Workshop on Penetration Testing and other Geomechanical Issues

University of Pisa, June 14th, 2016

New developments on the mechanical characterization of levees using CPT_U

Barbara Cosanti

D. Lo Presti

LEVEES

also called DIKES, DIGUES or
FLOOD DEFENCE EMBANKMENTS

✓ Earth structures that, under normal conditions, are not subject to the action of waves and currents ⇒ poor attention

✓ part of flood defence systems

✓ provide protection against fluvial and coastal flood events

LEVEES & FLOOD RISK

Need for

GOOD PERFORMANCE on the OCCASIONS when they are LOADED (STORMS & FLOODS)

- ✓Good design
- ✓Good inspection
- ✓Routine maintenance

LEVEES & RESILIENCE

ABILITY TO RETAIN AND RECOVER FUNCTIONAL PERFORMANCE UNDER THE STRESS OF KNOWN AND UNKNOWN ADVERSE EVENTS (SCHULTZ et al., 2012)

Capacity to accommodate:

- ✓ the loading situations for which they are being designed without damage and breach due to potential failure mechanisms
- ✓ situations in which they are overtopped: levees should be designed so that they do not fail for an extended period of time that reflects the time required to evacuate the flood area

In the event of any breach, the levees should be readily repairable.

EXISTING LEVEES

✓do not respect modern design and construction standards

✓ irregular in the standard and nature of their construction

✓ records of their construction and historical performance usually do not exist

LEVEE DESIGN

Main causes of failure:

- Erosion mechanisms → Internal erosion → External erosion
- Slope instability

poor degree of compaction = predisposing factor

LEVEE DESIGN

SOIL PROPERTIES

- Resistance to external and internal erosion
- Permeability
- Shear strength
- Density
- Resistance to liquefaction

LEVEE DESIGN

CONTROLS →

- material type
- material grading
- dry density
- moisture content

Strength, compressibility & permeability → f (degree of compaction; degree of saturation)

Assessment of soil density & water content → Estimate of the required parameters for the design of resilient levees

LEVEES: DESIGN-CONSTRUCTION PROCEDURE

$\left.\begin{array}{l}\gamma_d \\ w\end{array}\right\}$ reflect the ENGINEERING PROPERTIES → used for CONSTRUCTION CONTROL

1 LABORATORY TESTS on samples of the proposed soil materials → Definition of the engineering properties required for design

2 FIELD COMPACTION CONTROL TESTS are specified → Results of these tests become the STANDARD FOR CONTROLLING THE PROJECT

3 CONTROL TESTS → To ensure that the construction actually adheres to the COMPACTION SPECIFICATIONS

LEVEES: FIELD DENSITY TESTS

✓**Sand cone method**
(AASHTO T 191; ASTM D 1556)

✓**Balloon method**
(AASHTO T 205; ASTM D 2167).

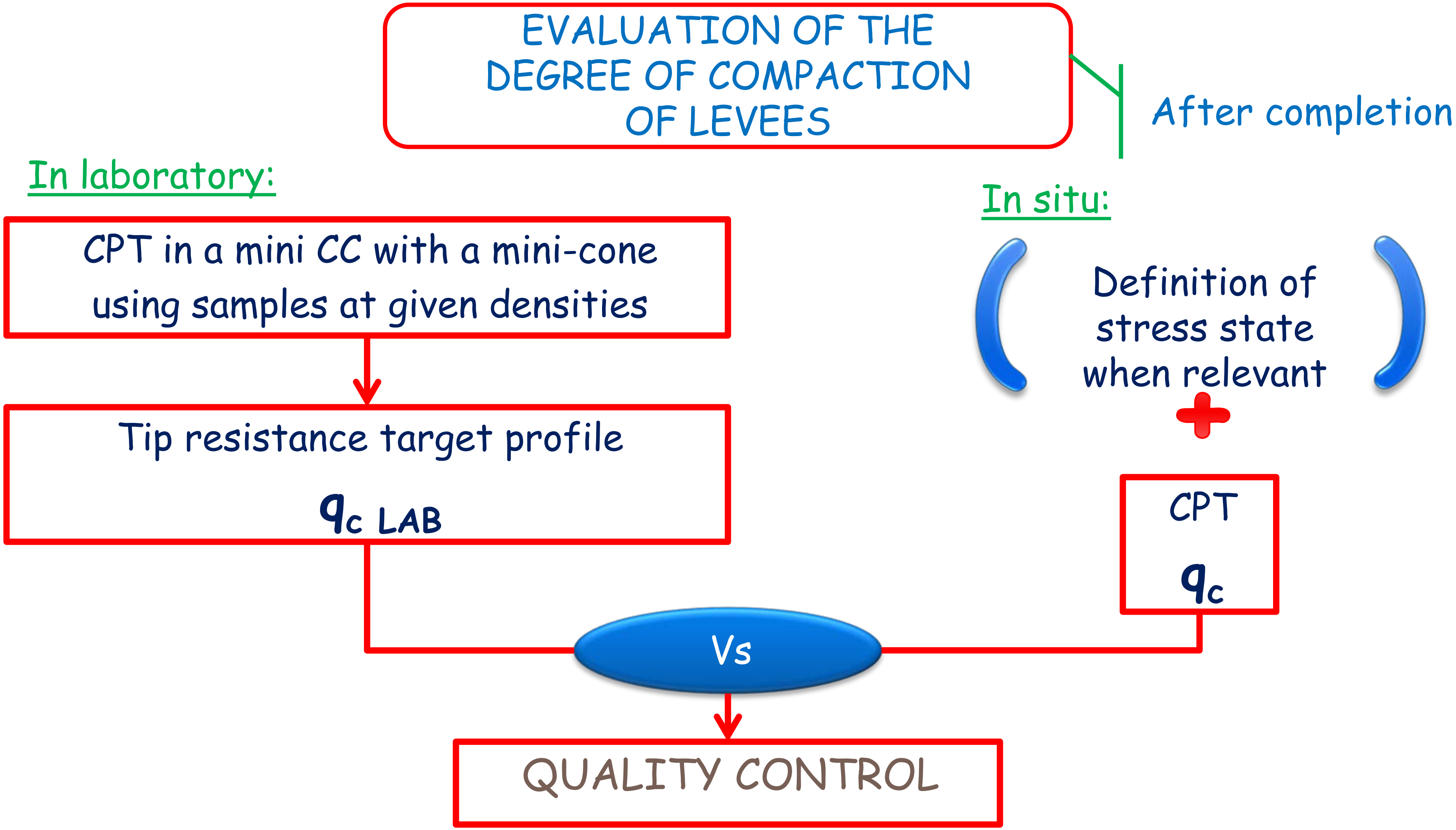
Common destructive field density tests

✓ **Nuclear methods**
(AASHTO T 238 and T 239; ASTM D 2922 and D 3017)

✓ **TDR**
(ASTM D 6780)

Nondestructive tests

THE PROPOSED METHOD



EVALUATION OF THE DEGREE OF COMPACTION OF LEVEES

After completion

In laboratory:

CPT in a mini CC with a mini-cone using samples at given densities

Tip resistance target profile

$q_{c\ LAB}$

In situ:

(Definition of stress state when relevant)

+

CPT

q_c

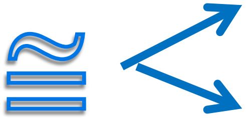
Vs

QUALITY CONTROL

- ✓ EXISTING LEVEES → DEGREE OF COMPACTION
- ✓ NEW LEVEES → EXPECTED q_c CORRESPONDING TO A PRESCRIBED DENSITY

HYPOTHESES

1. The tip resistance is not affected by the tip diameter



$\cong$

q_c from CPT (d= 35.7 mm)

q_{cLAB} from mini-CPT (d= 8 mm)

Even if:

$$V_{mini\text{-}CPT} = 1/4\ V_{CPT}$$

$$V = \frac{v \cdot d}{c_v}$$

Normalized velocity

(Finnie & Randolph, 1994)

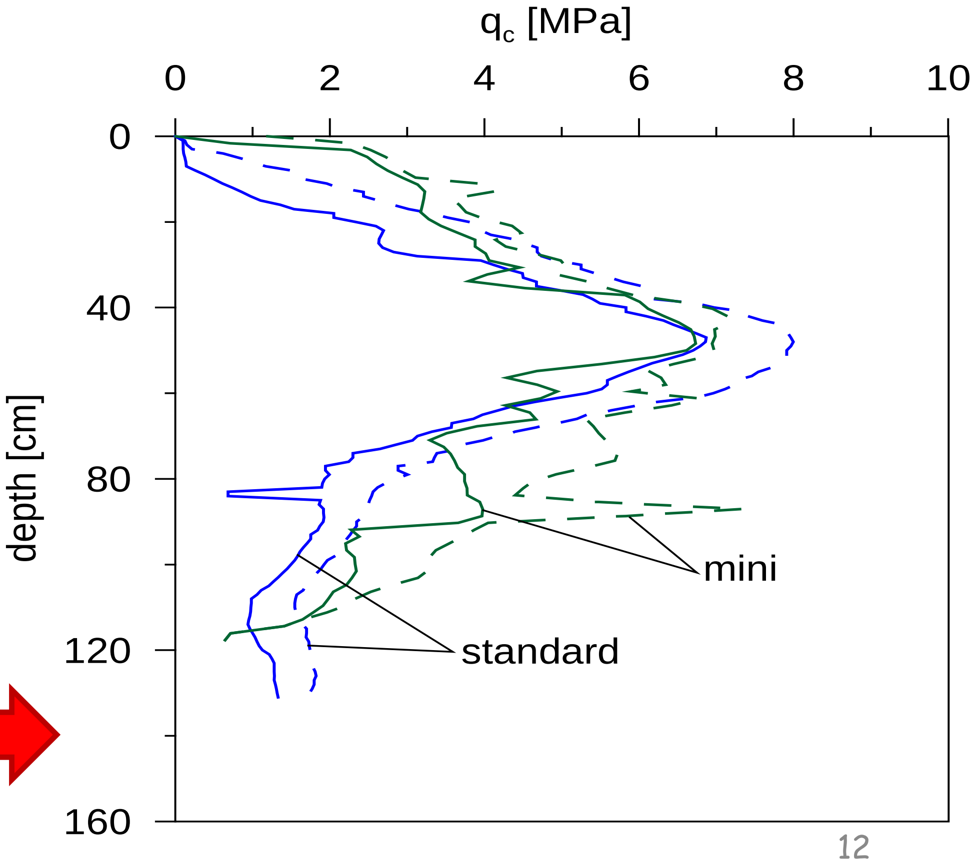
Where:

v: velocity of the penetrometer

d: diameter of the penetrometer

c_v: coefficient of consolidation of the soil

UNSATURATED SILT MIXTURES



HYPOTHESES

2. DRY SAND →

$$\boxed{q_c = C_0 \cdot \sigma'^{\,C1}_{v0} \cdot \sigma'^{\,C2}_{h0} \cdot e^{C_3 \cdot D_R}}$$

(Baldi et al. 1986, Jamiolkowski et al. 1988, Garizio 1997, Jamiolkowski et al. 2000, 2001)

IN SITU STRESS STATE

→

- Estimate of the vertical effective stress component σ'_v
- At-rest earth pressure coefficient → DMT test

$$\boxed{K_0 = 0.376 + 0.095 \cdot K_D + 0.0046 \frac{q_c}{\sigma'_{v0}}}$$

(Jamiolkowski et al. 1988)

HYPOTHESES

2. PARTIALLY SATURATED SILT MIXTURES

EVALUATION OF THE STRESS STATE (in situ & laboratory) → **soil suction**

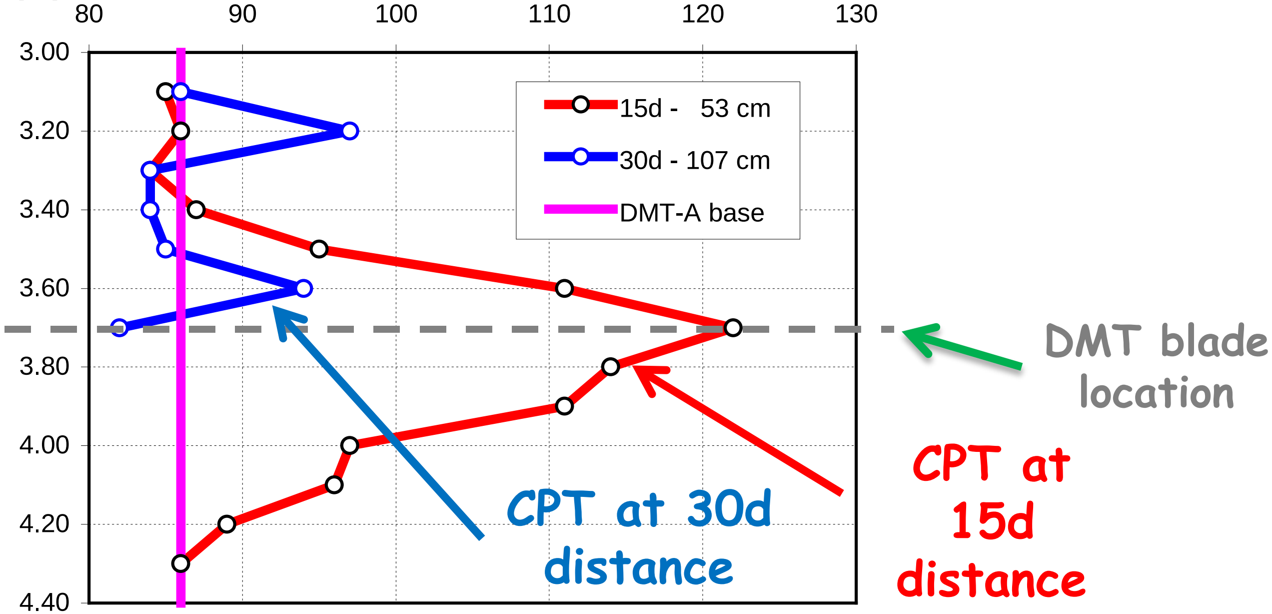
Relative density is not the relevant index for the compacted state of soil including a large amount of fines content (Tatsuoka, 2011).

