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MISE A JOUR DES ETUDES ET ASSISTANCE TECHNIQUE POUR LA CONSTRUCTION DU BARRAGE DE BISRI

BARRAGE BISRI

AVANT PROJET DETAILLE

PIECE 3: RAPPORT GEOTECHNIQUE

3-4: TECHNICAL NOTE ON LIQUEFACTION POTENTIAL ASSESSMENT

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BISRI DAM

TECHNICAL NOTE 3

LIQUEFACTION POTENTIAL ASSESSMENT

Introduction and goal

1 Bisri Dam (Lebanon) will be constructed with an inclined clay core supported by gravel/rockfill shoulders (Figure 1). It will have a maximum height of 74 m above natural ground level reaching an elevation of +468 m. Normal water level elevation will be +461 m.

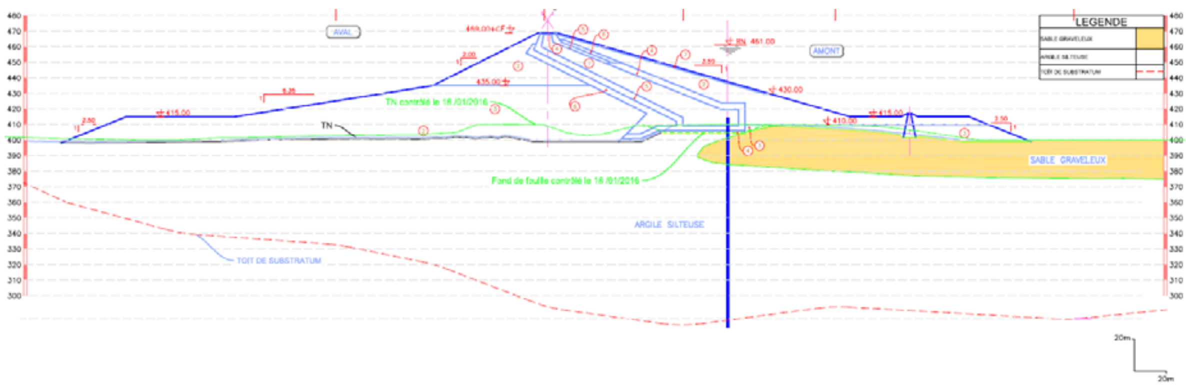

Figure 1. Typical cross-section of Bisri dam

2 The dam will be founded on a thick alluvial deposit of lacustrine origin including both coarse and fine–grained materials. In addition to min site investigations carried out in 1996/1997 and 2014, an extensive CPTU testing campaign has been performed in 2015. The characteristics of this camping and the main results obtains have been presented in a recent Technical Note (UPC 2016).

3 Bisri dam is sited in a highly seismic zone; it is of interest, therefore, to assess the potential for liquefaction of the foundation soils. In this Technical Note, the liquefaction assessment is performed using some of the results of the CPTu tests.

Liquefaction assessment

4 The initial design assumptions concerning the seismicity of the sire are presented in Table 1. In the following, the calculations will be performed using a value of horizontal peak ground acceleration (a_H) of 0.70g combined with an earthquake magnitude of 7.5.

Table 1. Seismic assumption of the initial design. NOVEC CDG development- Dar al Handasah nazih Taleb & Partners (2013).

Criteria	Source	
	Roum	Yammouneh
Length, km	50	600 - 1,000
Distance from Site, km	2	10
MCE Magnitude	7.3	8.5
Bracketed Duration, sec	20	45
Peak Ground Accelerations at Damsite - Horizontal - Vertical	 0.70 g 0.47 g	 0.55 g 0.37 g

5 The liquefaction analysis is performed using the method advocated by Robertson (2009b) for medium risk projects. The method is summarily described in Appendix 1. According to Boulanger and Idriss (2007) and Robertson (2009b) for high risk projects, it is important to test high quality samples with cyclic laboratory testing, but the feasibility of this option for sandy soils using conventional site investigation means can be questioned.

6 The ground more likely to be affected by liquefaction is located below the upstream and downstream berms (Figure 2), so the study is focused on those soil profiles. Consequently the CPTu tests considered for liquefaction analyses are CPTuVR1, CPTuVR2, CPTuVR5 and CPTuVR14. Their locations are indicted in Figure 3. CPTuVR1 and CPTuVR2 are located downstream and CPTuVR5 and CPTuVR14 are located upstream. The soil types estimated from the value of Soil Index Behaviour, Ic, are indicated in Figures 4a, 6a, 8a and 10a.

7 To broaden the scope of the analyses, it will be assumed that the height of the dam above the soil profile considered is 5m, 10, and 15 m resulting on an applied vertical stress of 100kPa, 200KPa and 300kPa.The depth of the water table ranges from 4 to 10 m. Obviously, the soil above the water table is not susceptible to liquefaction unless it becomes saturated at some stage..

