Foundations for infrastructure projects in MENA

an approach to reduce risk and construction costs

Geotechnica ME - 4th & 5th December 2013

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Foundations for infrastructure projects in MENA
Table of contents

- Some critical geotechnical issues in MENA
- Common geotechnical approach in MENA
- An approach to reduce risk and construction costs
- Benefit of GI specification and supervision by specialist consultant
- Limitations of AASHTO for pile design in MENA
- Pile design in weak carbonate rocks – $\tau_{max} = f(\sigma_c)$ & cost savings
- Can we account for pile end bearing resistance?
- The importance of preliminary pile testing for design
- Osterberg Cell (O-Cell) test to avoid overdesign
- BS EN 1997-1 & Pile foundations
- Pile design to EC7 and $\tau_{max} = f(\sigma_c)$ – cost savings

Some critical geotechnical issues in MENA

Geotechnical Investigation Practices:

- Clients often see GI as mandatory without appreciating its benefits
- Quality of investigation is very variable
- GI interpretation (GIR) is often produced by GI contractors
- GI contractors often provide the only supervision themselves
- GI standards and methods used on projects are often very low

Geotechnical Design issues:

- Contract specifications often demand use a mixture of codes
- Approval bodies often misunderstand the difference between soil and rock
- Approval bodies staff often is not specialised in geotechnical engineering
- Geotechnical design is often done by the GI contractor

GI = Ground Investigation

Common geotechnical approach in MENA

- ~~Early stage project involvement~~
- ~~Desk study~~
- Design/scoping and specification of high quality GI
- Carry out GI
- ~~Full time~~ GI supervision
- Factual reporting by contractor
- ~~Review of factual reporting~~
- Interpretation
- Ground model + parameters + design

STANDALONE GROUND INVESTIGATION CONTRACTOR

CONSERVATIVE / UN-ECONOMIC / RISK ADVERSE SOLUTION

An approach to reduce risk and construction costs

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SPECIALIST GEOTECHNICAL CONSULTANT

- Early stage project involvement
- Desk study
- Design/scoping and specification of high quality GI
- Carry out GI
- Full time GI supervision
- Factual reporting by contractor
- Review of factual reporting
- Interpretation
- Ground model + parameters + design

By GI Contractor

ECONOMIC RISK CONTROLLED FOUNDATION SOLUTION

MENA geotechnical practices are getting better

- D&B contracts are becoming more common
- Field data used in back-analysis to validate geotechnical parameters
- GI contractors are getting more experienced and invest in newer/better plant
- The benefits of GI supervision by a specialist consultant is more accepted and even mandated in some cases
- Clients are now better informed and have more experienced staff
- Unforeseen ground conditions are getting recognised as a latent condition

Benefit of GI specification and supervision by specialist consultant

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Conventional single tube core barrel

Rotary coring with double tube core barrel (and plastic lining)

Two sites a few kilometres away from each other, at similar depth

Non intact core recovered as medium to coarse gravel of weak dolomitic limestone

Weak to medium strong, off-white to pale yellow Limestone with inclusions of silt/clay

Benefit of specialist consultant in GI specification and supervision

Conventional single tube core barrel

Non intact core recovered as medium to coarse gravel of weak dolomitic limestone

↓

Rock modelled as soil

↓

Low strength/stiffness

↓

Conservative Design

Pile design often to AASHTO

Rotary coring with double tube core barrel (and plastic lining)

Weak to medium strong, off-white to pale yellow Limestone with inclusions of silt/clay

↓

Rock modelled as rock

↓

High strength/stiffness

↓

Cost Efficient Design

Pile design for Carbonate rock

Benefit of specialist consultant in GI specification and supervision

Pile design in weak carbonate rocks
The limitations of AASHTO for pile design in MENA

- Based on a 35 years old correlation
- It does not take account of the recent research and fully instrumented pile load tests
- Not suitable for carbonate rocks that are predominant in MENA
- AASHTO τ_{max} correlation it is the most conservative published method when compared with correlations for carbonate rocks,
- Gives artificially low τ_{max} for RQD<50%

