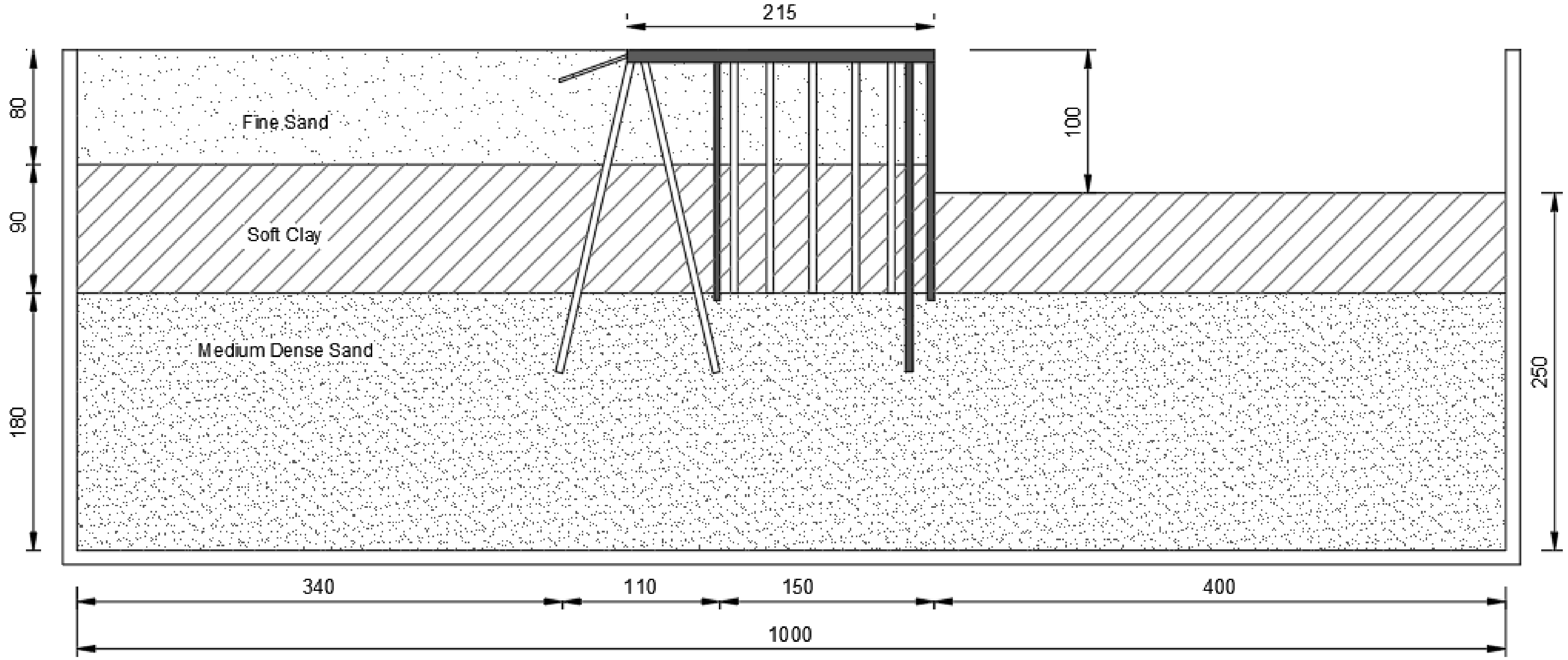
Centrifuge Modelling for the Retaining System

- Model Scaling (N=200)