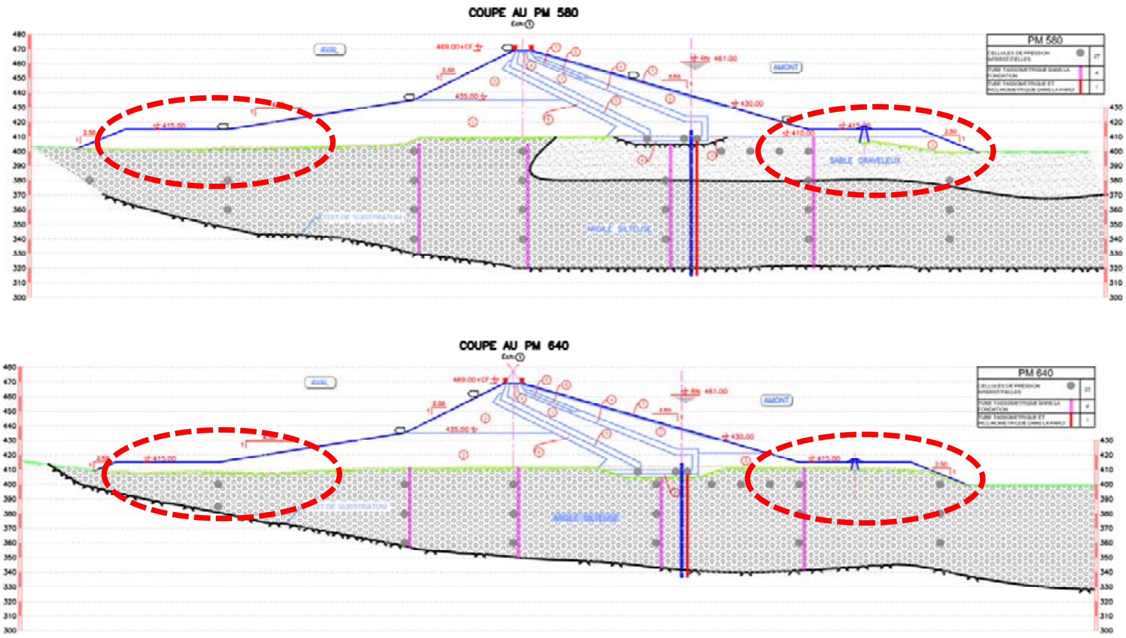

Figure 2. Potentially liquefiable foundation soil zones

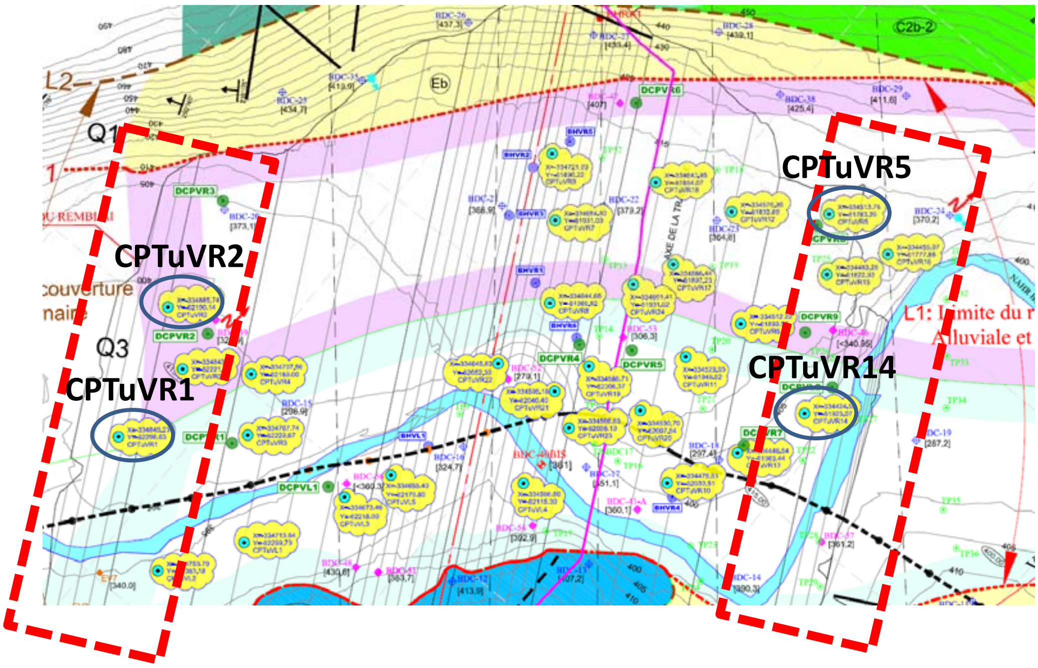

Figure 3. Location of the CPTu tests considered in the liquefaction analyses

8 Liquefaction analyses require the computation of the cyclic stress ratio (CSR) and the cyclic resistance ratio (CRR). CSR are computed following the method presented in Appendix 1 whereas CRR are estimated form CPTu results (see also Appendix 1). The results are presented in Figures 4, 6, 8 and 10 in terms of distributions of factor of safety (FoS), of vertical stresses, and of CRR & CSR values for the three assumed values of dam height. The factor of safety is computed as CRR/CSR.