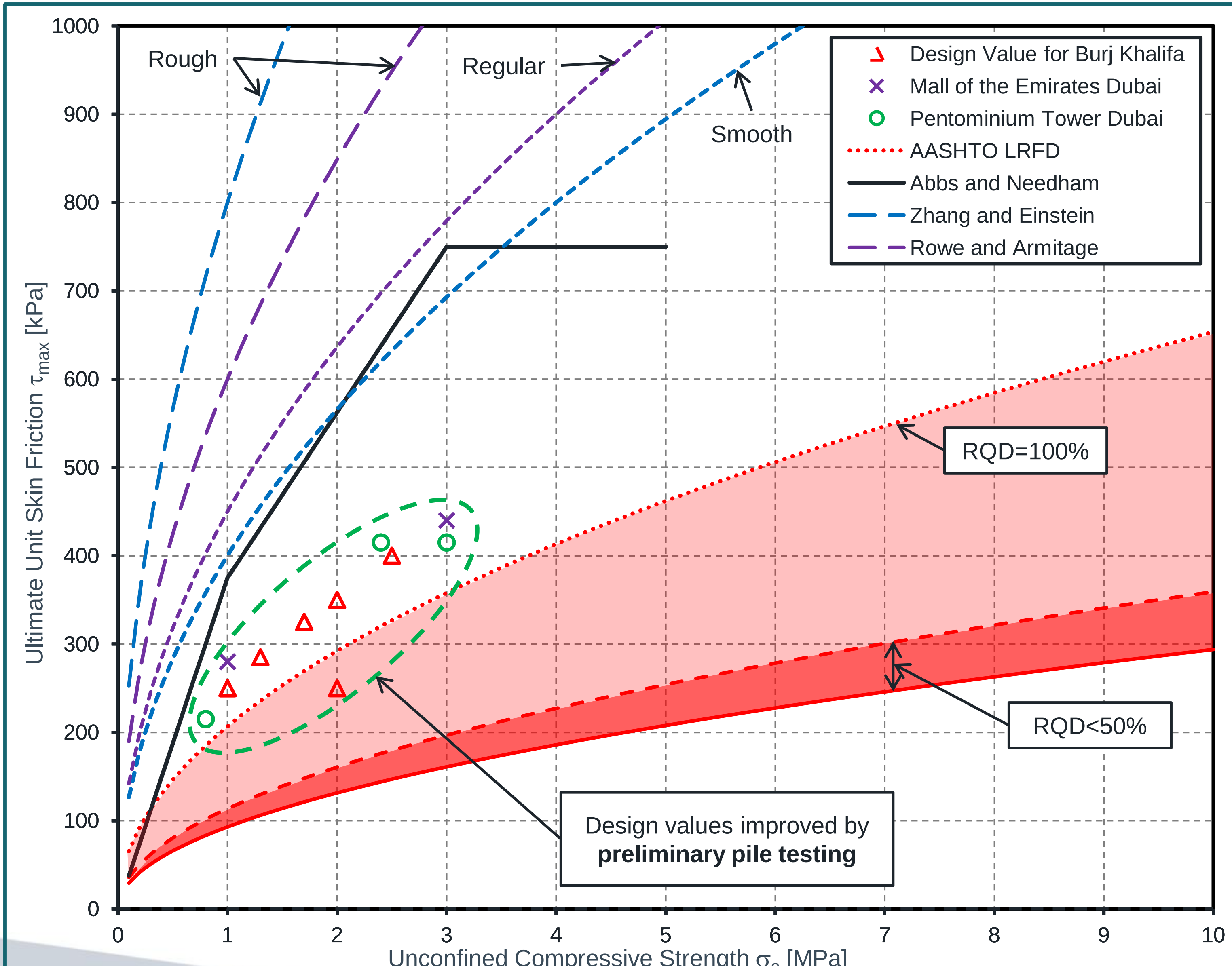
Table 10.4.6.5-1—Estimation of *E_m Based on RQD* *(after O'Neill and Reese, 1999)*

RQD (percent)	E_m/E_i	
	Closed Joints	Open Joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

Table 10.8.3.5.4b-1—Estimation of α_E ***(O'Neill and Reese, 1999)***

E_m/E_i	α_E
1.0	1.0
0.5	0.8
0.3	0.7
0.1	0.55
0.05	0.45

Pile design in weak carbonate rocks – $\tau_{max} = f(\sigma_c)$



Pile design in weak carbonate rocks - $\tau_{max}=f(\sigma_c)$

Year	Name	Recommended τ_{max}		Comments
1985	Abbs and Needham	$0.375*\sigma_c$ $0.375+0.1875*(\sigma_c-1)$ 0.750	$\sigma_c<1$MPa $\sigma_c=1-3$MPa $\sigma_c>3$MPa	Weak carbonate rock Calcarenite / Calcisiltite
1987	Rowe and Armitage	$0.45*(\sigma_c)^{0.5}$ $0.60*(\sigma_c)^{0.5}$	Regular Rough	Based on large number of field tests on weak rocks with no open discontinuities
1997	Zhang & Einstein	$0.40*(\sigma_c)^{0.5}$ $0.80*(\sigma_c)^{0.5}$	Smooth Rough	Recommendation base on a review of numerous available relationships
2010	AASHTO	$0.65*\alpha_E*p_a*(\sigma_c/p_a)^{0.5}$		Based on Horvath & Kenney, 1979 for Shale and Mudstone

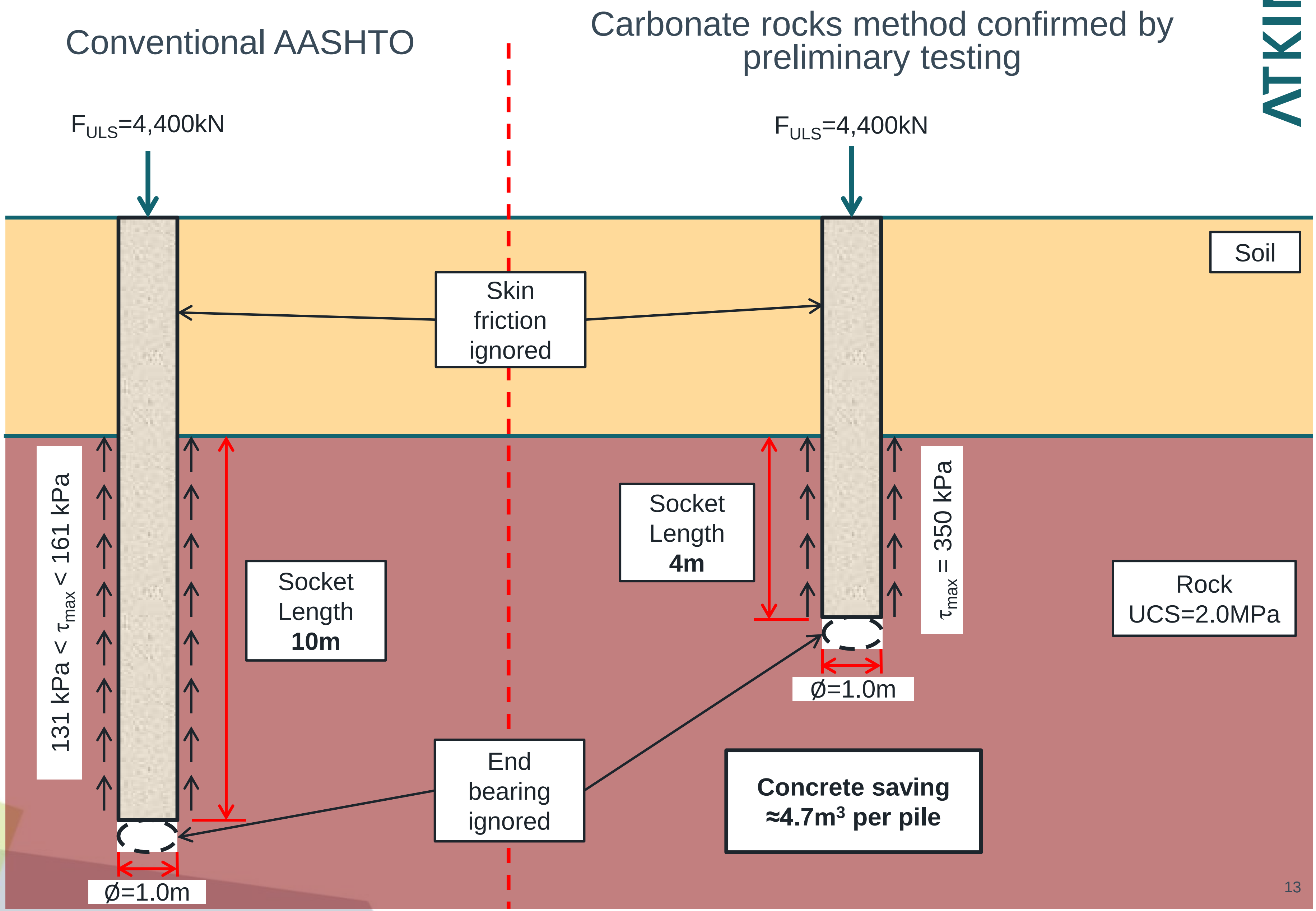
Legend:

σ_c Unconfined Compressive Strength (UCS)

α_E Reduction factor to account for jointing in rock

p_a Atmospheric Pressure (0.101 MPa)

Pile design in weak carbonate rocks = cost savings



Can we account for end bearing resistance?

- YES!
- There are many methods available to estimate the end bearing resistance of a pile socketed into weak rock, for example:
  - AASHTO LRFD: $q_p = 2.5*\sigma_c$
  - Zhang & Einstein, 1998: $q_p = (3.0 \text{ to } 6.6)*(\sigma_c)^{0.5}$

For $\sigma_c = 2.0$MPa

$4{,}200\text{kPa} < q_p < 9{,}300\text{kPa}$

Supplementary ultimate load bearing capacity for a 1.0m diameter pile:

3,300kN to 7,300kN

Same order of magnitude than **skin friction** contribution

Can we account for end bearing resistance?

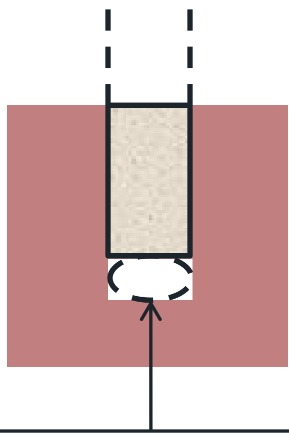
- Necessary precautions to account for end bearing

AASHTO LRFD

C10.8.3.5.4c

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

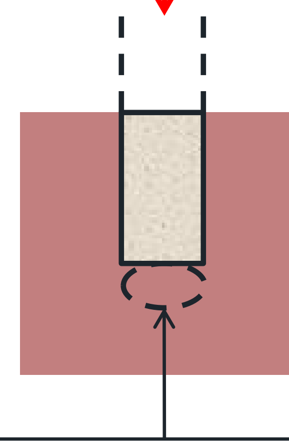
The use of Eq. 10.8.3.5.4c-1 also requires that there are no solution cavities or voids below the base of the drilled shaft.



No base cleaning = GAP

End bearing ignored

Often, programme constraints do not allow this procedure



Base cleaning = NO GAP

End bearing considered

ACCURATE SKIN FRICTION ESTIMATION IS PARAMOUNT

The importance of preliminary pile testing for design

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Conventional approach in MENA

Low quality GI

↓

Conservative skin friction estimate

↓

Conservative pile design

↓

Verification pile test
& Construction

Conservative design is built

No contingency measures if the verification test fails

Specialist Consultant

High quality GI

Realistic skin friction estimate ↔ Preliminary pile test to failure (O-Cell)