Centrifuge Modelling for the Retaining System

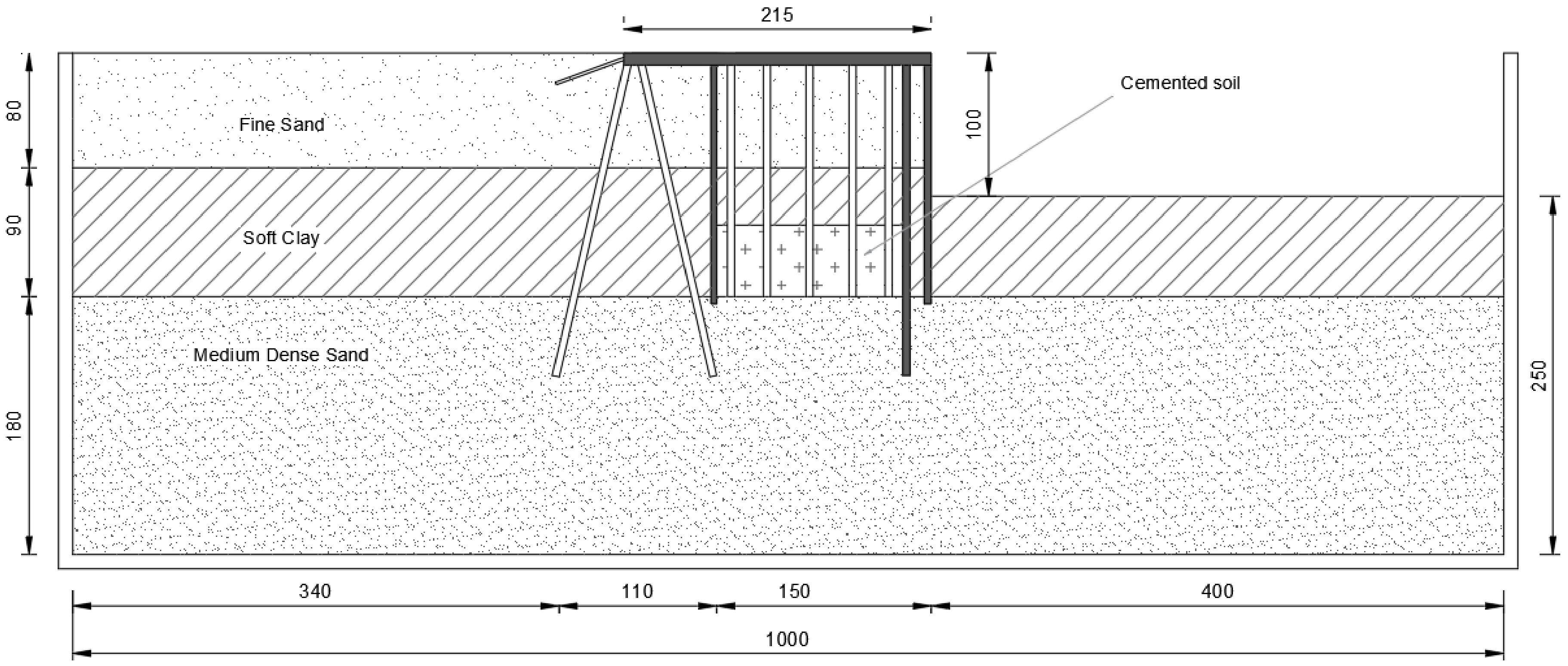
- Retaining Existing Soil Stratigraphy



Dimensions in (mm)