9 Factors of safety (FoS) are plotted again individually for each of the three dam heights in Figures 5, 7, 9, 11. It can be observed:

- For the VR-1 soil profile, the soils for 12 m below the water table are sandy soils yielding FoS lower than 1. From 16 to 30 m depth, the soil is clayey and FoS reach values around 1 for the two higher dam assumptions.
- In the case of VR-2 soil profile, again the shallow sandy soils are very liquefiable (low FoS in all cases) whereas values close or above one are obtained for the more clayey soils below 18 m depth.
- For the VR-5 profile the lower clayey soil is more interlayered providing an alternation of low and higher FoS. The shallower sandy zone gives again low FoS. Note that in this CPTu tests, the water table is 10m deep.
- In the VR-14 the soil is very sandy except at depths deeper than 24 m. Low FoS are obtained throughout.

Conclusion

10 The computations performed indicate that the foundation soils are potentially liquefiable; the sandy layers being significantly more susceptible. This is not surprising considering the very high design seismic acceleration and the soft/loose nature of the alluvial materials that constitute the dam foundation.

Barcelona, 7th March 2015

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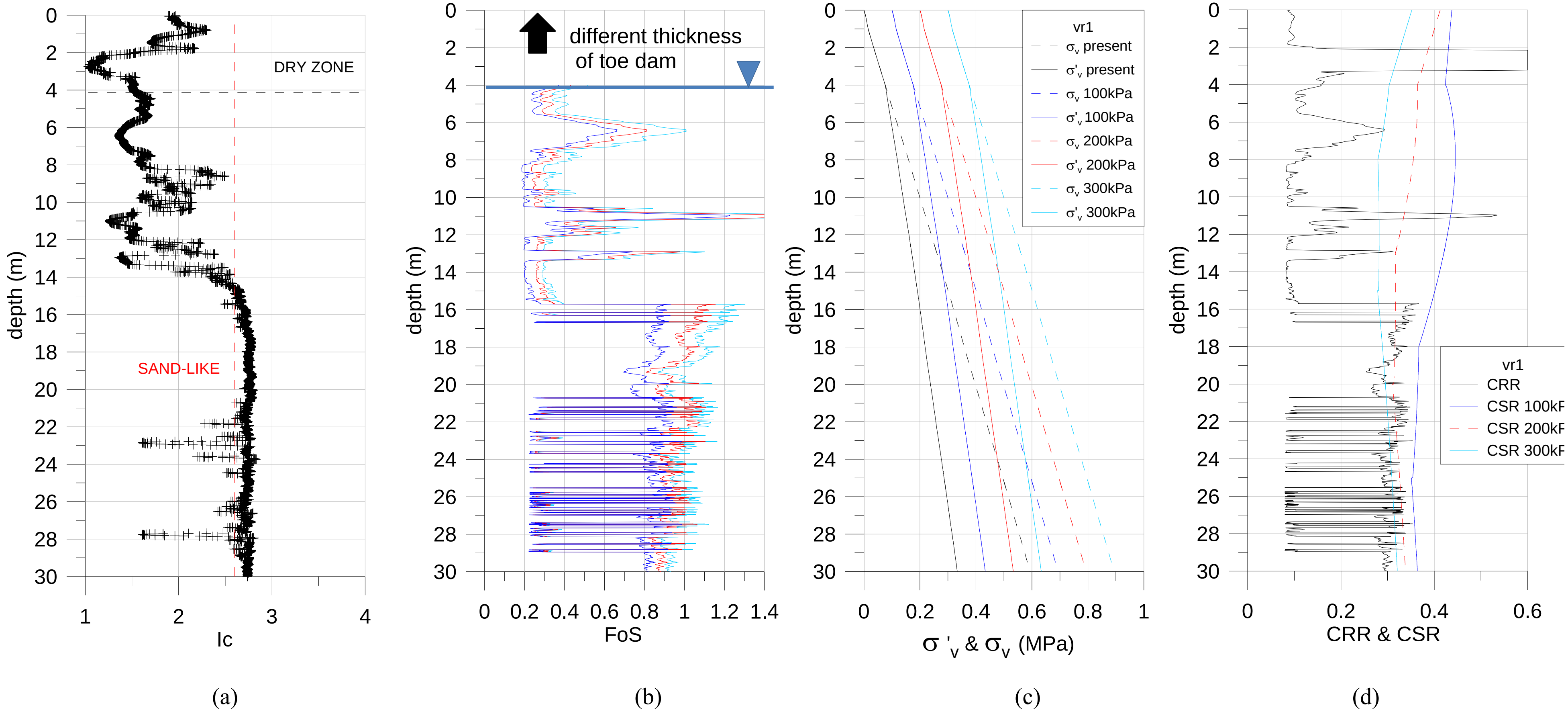


Figure 4. VR-1 CPTu, (a) Soil index behaviour, (b) Factor of safety, (c) vertical stresses for CSR calculations, (d) CRR & CSR.