↓

Refined skin friction
& design update

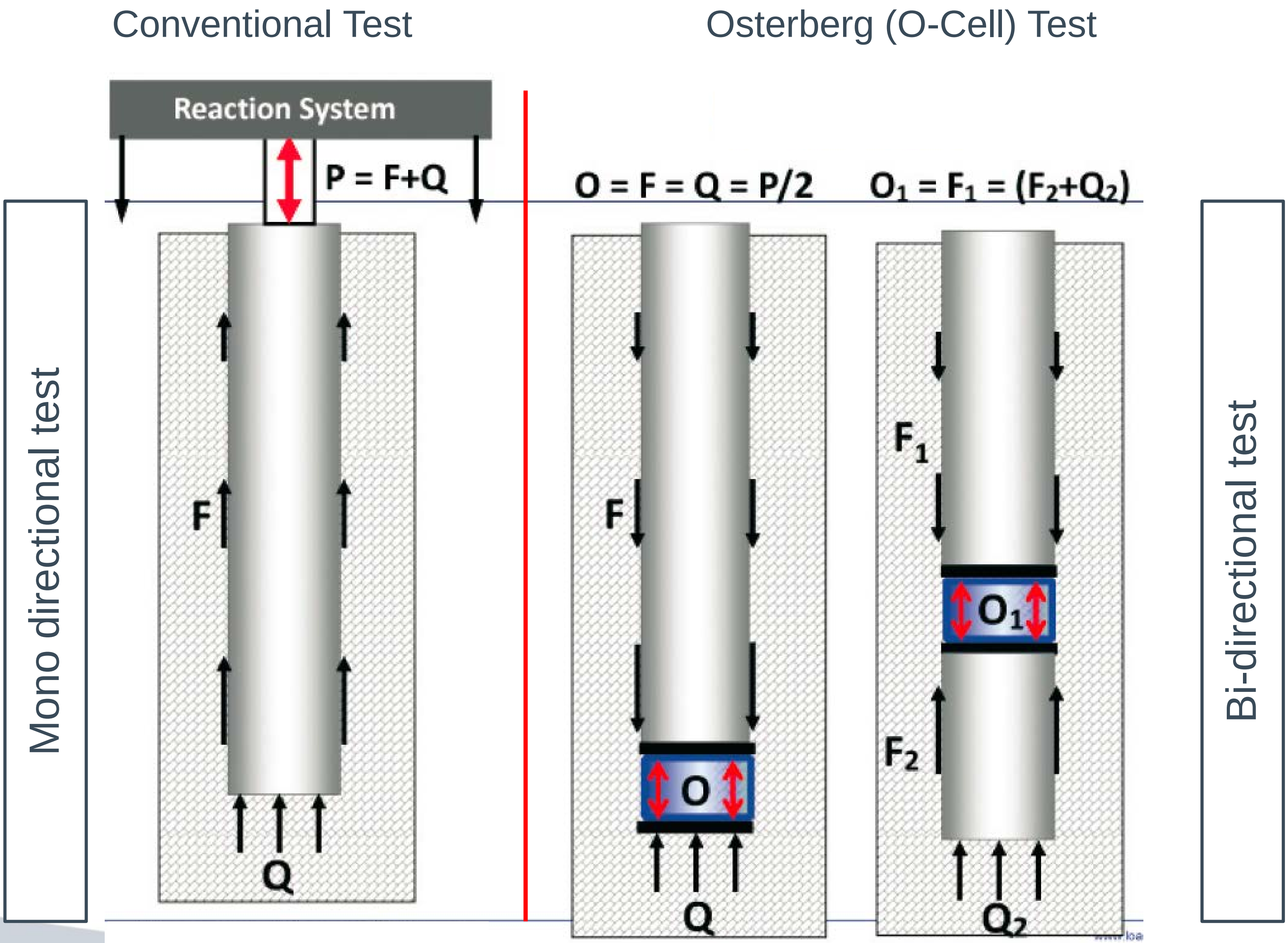
↓

Construction

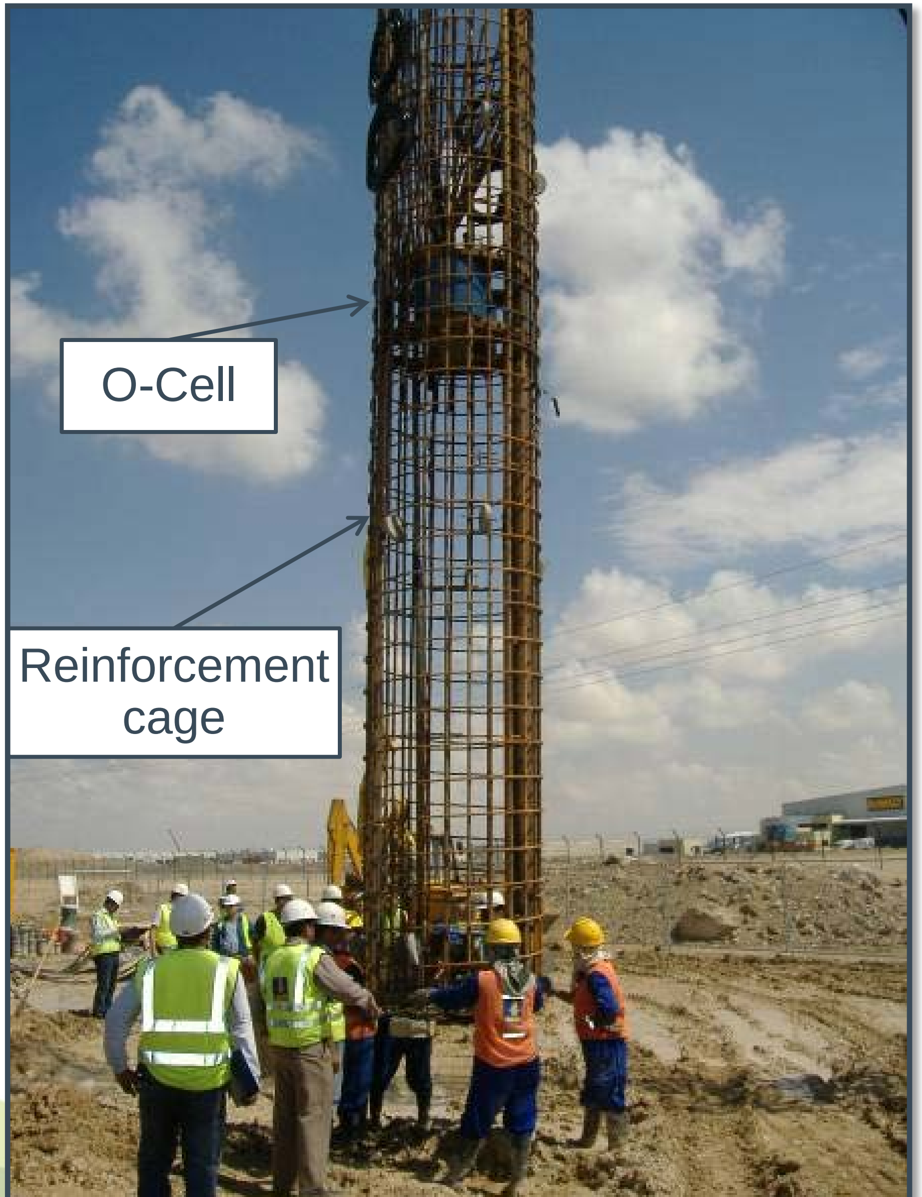
Cost-efficient design is built

Design assumptions and installation methods verified before construction

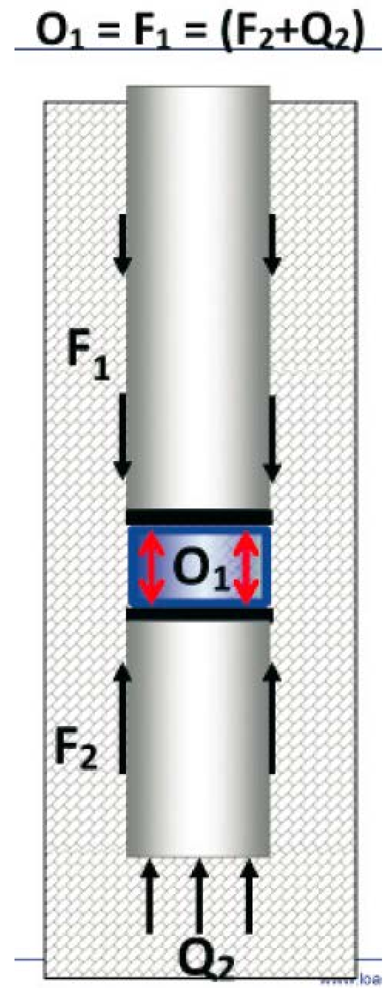
Pile testing solutions



Pile testing solutions



Osterberg (O-Cell) Test