Centrifuge Modelling for the Retaining System

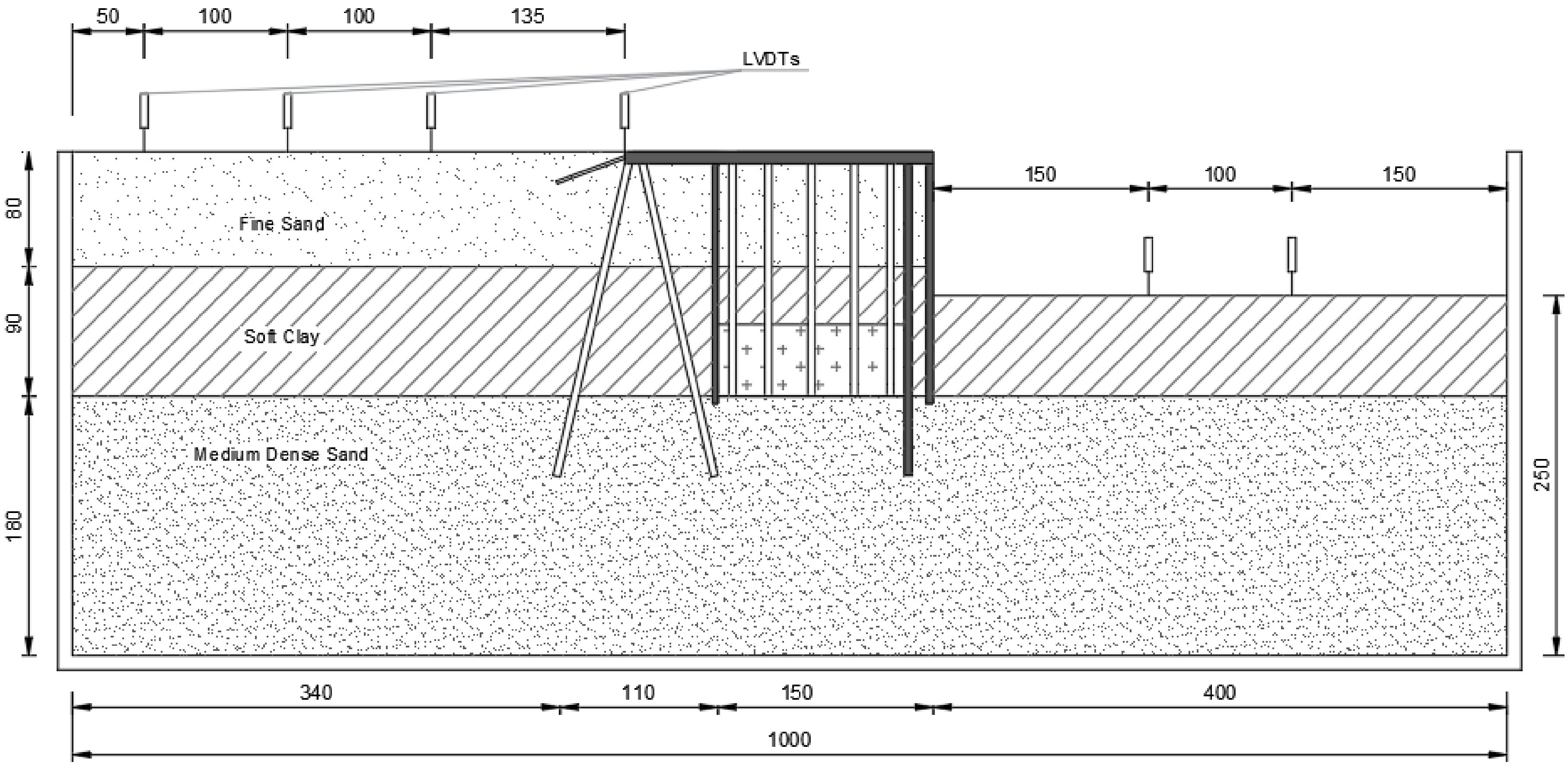
- Retaining + Soil Improvement



Dimensions in (mm)

Test Instrumentation

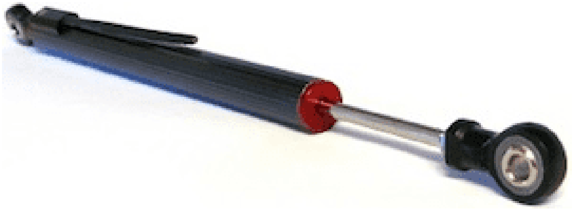
1- Linear Variable Differential Transducers (LVDT)

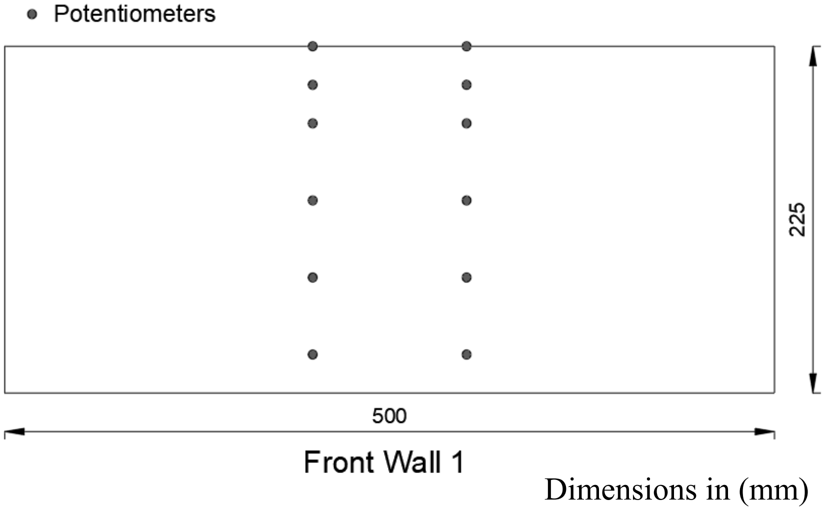


Dimensions in (mm)