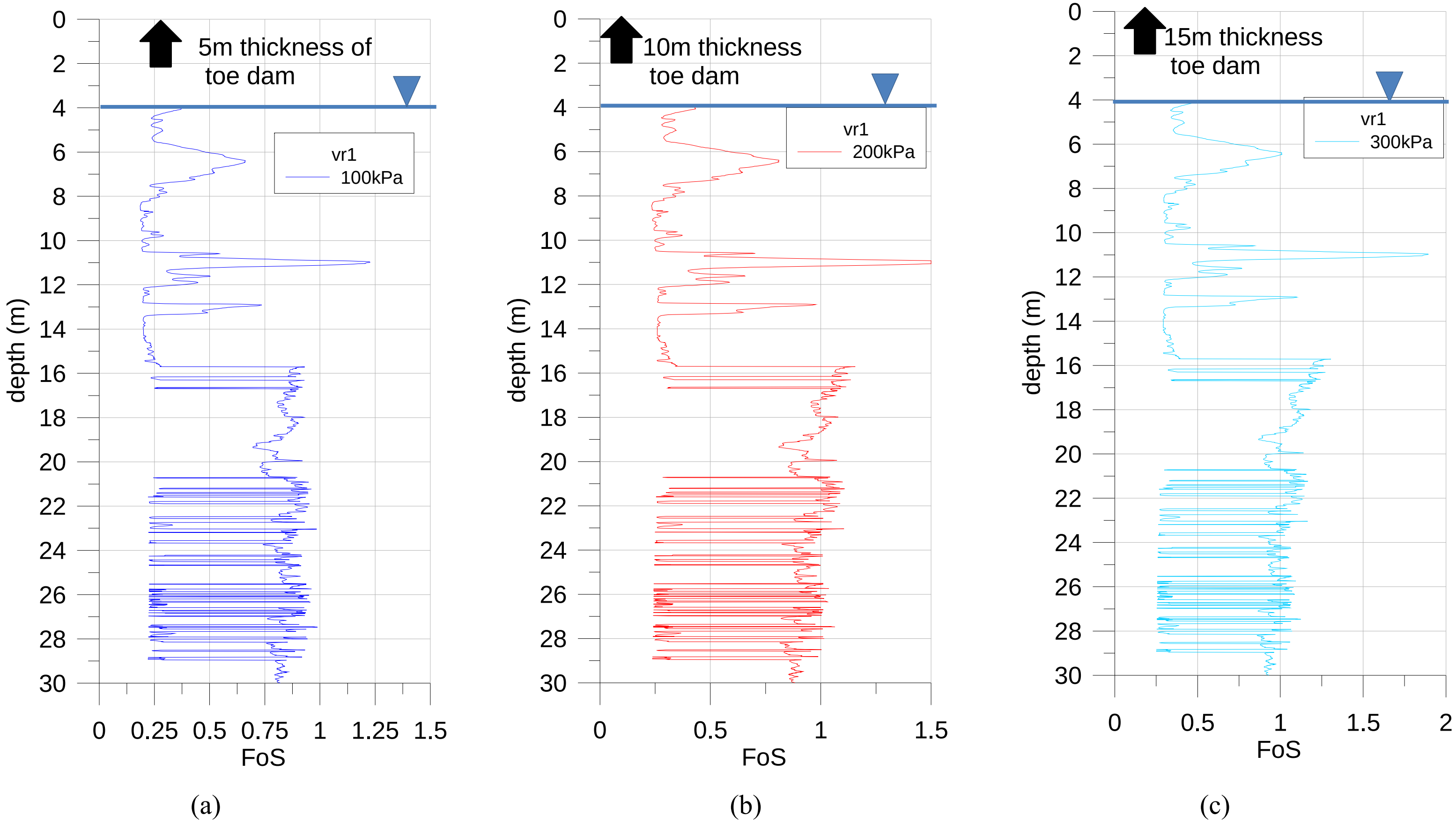


Figure 5. VR-1 CPTu, Factors of safety (FoS) computed assuming a dam height of (a) 5m (100 kPa), (b) 10 m (200kPa), (c) 15 m (300 kPa).