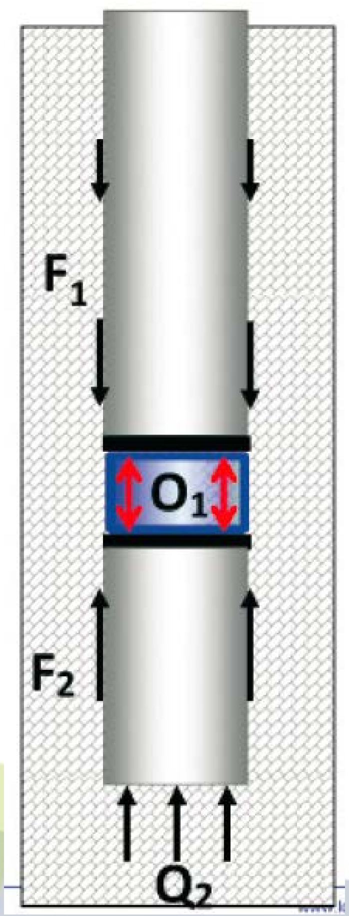


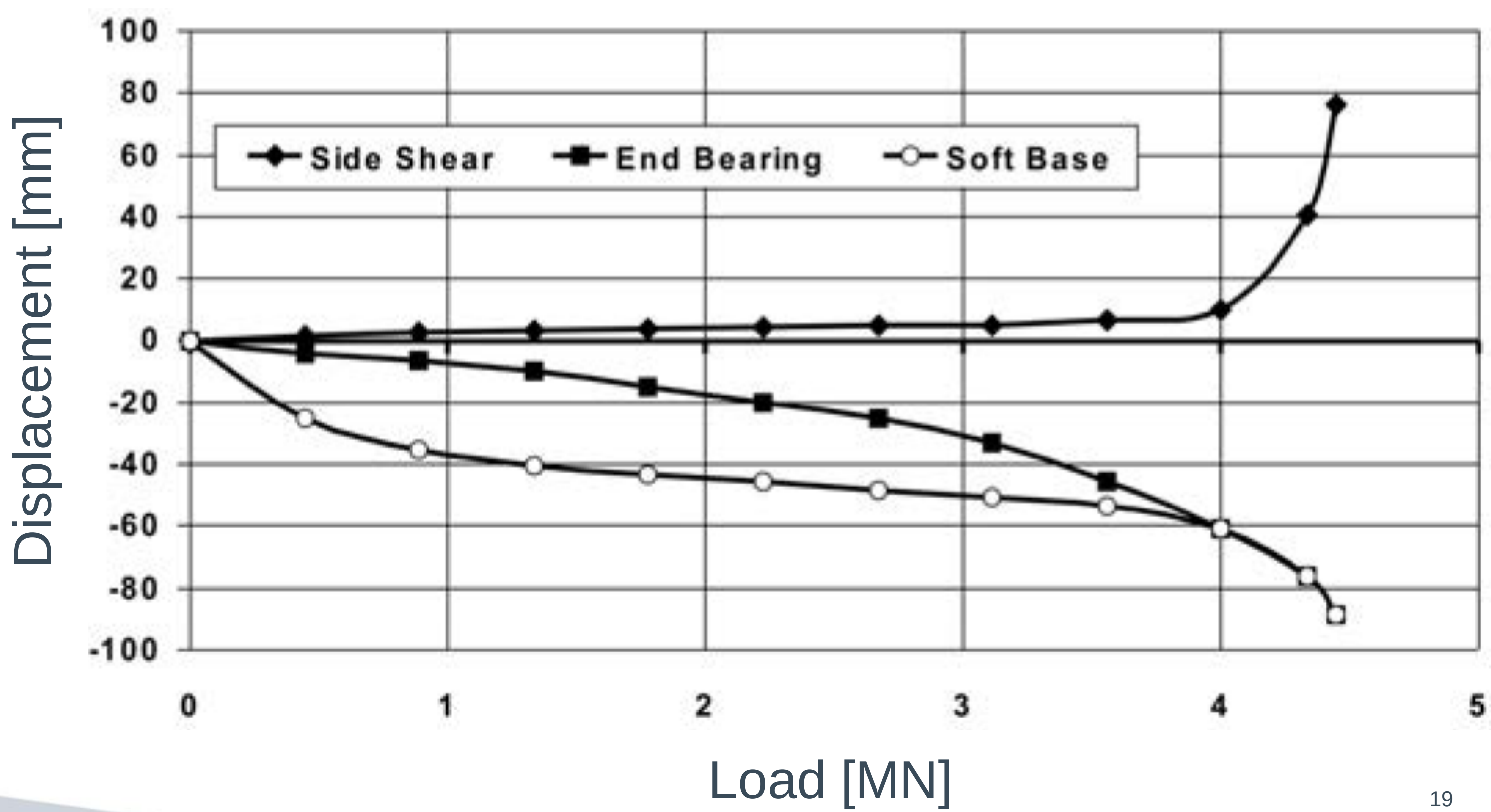
Bi-directional test

Osterberg Cell (O-Cell) test – Typical results

- Independent measurements of side shear and end bearing
- The test results are easier to incorporate in the design
- Helps identify improper construction techniques
- Conventional reaction system not required