The degree of compaction defined for a certain compaction energy is more appropriate.

HYPOTHESES

3. The effects of the mini-chamber sizes can be considered negligible if:

$$\frac{\textbf{CC diameter}}{\textbf{Mini-cone diameter}} = 40$$



When the horizontal distance between DMT and CPT is ≥ 20 times the cone diameter the DMT is no longer sensitive to the passage of the cone.

4. It is acceptable:

$$\frac{\textbf{Mini-cone diameter}}{\textbf{Mean grain size}} \geq 300$$

(Baldi and O'Neill, 1995; Parkin, 1988; Salgado, 2013; Schmertmann, 1978)

THE EQUIPMENT

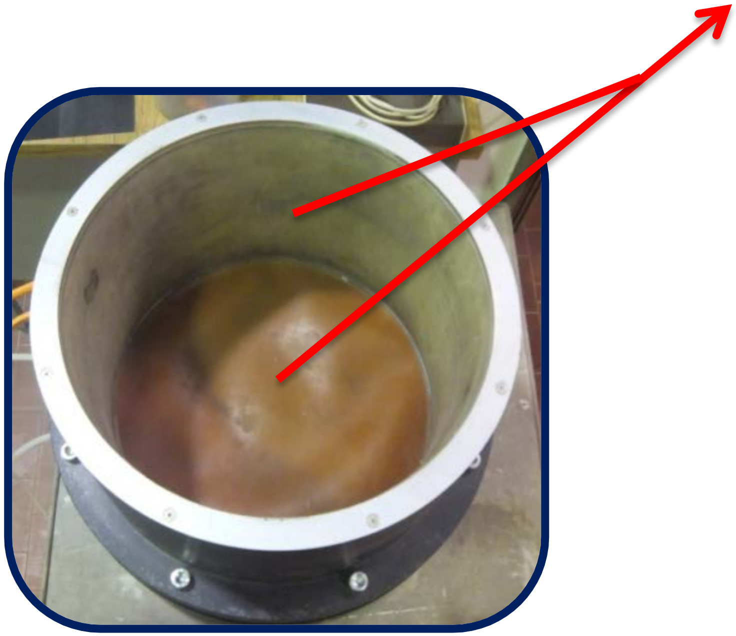
Developed by the Geotechnical Laboratory of the University of Pisa in partnership with Pagani Geotechnical Equipment

THE EQUIPMENT

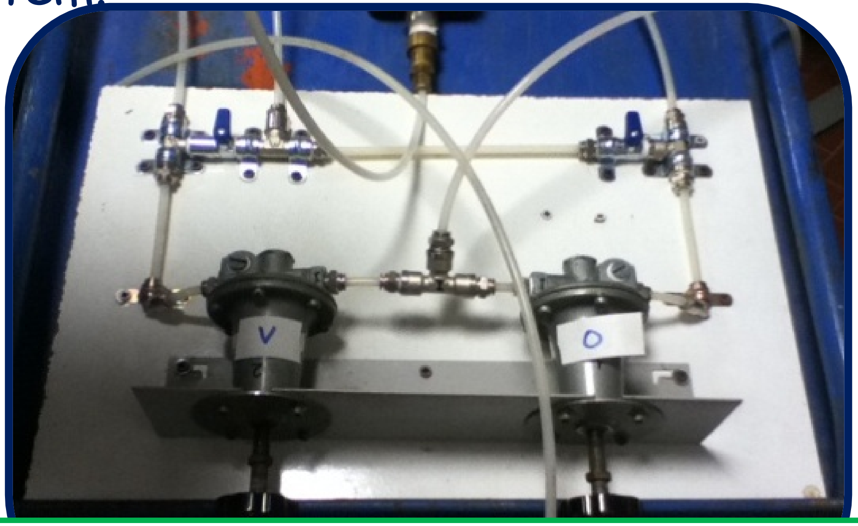
Diameter = 320 mm; Height = 210 mm

Top boundary → rigid
Lateral and bottom boundaries → flexible →
provided with latex membranes

THE EQUIPMENT



The membranes allow the independent application of horizontal and vertical stresses through a compressed air system.

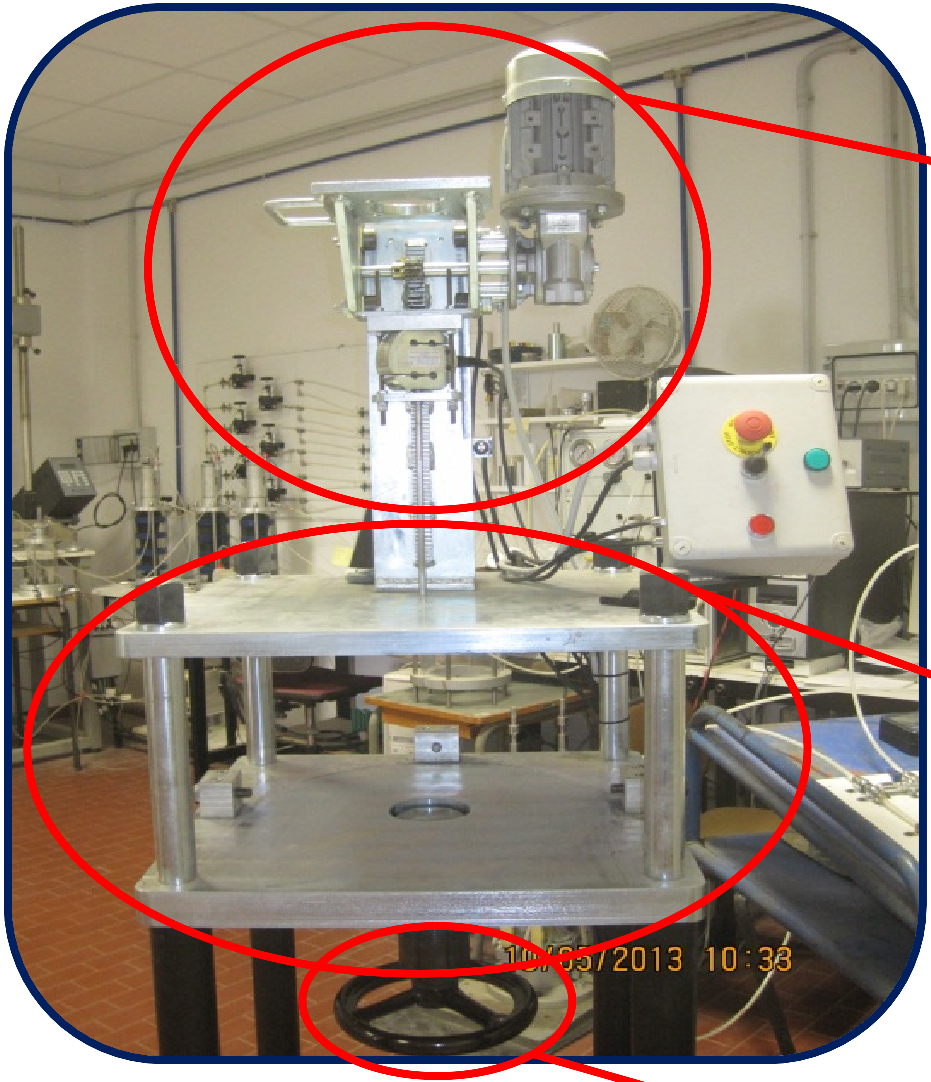


Manual air pressure regulators for the vertical and horizontal stresses.

All the possible chamber boundary conditions can be applied:

✓BC1 = σ_h = cost; σ_v = cost
✓BC2 = ε_h = 0; ε_v = 0
✓BC3 = ε_h = 0; σ_v = cost
✓BC4 = σ_h = cost; ε_v = 0

THE EQUIPMENT



Electric step motor

Stainless steel frame

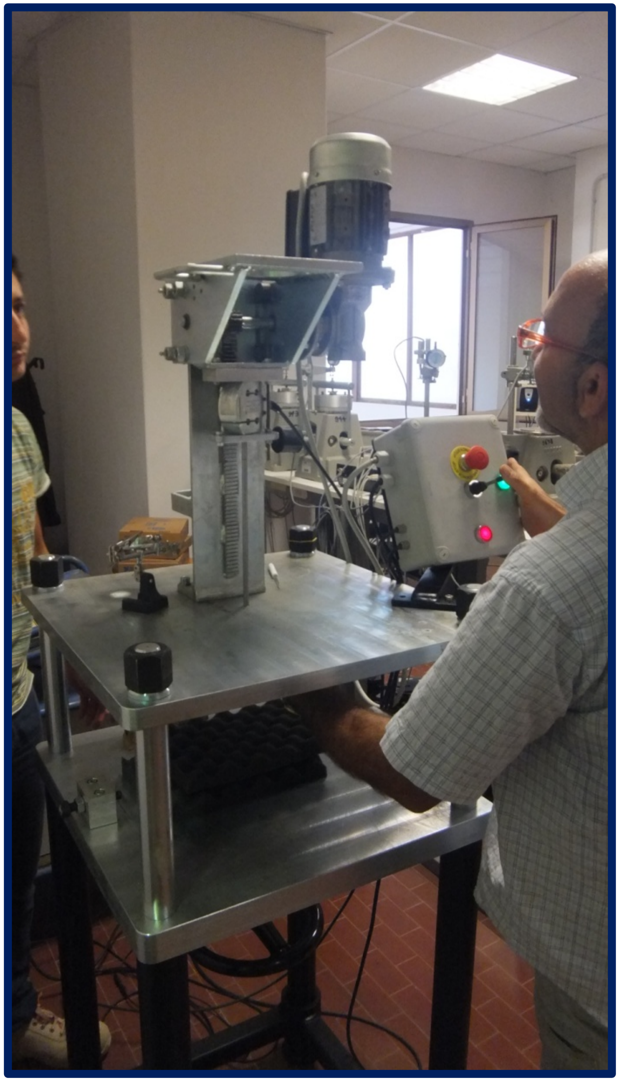
Locking system

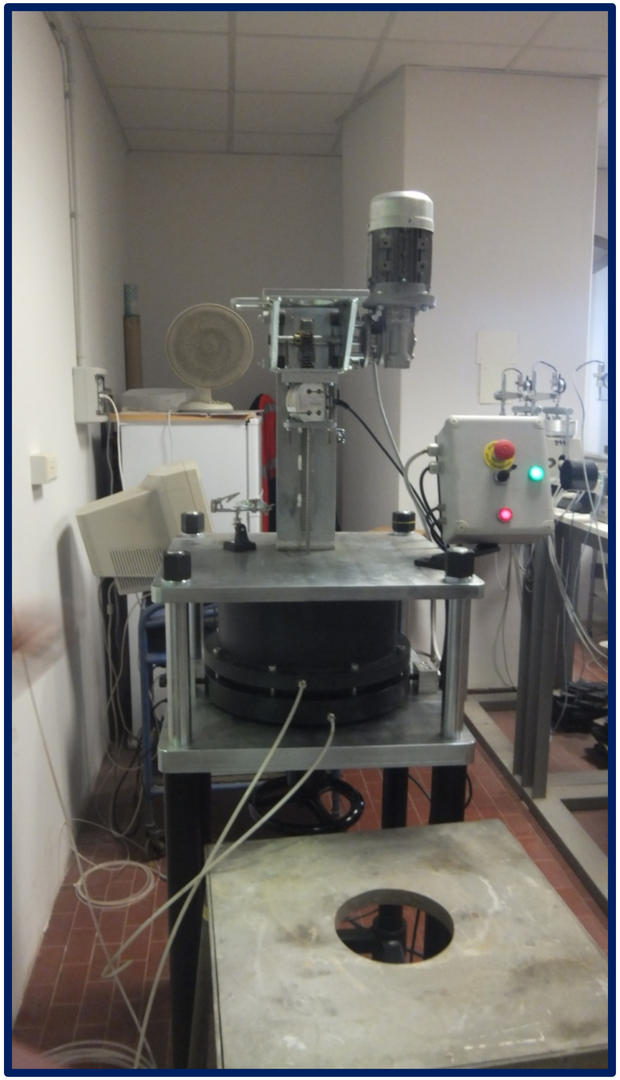
THE EQUIPMENT

Mini CPT:

- ✓ 60° conical tip
- ✓ cone diameter = 8 mm
- ✓ external sleeve
- ✓ standard rate of 20 mm/s
- ✓ load cell external to the cone

THE EQUIPMENT





TESTED MATERIALS

TICINO SAND

dry

Preliminary tests

↓

for validating the equipment
(mini CC and mini-cone).