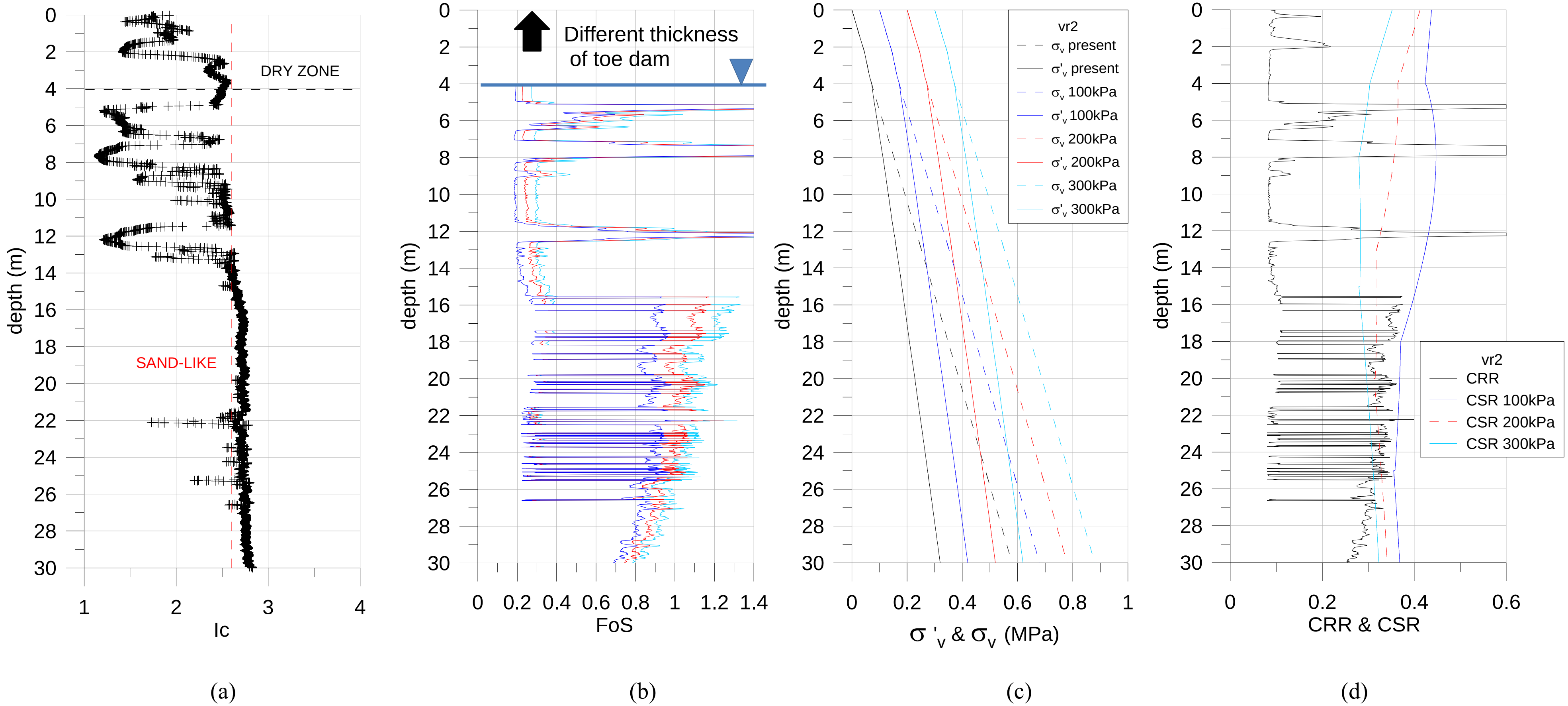


Figure 6. VR-2 CPTu, (a) Soil index behaviour, (b) Factor of safety, (c) vertical stresses for CSR calculations, (d) CRR & CSR.