Test Instrumentation

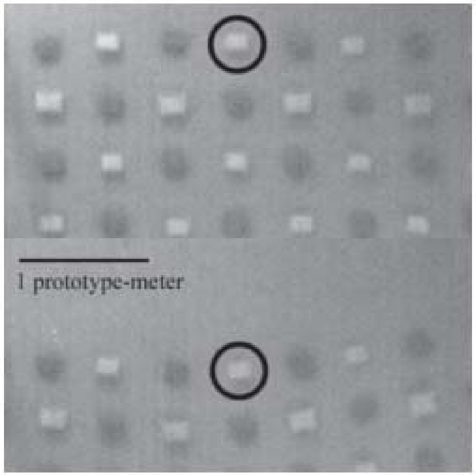
2- Linear Potentiometer





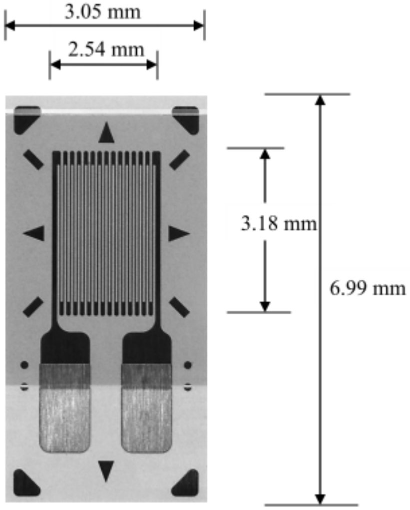
3- Camera Monitoring

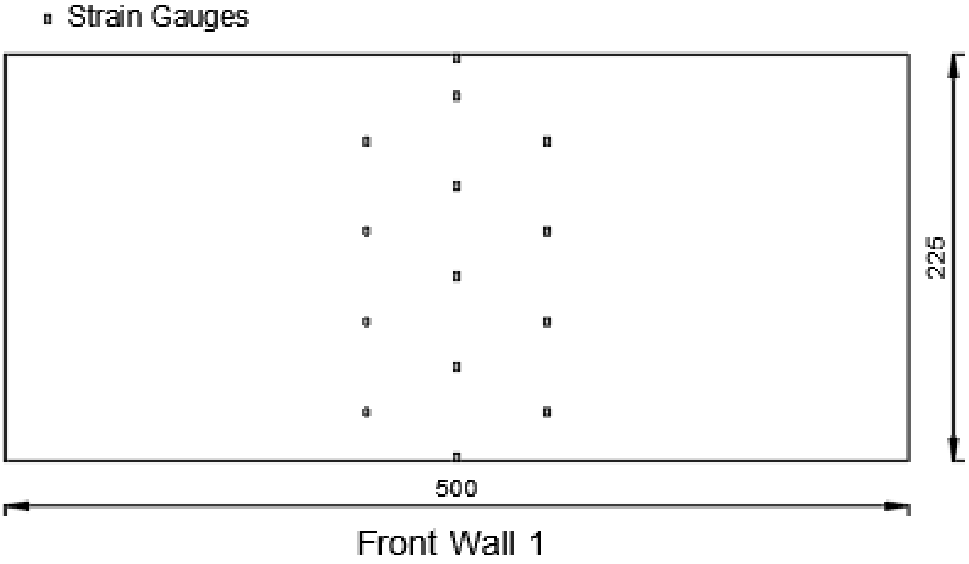
- High speed video cameras captures the motion of targets installed on the surface of the retaining wall.

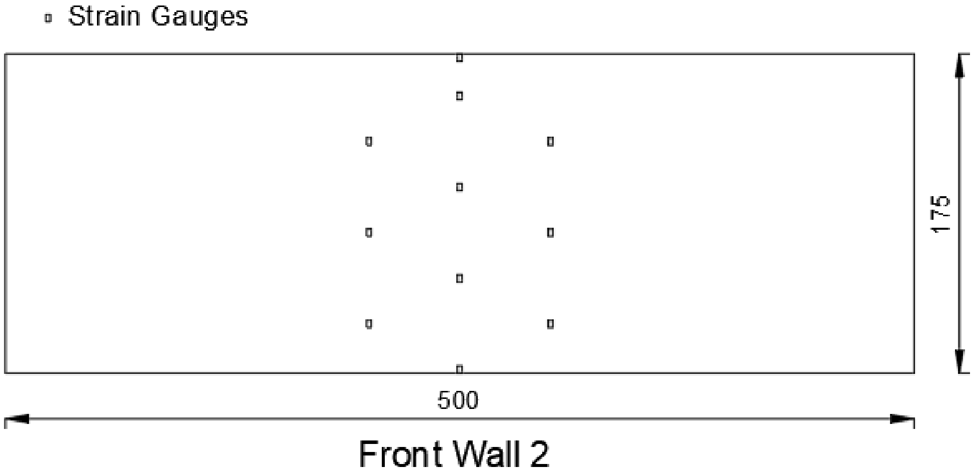


Test Instrumentation

4- Strain Gauges

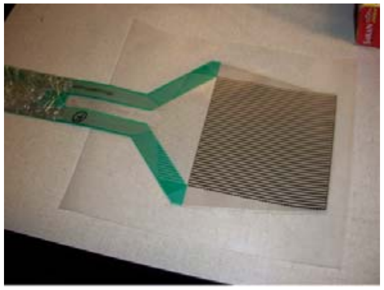






Dimensions in (mm)

5- Tactile Pressure Sensors



THANKS !