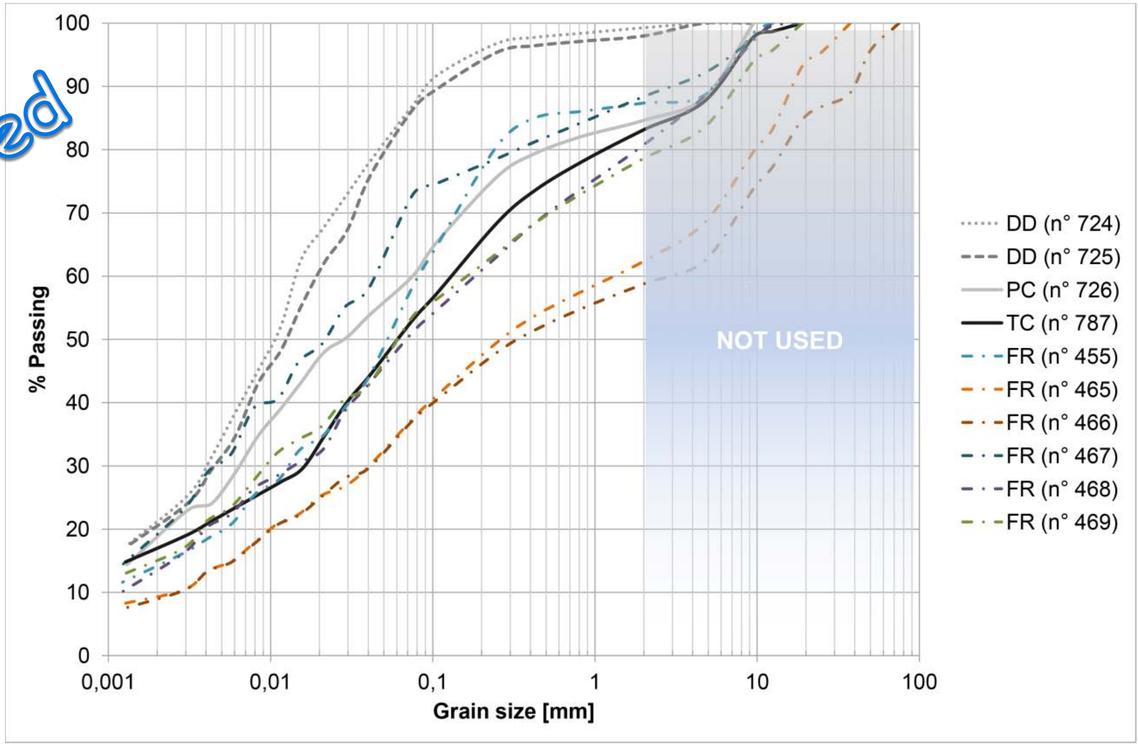
Dry sand samples are reconstituted inside the CC to a given D_R by dry pluviation.

TESTED MATERIALS

FINE-GRAINED SOILS
(Building materials)

partially saturated

Only the soil passing the 2 mm sieve is used



Soil type	Modified Proctor (ASTM D1557)				Atterberg Limits (ASTM D 4318)			Soil classification		
Abbreviation	γ_{dmax} [kg/m^3]	W_{opt} [%]	e_{opt}	$(Sr)_{opt}$ [%]	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	AASHTO M 145 (1991)	Gs	d_{50} [mm]
FR	2047	9.43	0.33	78	26÷31	18÷24	7÷10	A4÷A6	2.72	0.002÷0.025
PC	1950	10.7	0.39	74	25	19	6	A4	2.71	0.085
DD	1820	13.1	0.49	73	31.5	23.5	8	A4	2.71	0.01
TC	1895	12	0.42	77	25	6	19	A6	2.69	0.02

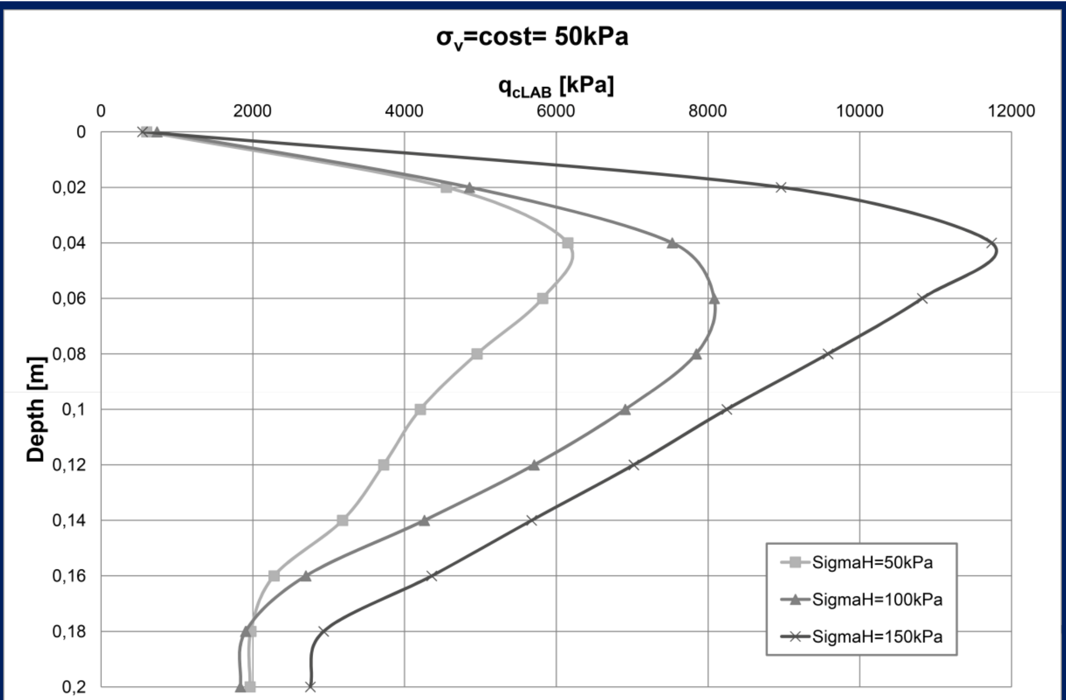
TESTED MATERIALS

partially saturated

FINE-GRAINED SOILS

- ✓ Samples are reconstituted in 5 layers in a mold
- ✓ The soil is prepared at a given w and compacted to a given γ_d using static compaction
- ✓ The compaction effort, required to consolidate each layer and the sample, is recorded: $\boxed{E = (1/2 \cdot \sum_{i=1}^{5} F_i \cdot \delta_i)/\sum_{i=1}^{5} V_i}$

TEST RESULTS

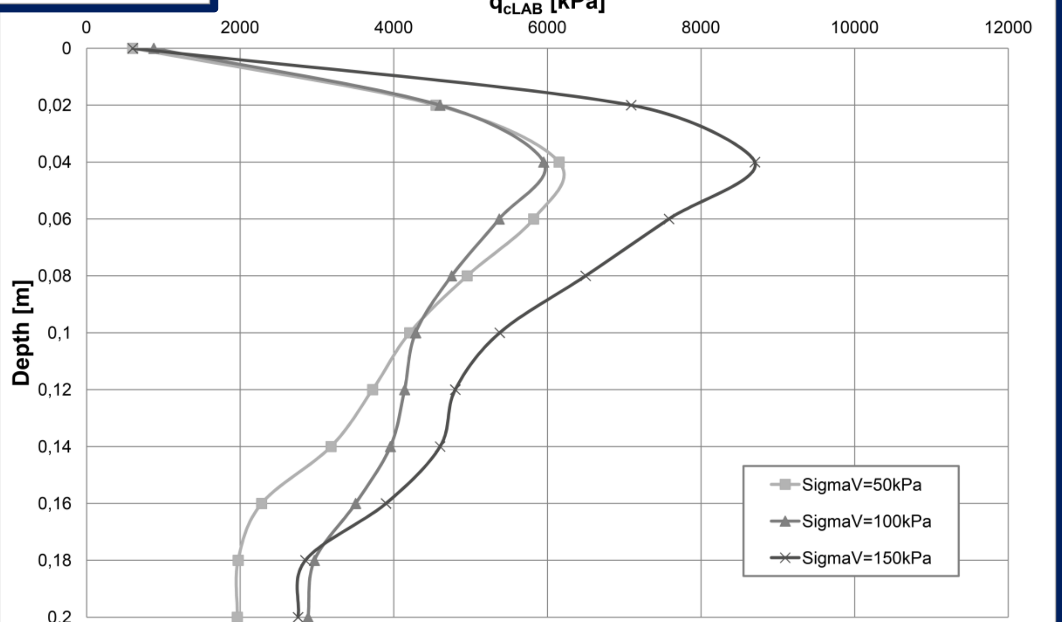


TICINO SAND

dry sand

TS4

Dr = 60%



TEST RESULTS

σ'h = constant = 50 kPa

σ'v [kPa]	σ'h [kPa]	Qc measured [kPa]	C1	Qc correlation [kPa]
50	50	4377	0.229	4225
100	50	4501		4952
150	50	5772		5434

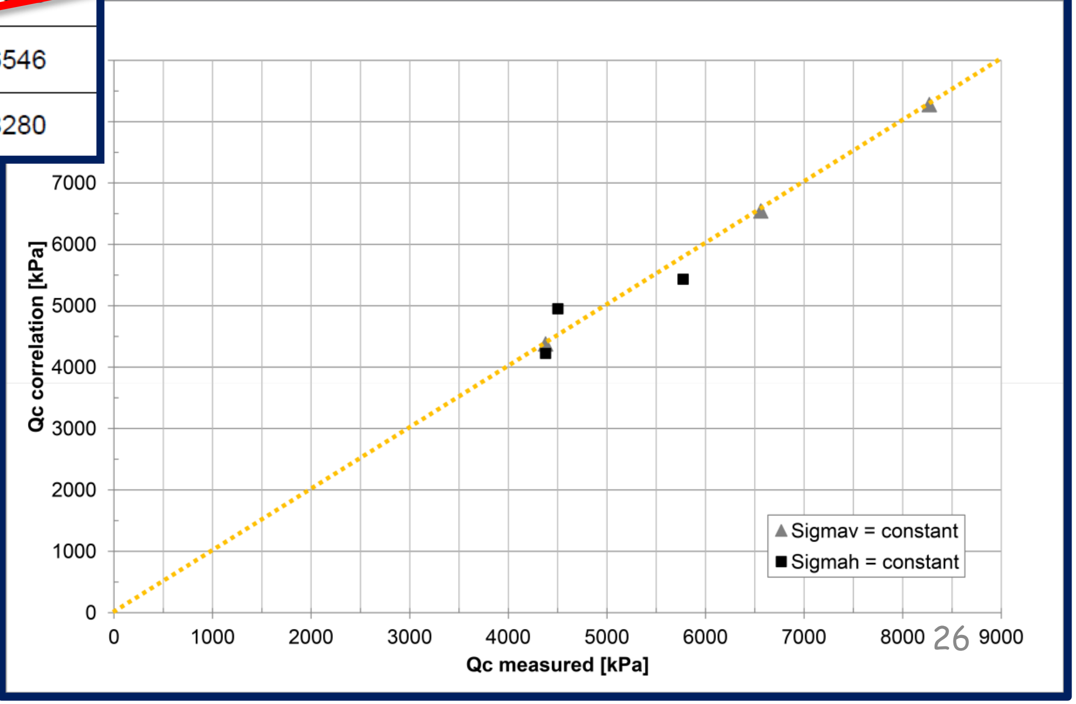
σ'v = constant = 50 kPa

σ'v [kPa]	σ'h [kPa]	Qc measured [kPa]	C2	Qc correlation [kPa]
50	50	4377	0.580	4380
50	100	6560		6546
50	150	8269		8280

TICINO SAND

dry sand

$$q_c = C_0 \cdot \sigma_{v0}'^{\,C1} \cdot \sigma_{h0}'^{\,C2} \cdot e^{C_3 \cdot D_R}$$



C2 > C1

Results in agreement with already published results (Jamiolkowski et al. 2001; Arroyo et al. 2011)