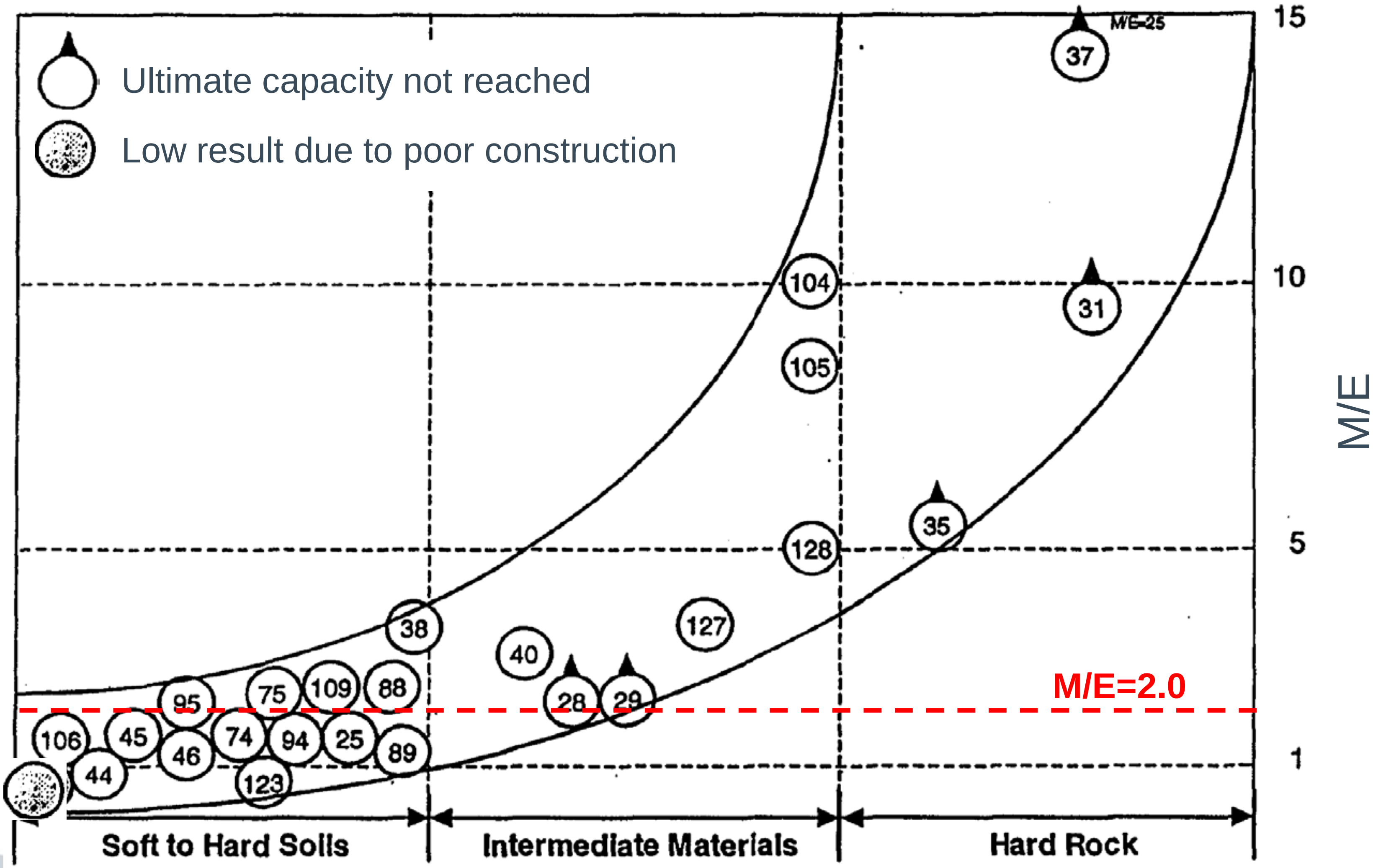
$O_1 = F_1 = (F_2+Q_2)$



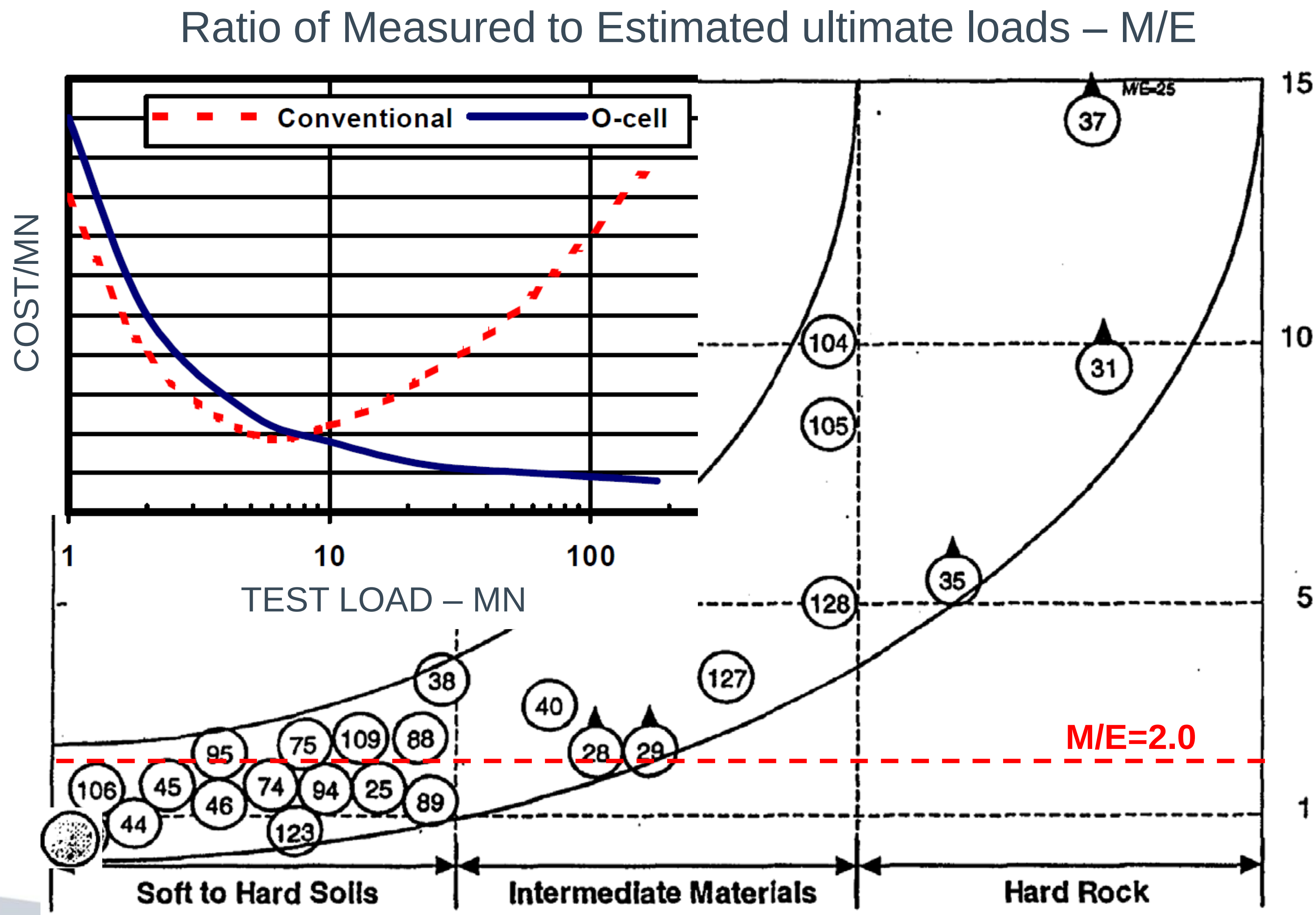


Osterberg Cell (O-Cell) test to avoid overdesign

Ratio of Measured to Estimated ultimate loads – M/E



Osterberg Cell (O-Cell) test to avoid overdesign



BS EN 1997-1 & Pile foundations

Eurocode 7 prescribes the following (Clause 7.6.2.1):

"*To demonstrate that the pile foundation will support the design load with adequate safety against compressive failure, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations:*"

$$F_{c;d} \leq R_{c;d}$$

Where: $F_{c;d}$ is the design axial compression load on a pile or a group of piles

$R_{c;d}$ is the design value of R_c, the compressive resistance of the ground against a pile, at the ultimate limit state

The design resistance in compression is given by:

ULS shaft resistance ULS base resistance

$$R_{c;d} = \frac{\sum_i (A_{s,i} \times q_{sk,i})}{\gamma_{Rd} \times \gamma_s} + \frac{A_b \times q_{bk}}{\gamma_{Rd} \times \gamma_b}$$

γ_{Rd} = Model Factor

Shaft Factor

Base Factor

BS EN 1997-1 & Pile foundations