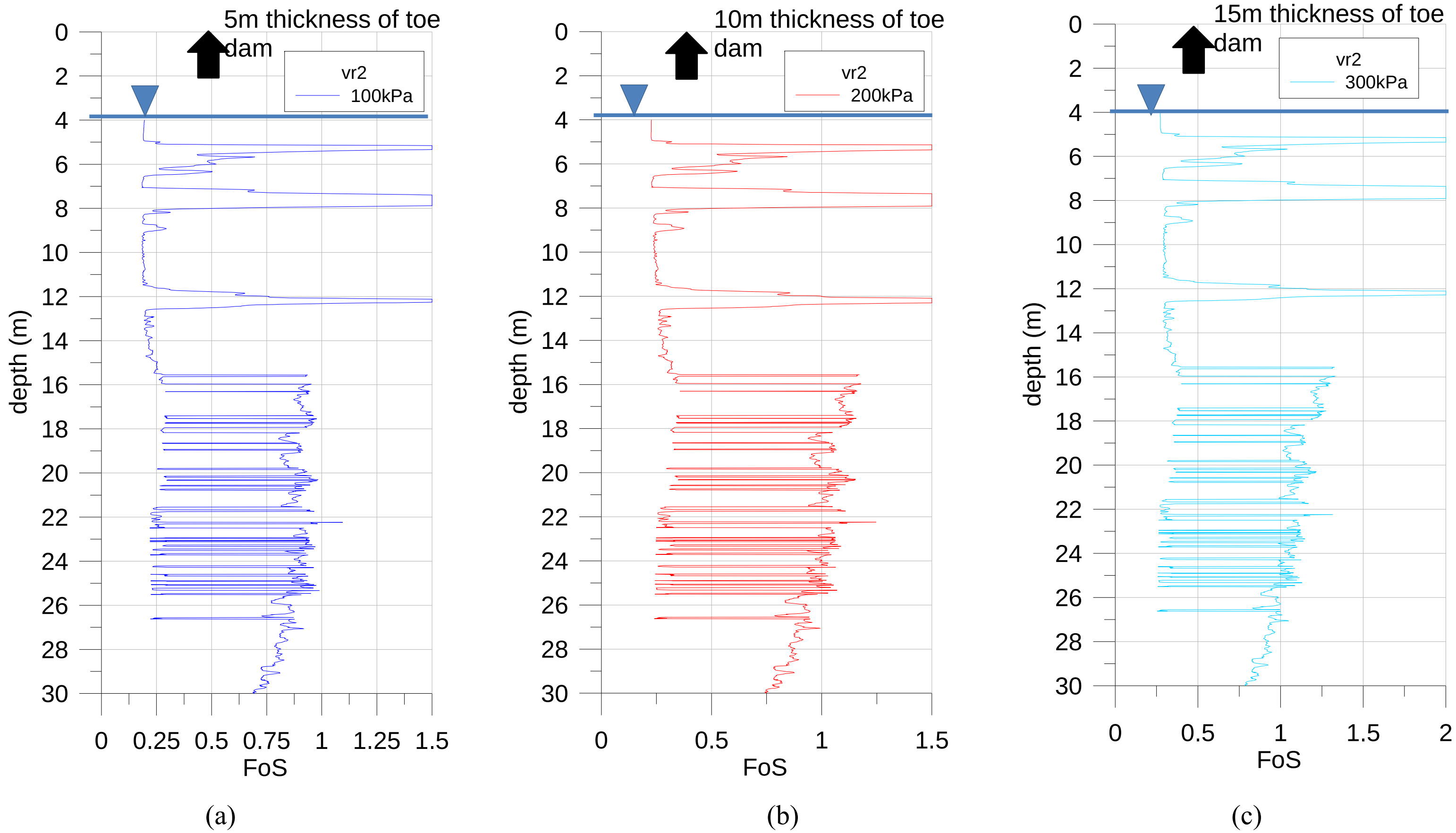


Figure 7. VR-2 CPTu, Factors of safety (FoS) computed assuming a dam height of (a) 5m (100 kPa), (b) 10 m (200kPa), (c) 15 m (300 kPa).