EXPERIMENTAL PROGRAM

Soil type	Boundary stresses		Unit weight			Water content				
Abbreviation	σ'_v [kPa]	σ'_h [kPa]	γ_d [kN/m^3]	γ_{dmax} [kN/m^3]	γ_d/γ_{dmax}	W [%]	W$_{opt}$ [%]	E [MJ/m^3]	σ'_{pmax} [kPa]	Qc [MPa]
DD	30	30	14.56	17.85	0.82	13.2	13.1	0.395	8224	2.807
DD	50	50	14.56	17.85	0.82	13.2		0.238	6157	1.786
DD	80	80	14.56	17.85	0.82	13.2		0.299	6752	1.512
DD	30	30	16.38	17.85	0.92	13.2		1.324	24474	4.751
DD	50	50	16.38	17.85	0.92	13.2		1.413	24523	4.063
DD	80	80	16.38	17.85	0.92	13.2		1.501	24523	4.990
PC	30	30	15.60	19.13	0.82	10.8	10.7	0.62	13731	3.274
PC	50	50	15.60	19.13	0.82	10.8		0.697	14712	3.648
PC	80	80	15.60	19.13	0.82	10.8		0.545	13731	3.850
PC	30	30	17.55	19.13	0.92	10.8		2.407	39627	7.191
PC	50	50	17.55	19.13	0.92	10.8		2.76	40707	7.877
PC	80	80	17.55	19.13	0.92	10.8		2.211	36979	7.603
FR	30	30	18.50	2.05	0.92	12.0	9.43	4.123	46864	6.533
FR	30	30	18.50	2.05	0.92	12.0		3.315	43136	6.535
FR	30	30	18.50	2.05	0.92	12.0		2.938	37465	6.767
FR	30	30	18.00	2.05	0.90	12.0		1.735	22730	3.254
FR	30	30	18.00	2.05	0.90	12.0		1.735	24005	3.568
FR	30	30	18.00	2.05	0.90	12.0		1.828	24400	4.056
FR	30	30	16.00	2.05	0.80	12.0		0.511	8608	1.843
FR	30	30	16.00	2.05	0.80	12.0		0.463	8313	1.736
FR	30	30	16.00	2.05	0.80	12.0		0.475	7823	2.022
FR	30	30	16.00	2.05	0.80	4.0		0.26	10103	2.036
FR	30	30	16.00	2.05	0.80	4.0		0.307	9809	1.479
FR	30	30	16.00	2.05	0.80	4.0		0.346	10790	1.827
FR	30	30	16.00	2.05	0.80	8.0		0.579	15990	3.077
FR	30	30	16.00	2.05	0.80	8.0		0.622	15891	2.533
FR	30	30	16.00	2.05	0.80	8.0		0.564	15303	2.455

FINE-GRAINED SOILS

partially saturated

DD; PC:

γ_d = 80÷92%γ_{dmax} (Modified Proctor)

w = w$_{opt}$

FR:

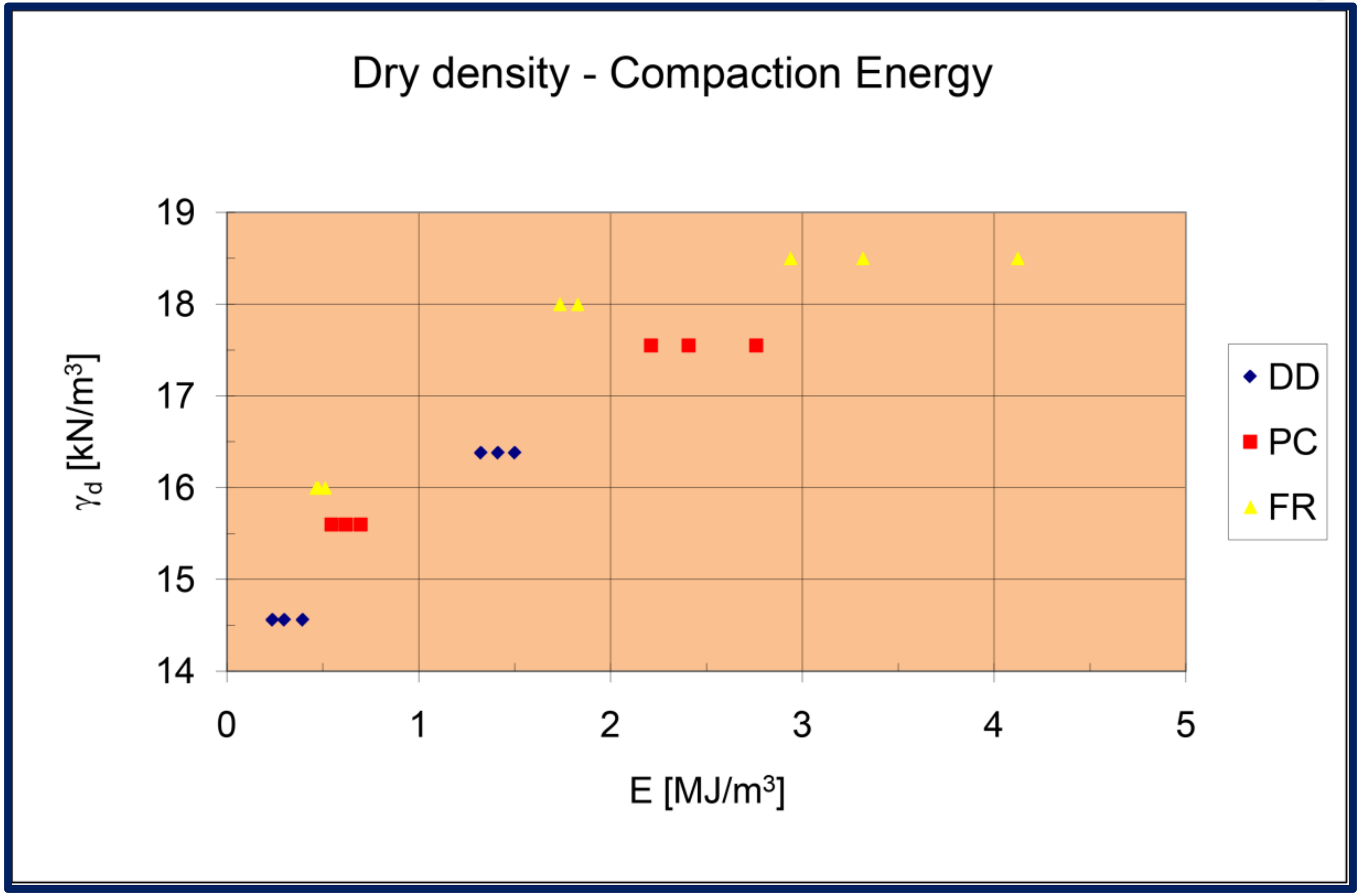
γ_d = 80%γ_{dmax} (Modified Proctor)