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For the design of pile foundations, the UK National Annex to BS EN 1997-1 proposes different set of partial factors that relate to the amount of in situ pile testing carried out.

Clause A.3.3.2 recommends the following model factors:

- Without preliminary load test $\gamma_{Rd} = 1.4$
- With preliminary load test $\gamma_{Rd} = 1.2$

"If serviceability is verified by load tests (preliminary and/or working) carried out on more than 1% of the constructed piles to loads not less than 1.5 times the representative loads for which they are designed, the resistance factors can be reduced:"

For bored piles	γ_s	γ_b
• Without testing	1.6	2.0
• With testing	1.4	1.7

BS EN 1997-1 & Pile foundations in MENA

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- In MENA:
  - end bearing is often ignored
  - Bored piles are often used
- Using BS EN 1997-1 and the UK National Annex leads to savings:

	$\gamma_{Rd} \times \gamma_s$	Saving
FOS without testing	2.24	N/A
FOS with testing to UK NA	1.68	-25%

Pile design to EC7 and $\tau_{max}=f(\sigma_c)$ – cost savings

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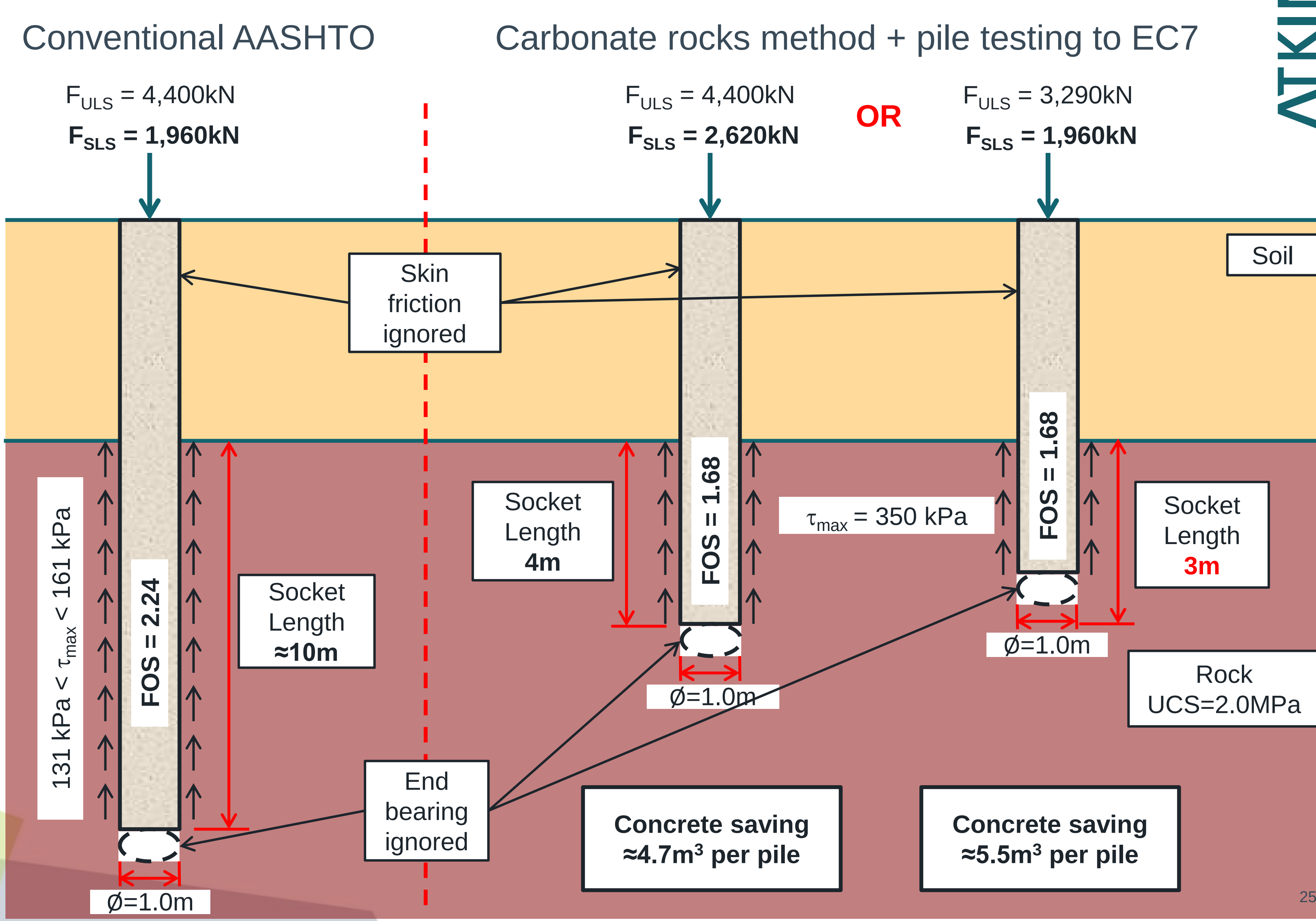