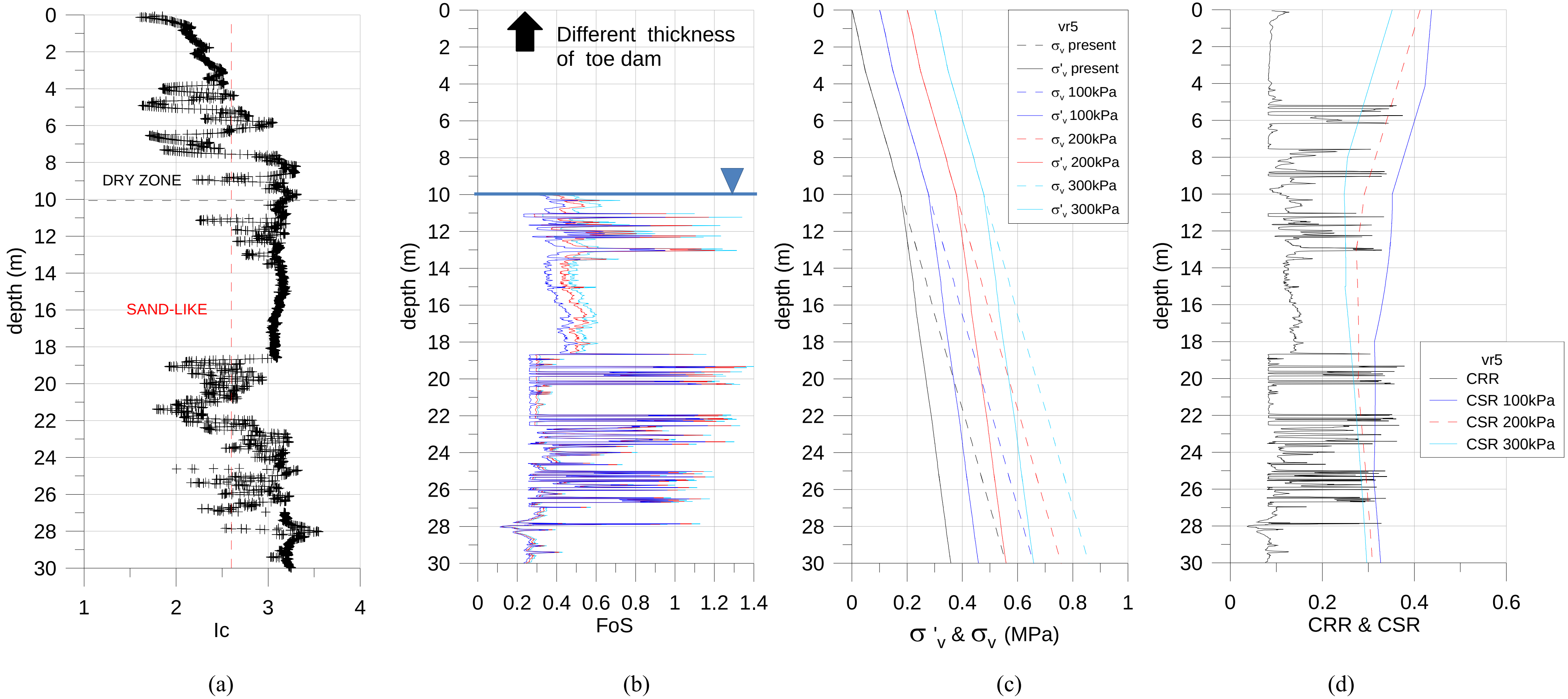
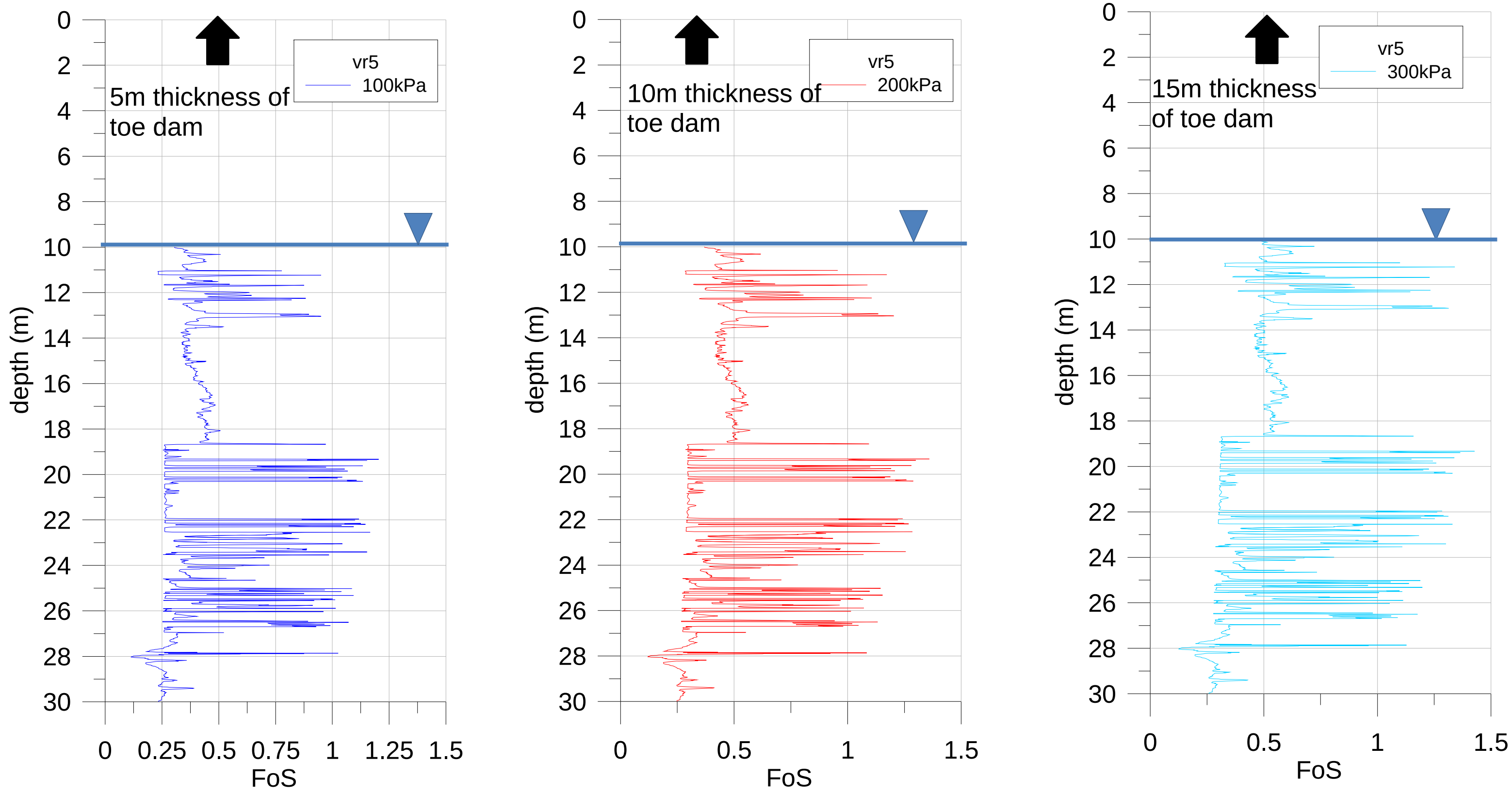