w= 4; 8; 12%

TEST RESULTS

FINE-GRAINED SOILS

partially saturated

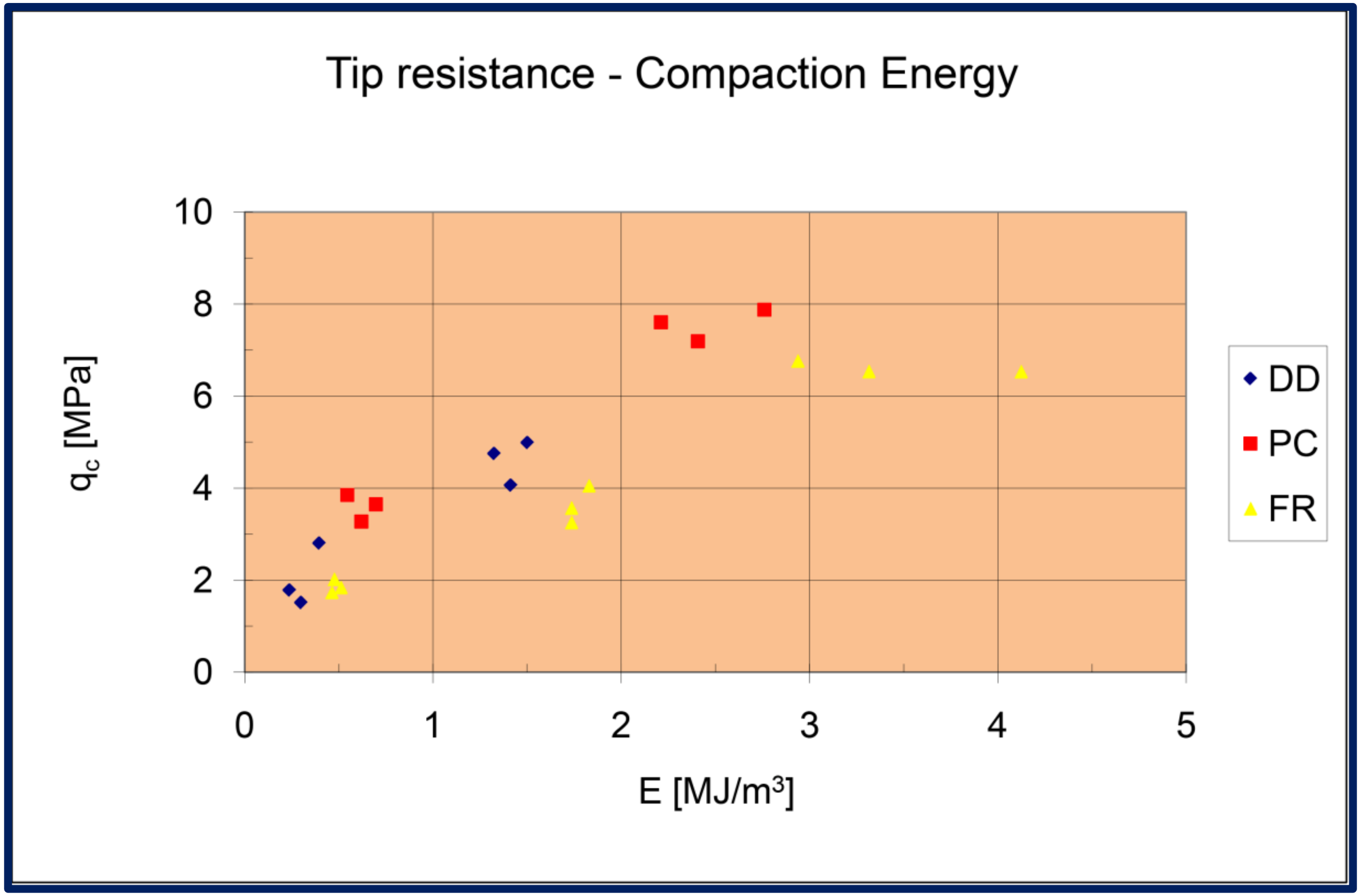


Barbara Cosanti, PhD

TEST RESULTS

FINE-GRAINED SOILS

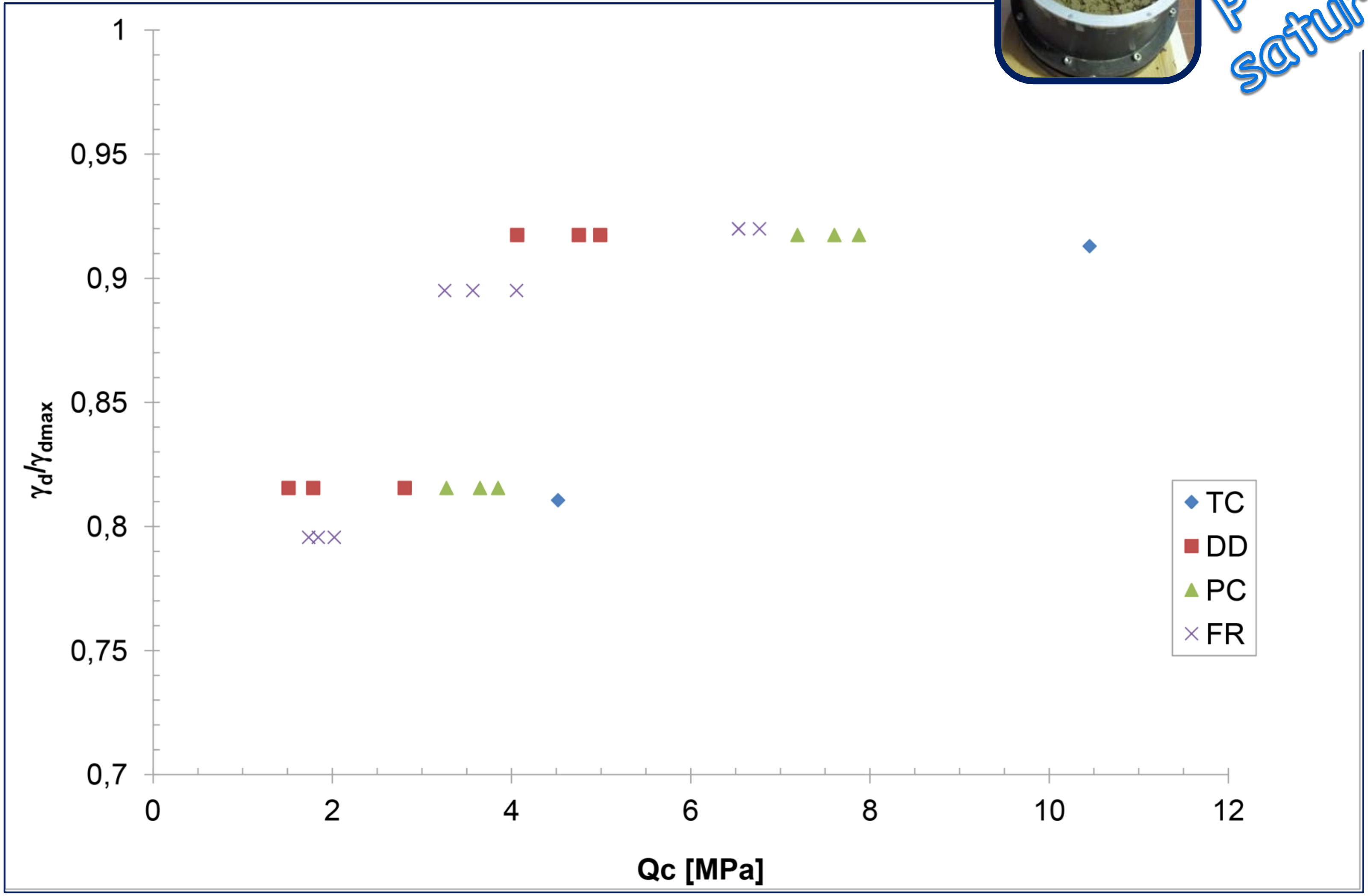
partially saturated



TEST RESULTS

FINE-GRAINED SOILS

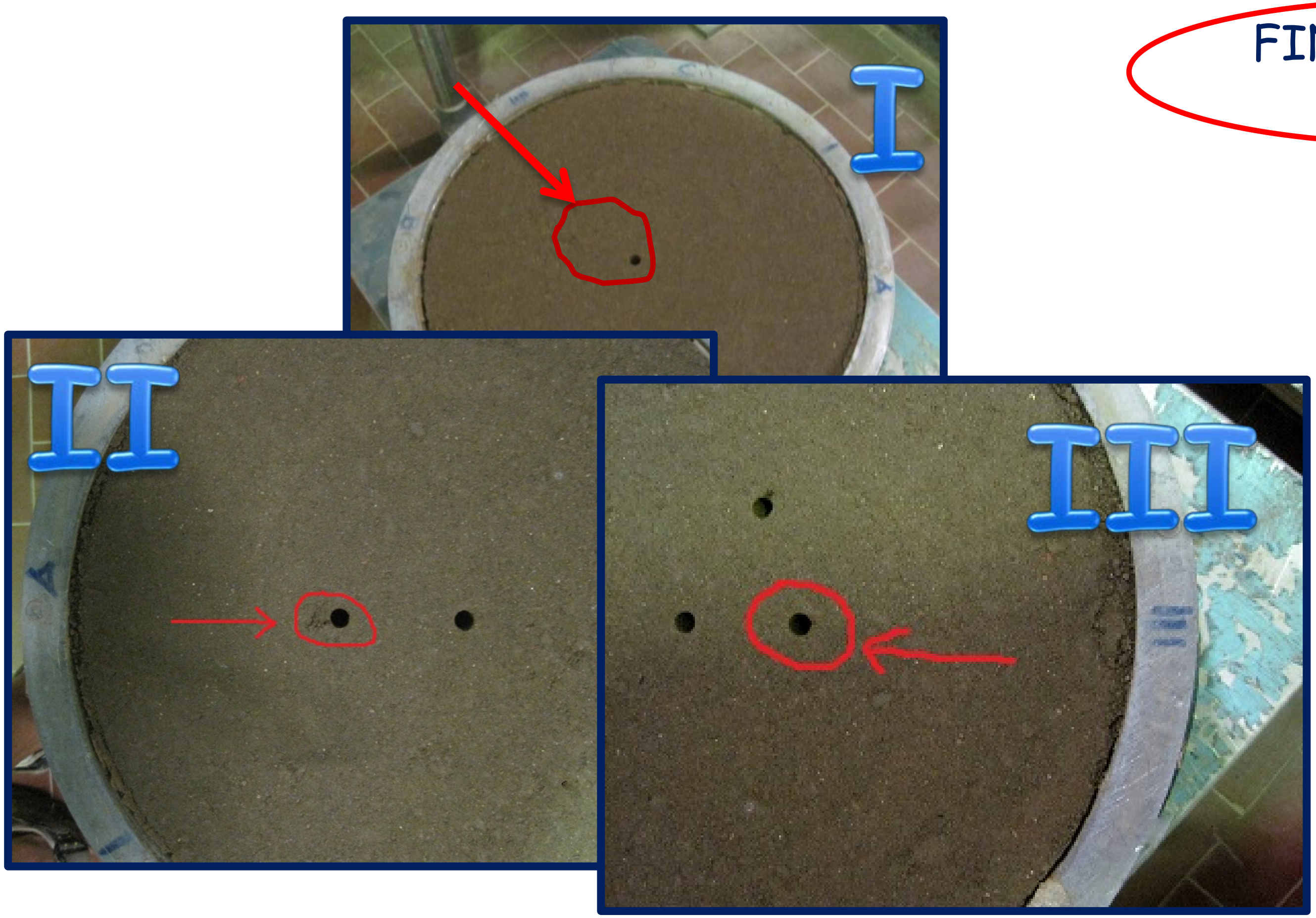
partially saturated



WATER CONTENT EFFECT

FINE-GRAINED SOILS

partially saturated



PC A4; γ_{dmax} = 1950 kg/m^3; w_{opt}=10.7%

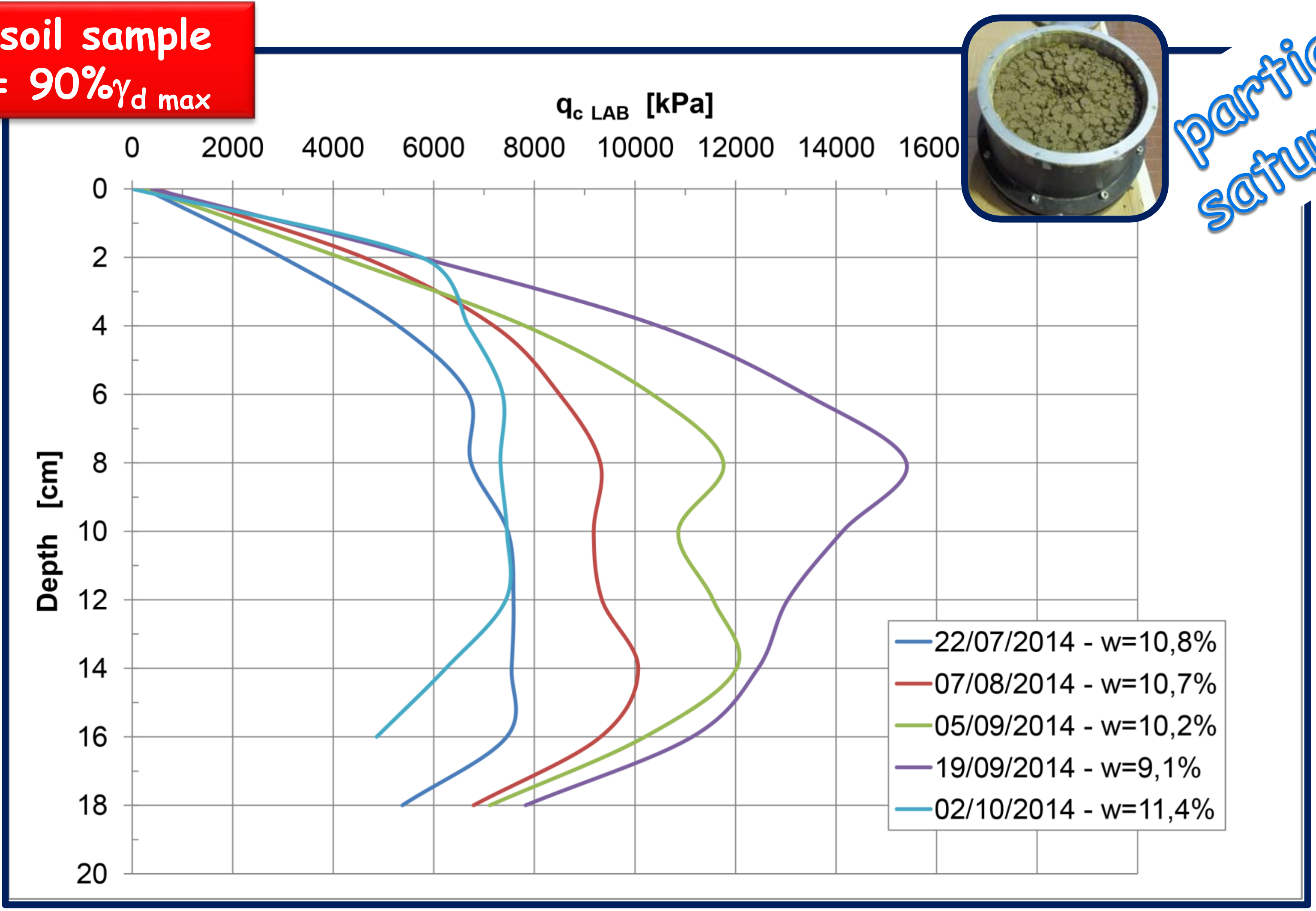
DD A4; γ_{dmax} = 1820 kg/m^3; w_{opt}=13.1%