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By GI Contractor

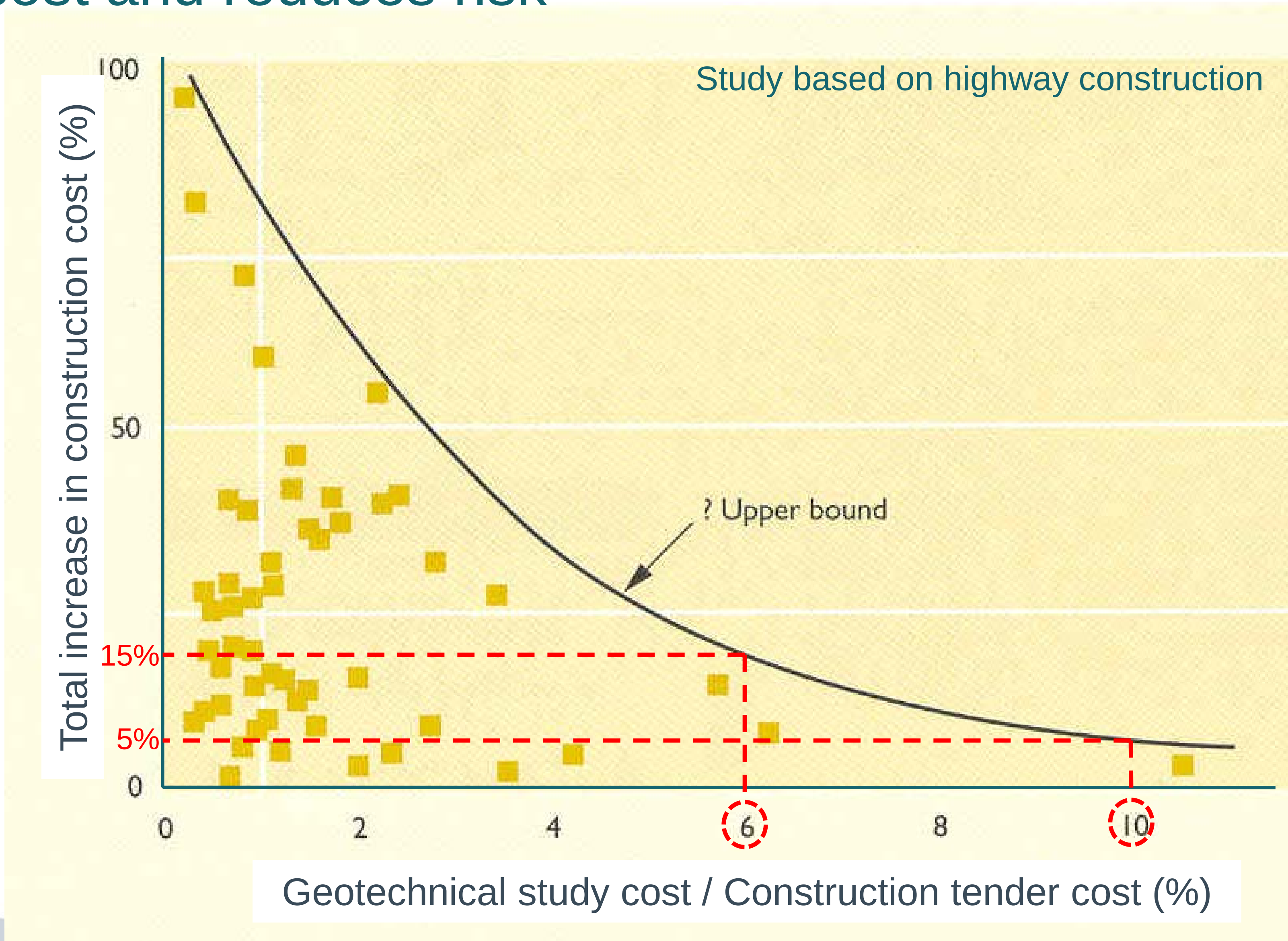
USE BS EN 1997-1 + UK NA

Prelim. Pile Testing

O-Cell testing

ECONOMIC RISK CONTROLLED FOUNDATION SOLUTION

Investment in high quality GI saves on construction cost and reduces risk



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QUESTIONS?

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