TC A6; γ_{dmax} = 1895 kg/m^3; w_{opt}=12%

WATER CONTENT EFFECT

FINE-GRAINED SOILS

PC soil sample
$\gamma_d = 90\% \gamma_{d\ max}$



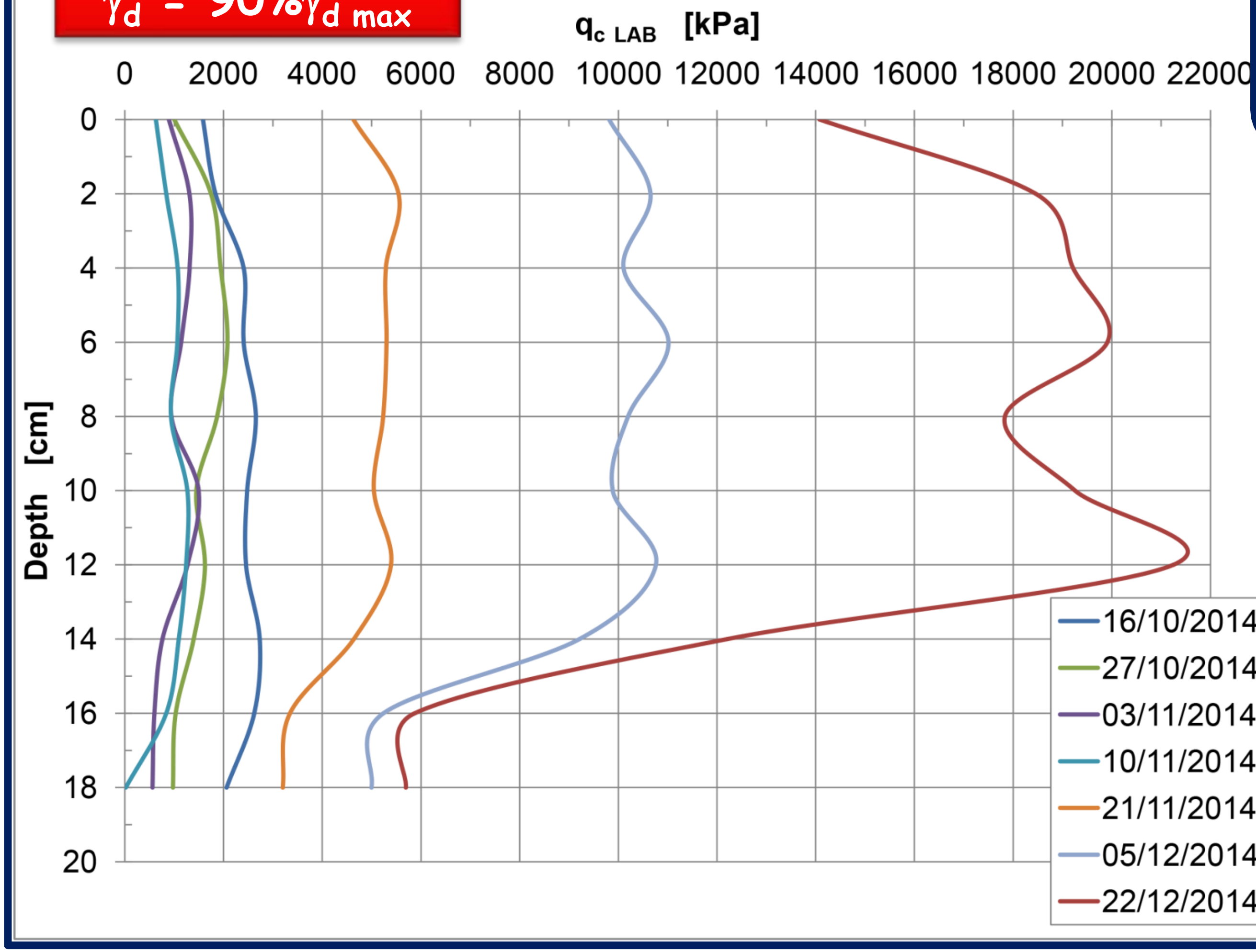
partially saturated

WATER CONTENT EFFECT

FINE-GRAINED SOILS

DD soil sample

$\gamma_d = 90\% \gamma_{d\,max}$

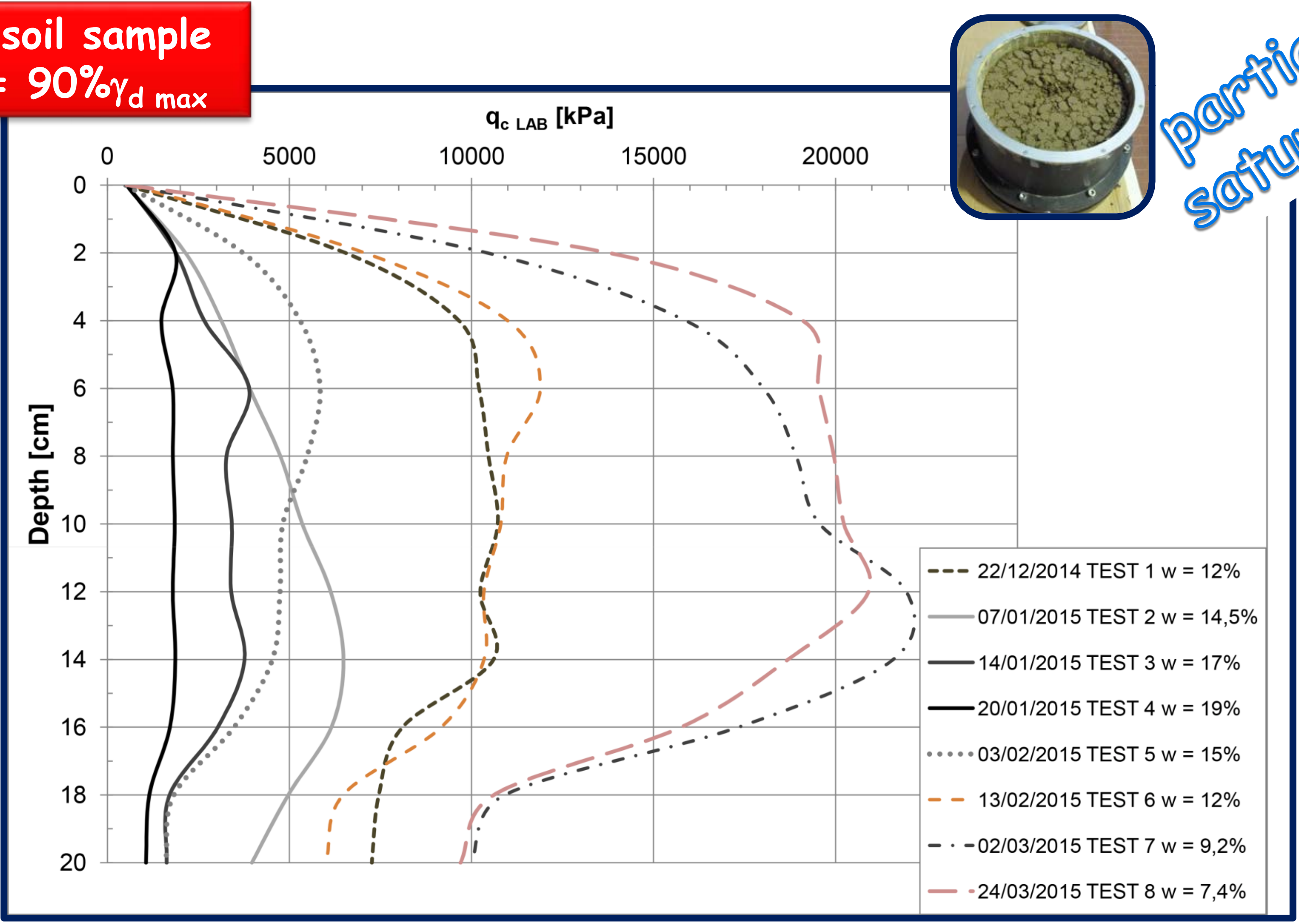


partially saturated

WATER CONTENT EFFECT

FINE-GRAINED SOILS

TC soil sample
$\gamma_d = 90\%\gamma_{d\,max}$

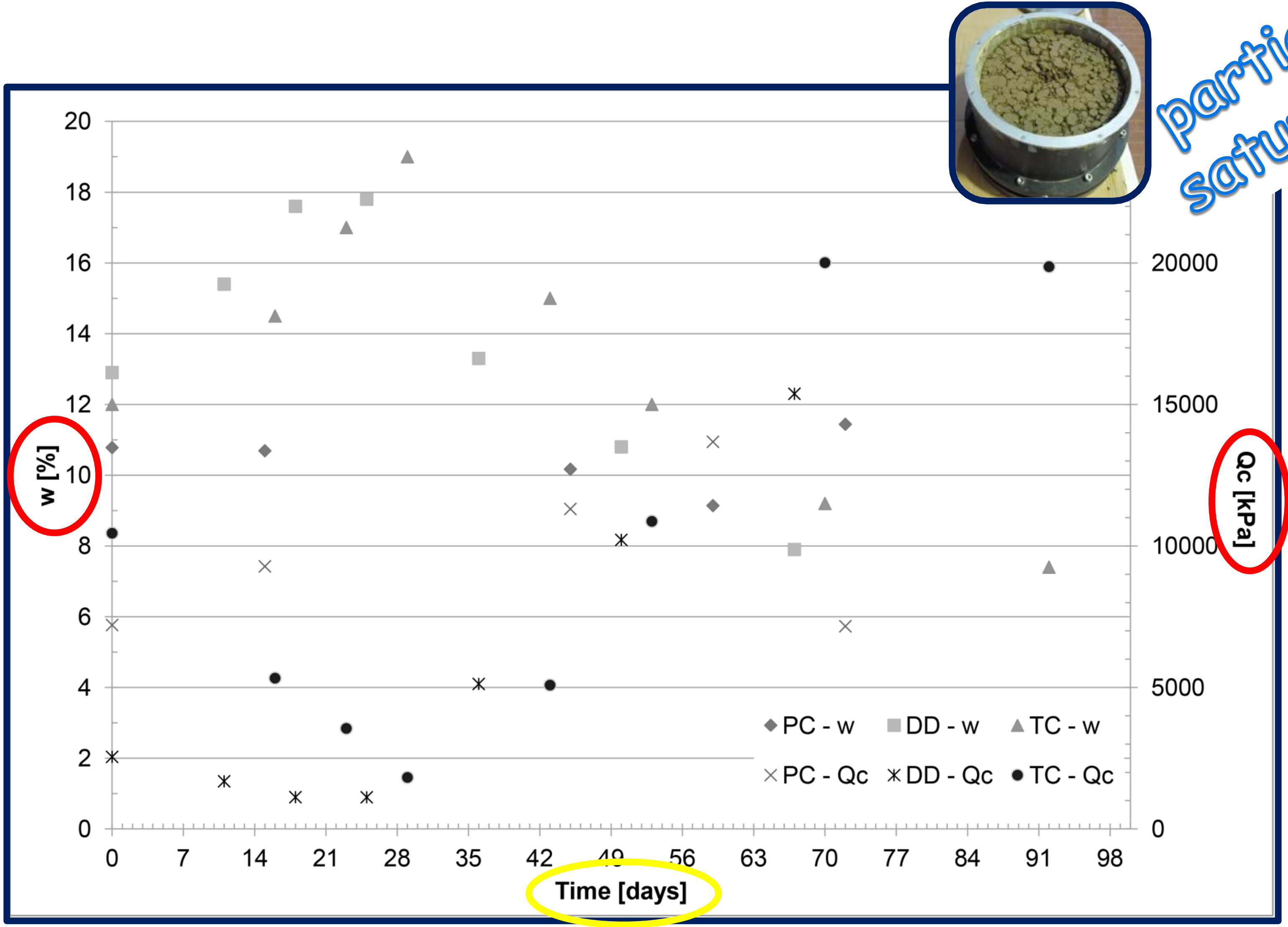


partially saturated

WATER CONTENT EFFECT

FINE-GRAINED SOILS

partially saturated

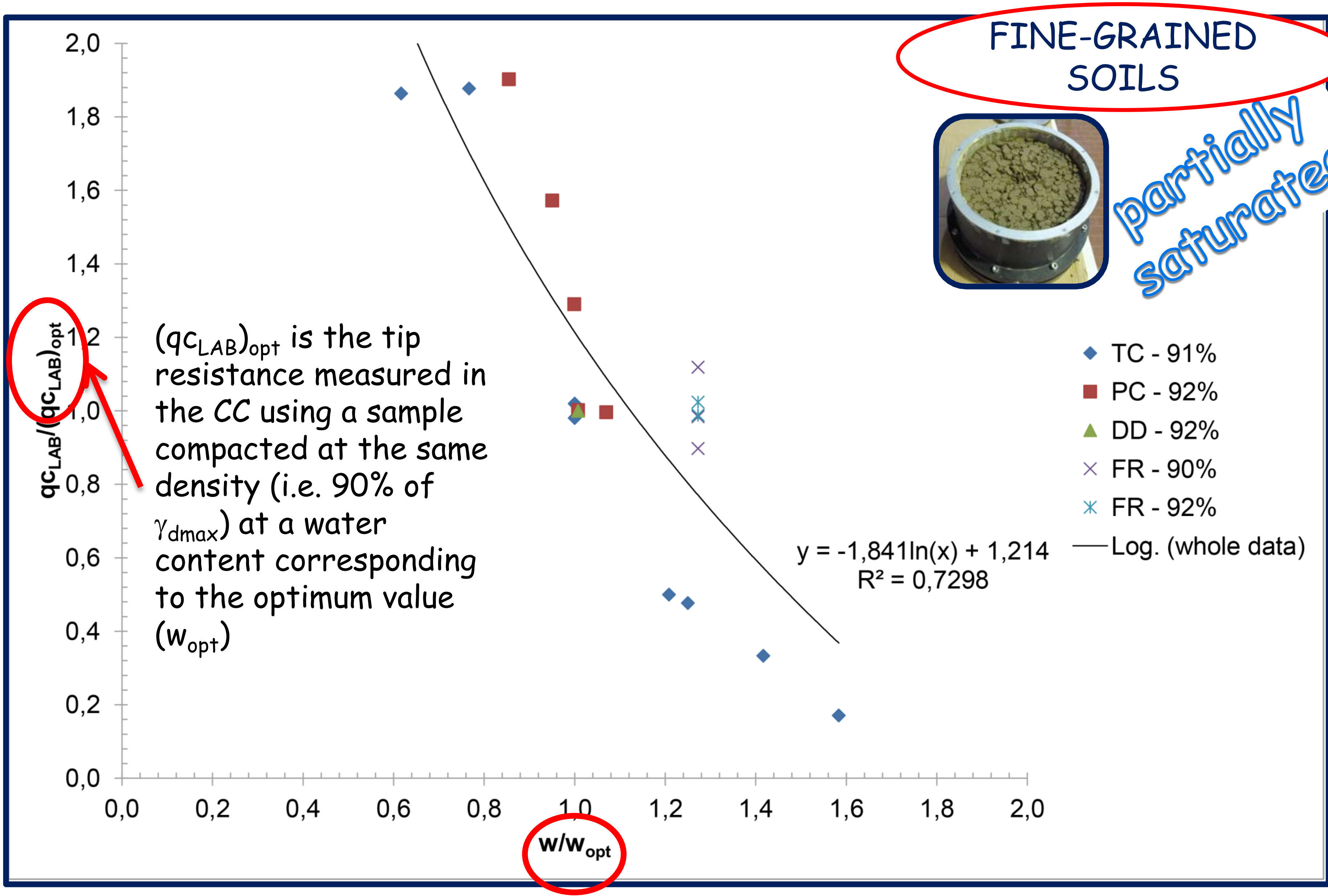


BARBARA COSANTI, PhD

WATER CONTENT EFFECT

FINE-GRAINED SOILS

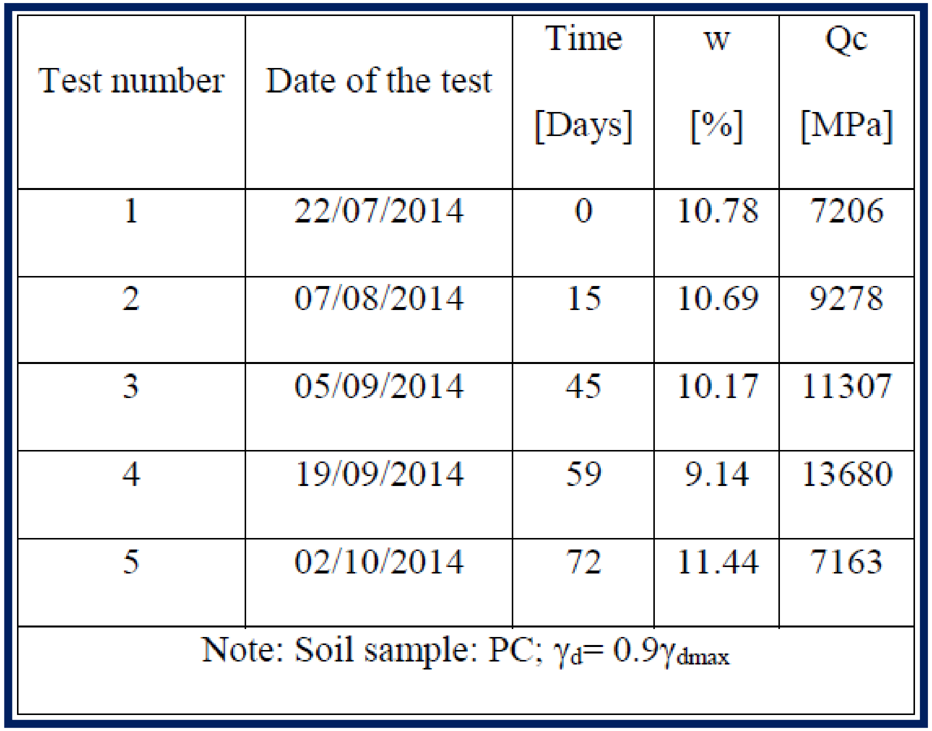
partially saturated



$(qc_{LAB})_{opt}$ is the tip resistance measured in the CC using a sample compacted at the same density (i.e. 90% of γ_{dmax}) at a water content corresponding to the optimum value (w_{opt})

WATER CONTENT EFFECT

PC w_{opt}=10.7%



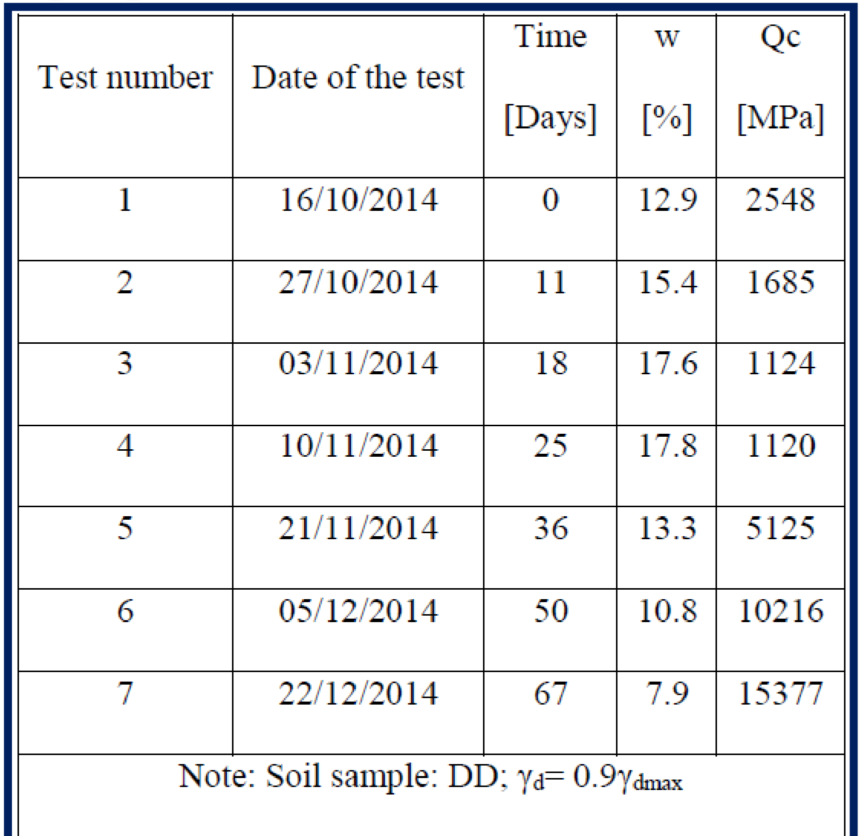
Test number	Date of the test	Time [Days]	w [%]	Qc [MPa]
1	22/07/2014	0	10.78	7206
2	07/08/2014	15	10.69	9278
3	05/09/2014	45	10.17	11307
4	19/09/2014	59	9.14	13680
5	02/10/2014	72	11.44	7163

Note: Soil sample: PC; γ_d= 0.9γ_{dmax}

FINE-GRAINED SOILS

partially saturated

DD w_{opt}=13.1



Test number	Date of the test	Time [Days]	w [%]	Qc [MPa]
1	16/10/2014	0	12.9	2548
2	27/10/2014	11	15.4	1685
3	03/11/2014	18	17.6	1124
4	10/11/2014	25	17.8	1120
5	21/11/2014	36	13.3	5125
6	05/12/2014	50	10.8	10216
7	22/12/2014	67	7.9	15377

Note: Soil sample: DD; γ_d= 0.9γ_{dmax}

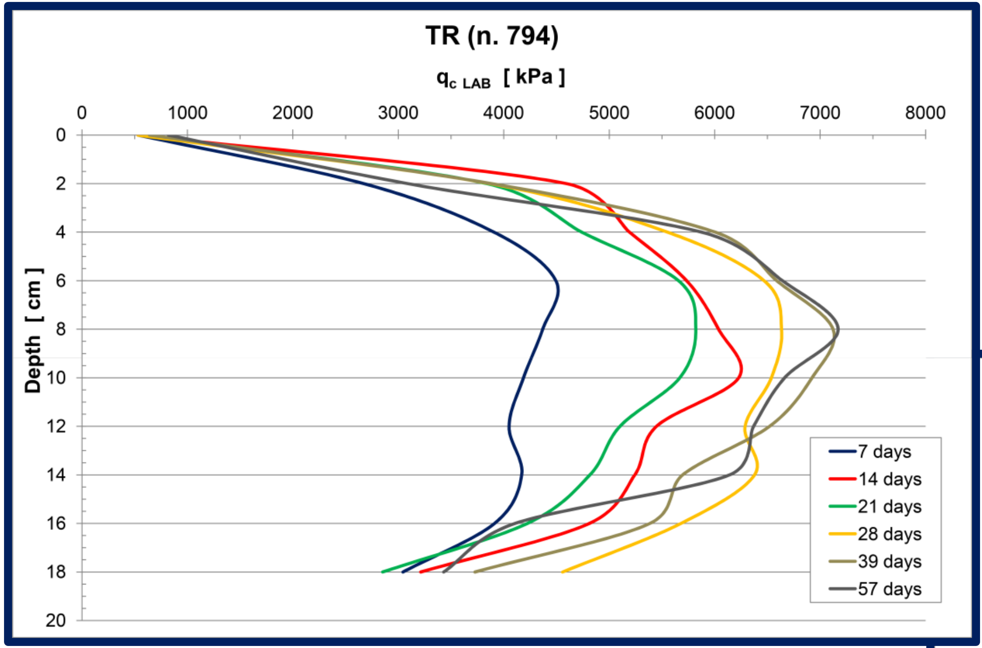
TIME EFFECT

TR A3

γ_{dmax} = 1960 kg/m^3

w_{opt}=12.1%

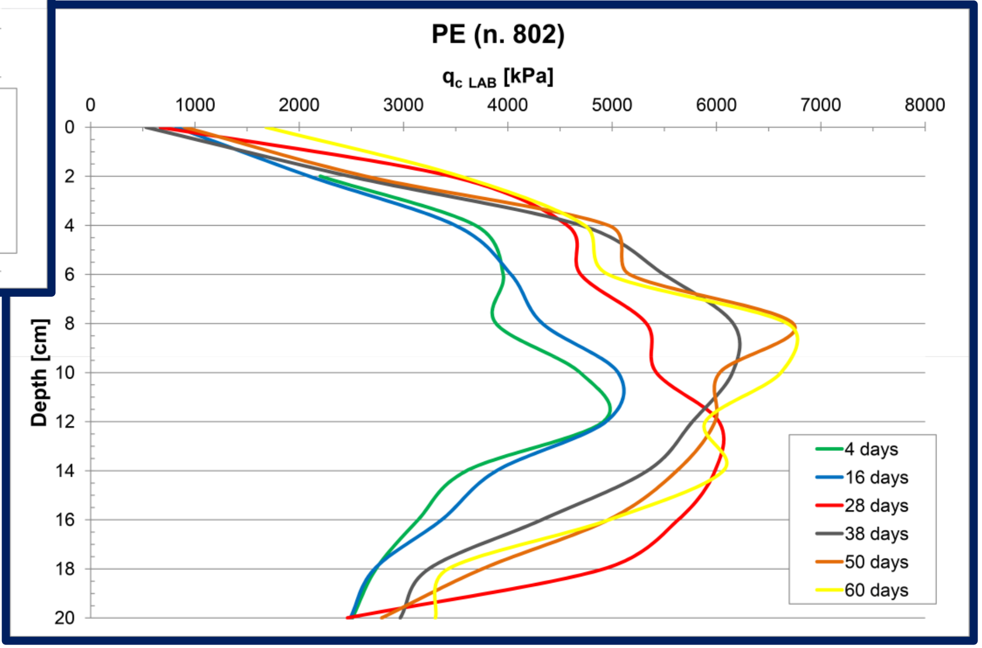
$w = w_{opt}$ = constant



PE A4

γ_{dmax} = 1860 kg/m^3

w_{opt}=10.5%



Barbara Cosanti, PhD

TIME EFFECT

FINE-GRAINED SOILS

partially saturated

TR A3

γ_{dmax} = 1960 kg/m^3

w_{opt}=12.1%

TR (n. 794)		
Test	Time [Days]	Qc [kPa]
1	7	4253
2	14	5738
3	21	5413
4	28	6461
5	39	6570
6	57	6597

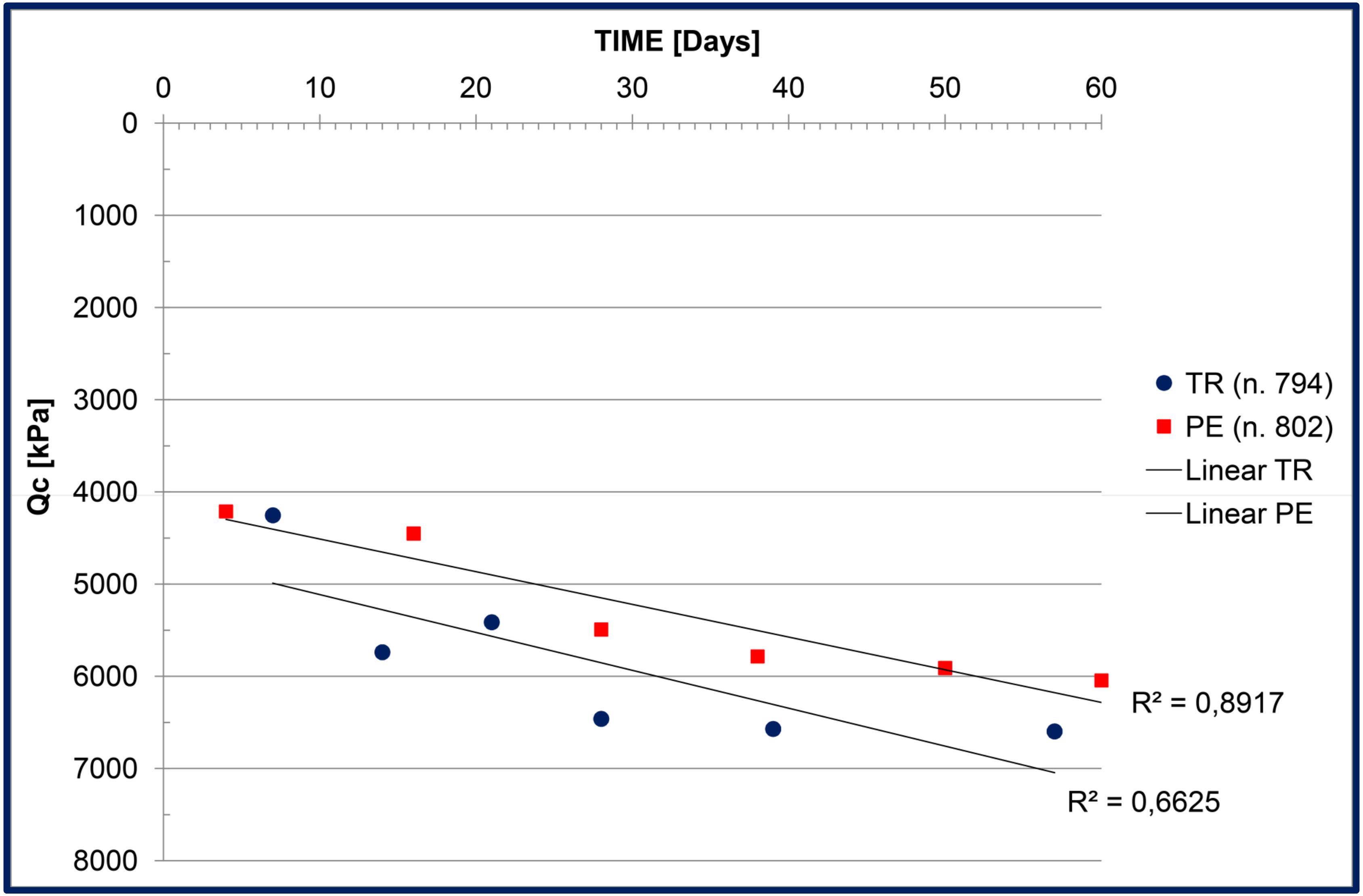
PE A4

γ_{dmax} = 1860 kg/m^3

w_{opt}=10.5%

PE (n. 802)		
Test	Time [Days]	Qc [kPa]
1	4	4211
2	16	4451
3	28	5492
4	38	5784
5	50	5908
6	60	6044

TIME EFFECT


TIME [Days]
0
10
20
30
40
50
60
Qc [kPa]
0
1000
2000
3000
4000
5000
6000
7000
8000
● TR (n. 794)
■ PE (n. 802)
Linear TR
Linear PE
R² = 0,8917
R² = 0,6625

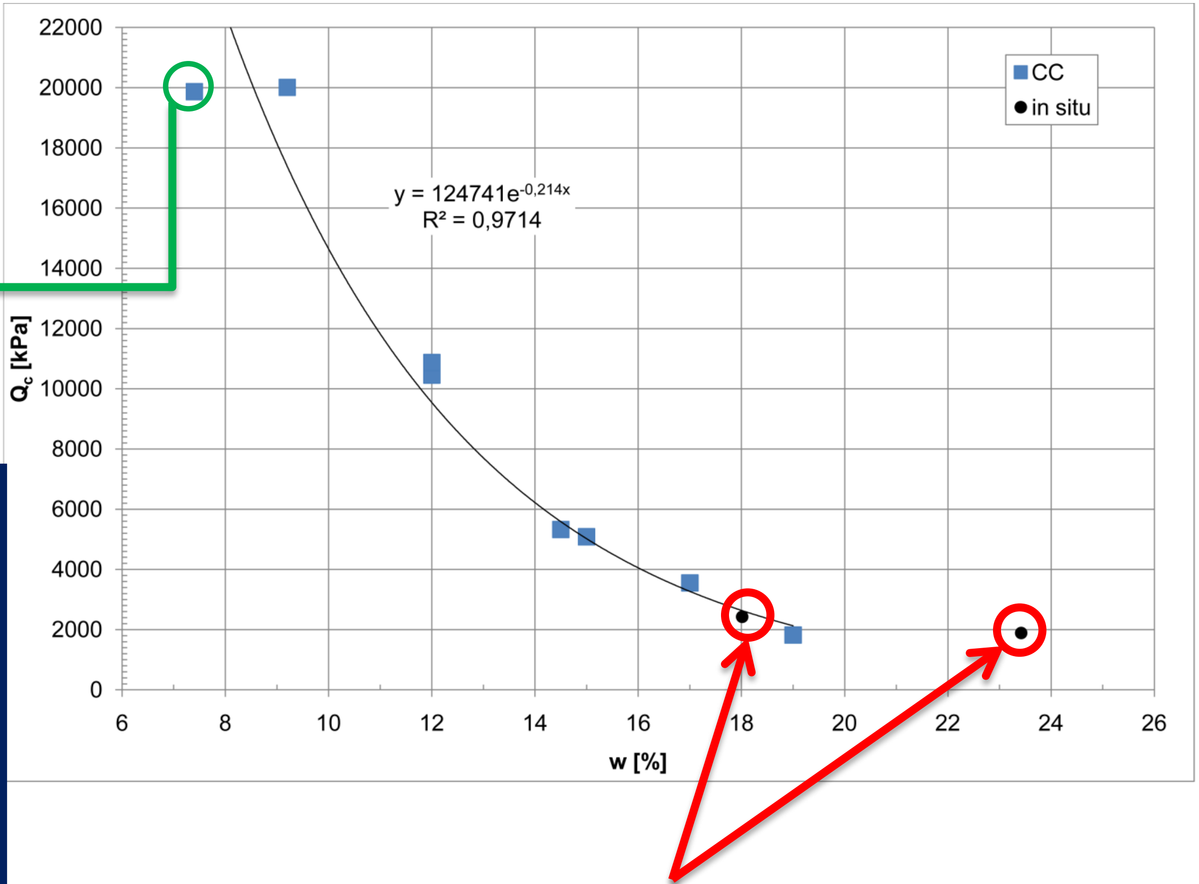
CASE HISTORY

TC

A6

γ_{dmax} = 1895 kg/m^3

w_{opt}=12%

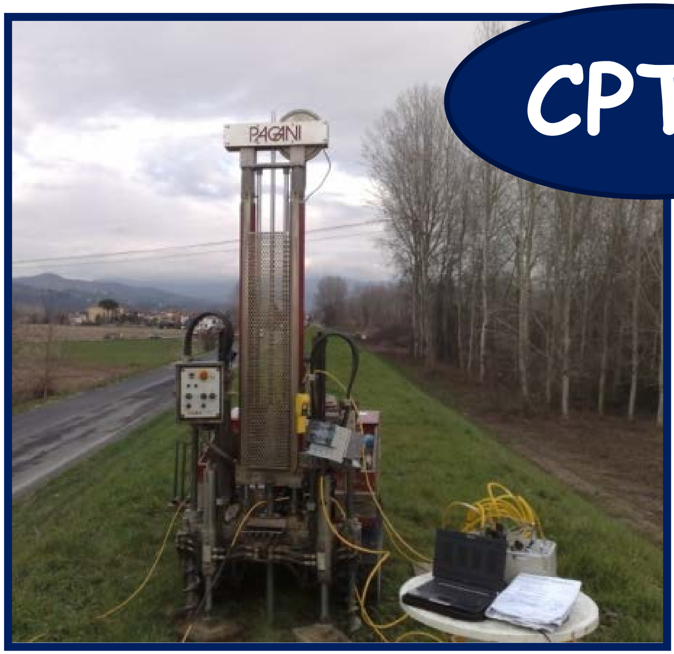


In situ CPTUs

CC tests:

Test number	Date of the test	Time [Days]	w [%]	Qc [MPa]
1	22/12/2014	0	12	10.451
2	07/01/2015	16	14.5	5.329
3	14/01/2015	23	17	3.553
4	20/01/2015	29	19	1.821
5	03/02/2015	43	15	5.083
6	13/02/2015	53	12	10.870
7	02/03/2015	70	9.2	20.010
8	24/03/2015	92	7.4	19.867

Note: Soil sample: TC; γ_d= 0.9γ_{dmax}



CPTu

q_c

VS

LFWD

E_d

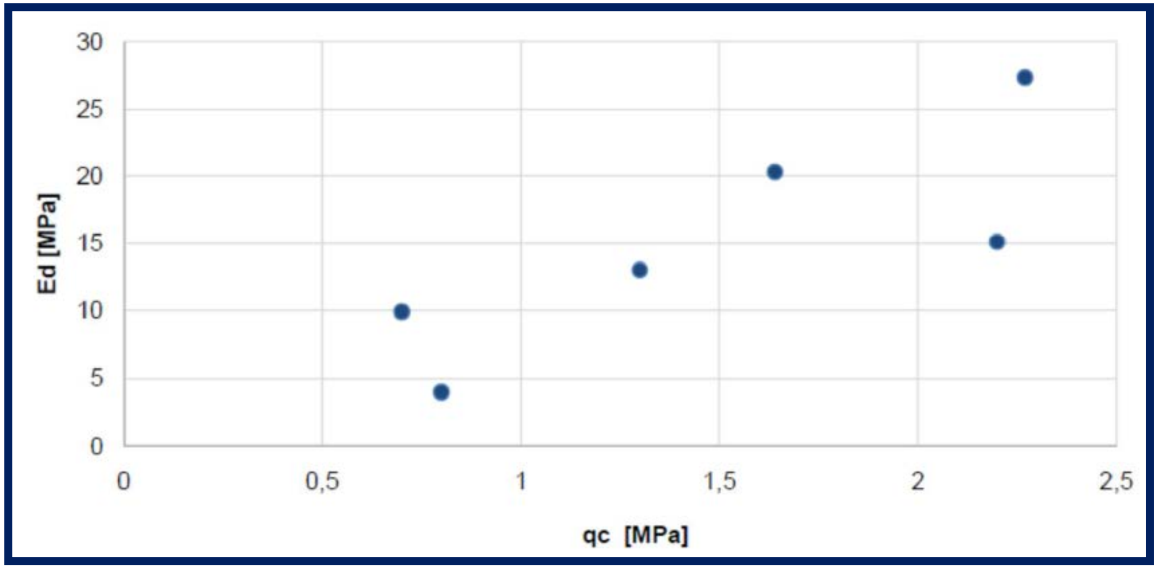
Hypothesis:

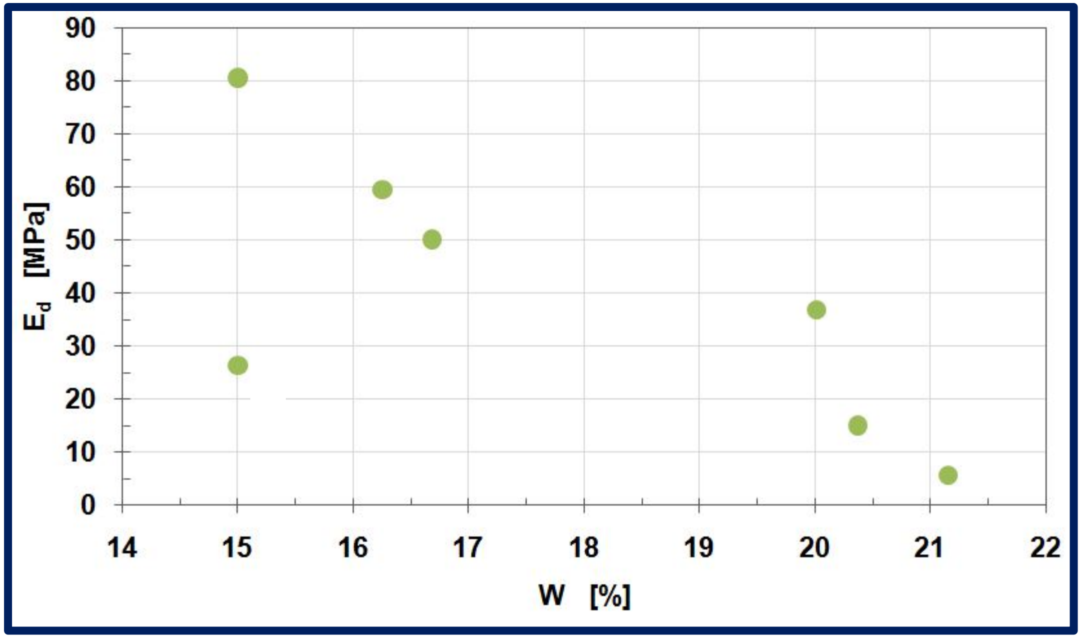
Correlation between E_d and q_c

Average q_c measured within the influence depth for the LFWD (approximately 1.5 times the diameter of the loading plate = 45cm)

SIMPLE PREVISION METHOD FOR QUALITY CONTROL

CPTu vs LFWD





As the water content decreases, E_d value increases

CONCLUSIONS

TICINO SAND

✓ In the case of granular soils, the tip resistance mainly depends on the relative density and the horizontal effective stress with a minor effect of the vertical effective stress.

PARTIALLY SATURATED FINE GRAINED SOILS

✓ The tip resistance essentially depends on the compaction energy (or maximum compaction stress) and water content.

✓ The tip resistance increases with water content reduction and elapsed time.

CONCLUSIONS

PARTIALLY SATURATED FINE GRAINED SOILS

✓ For a given soil and water content a correlation exists between tip resistance and soil dry density that can be used in practice to define a target tip resistance profile.

✓ For the fill compacted at a specified water content, the compacted dry density can be inferred from the field measurements of qc, after the correction of the qc measured values for the actual water content.

✓ Correlations between tip resistance and dynamic modulus as inferred from LFWD could be used for expeditious controls during the levee construction.

Thank you

barbara.cosanti@gmail.com

https://it.linkedin.com/pub/barbara-cosanti/5a/a08/909