Figure 8. VR-5 CPTu, (a) Soil index behaviour, (b) Factor of safety, (c) vertical stresses for CSR calculations, (d) CRR & CSR.



(a) (b) (c)

Figure 9. VR-5 CPTu, Factors of safety (FoS) computed assuming a dam height of (a) 5m (100 kPa), (b) 10 m (200kPa), (c) 15 m (300 kPa).

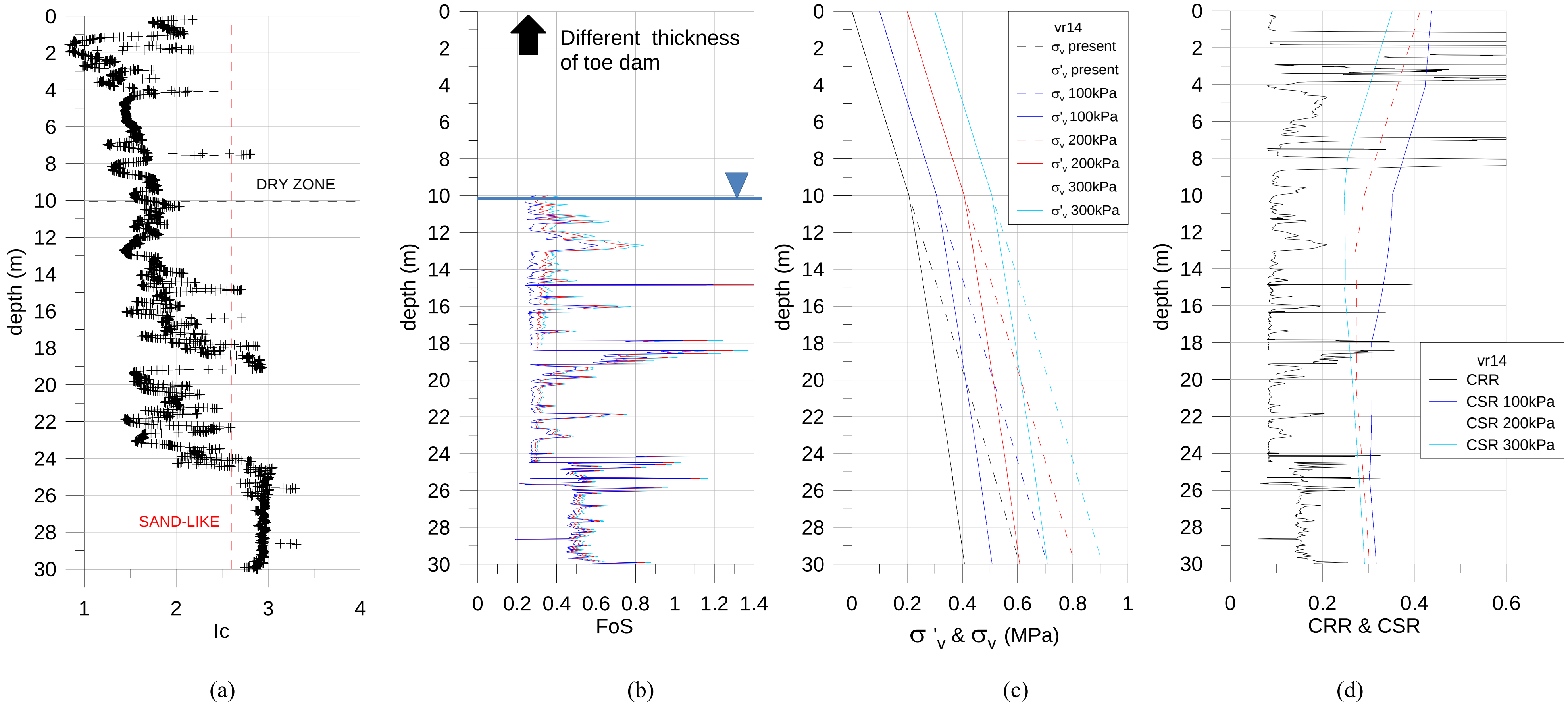
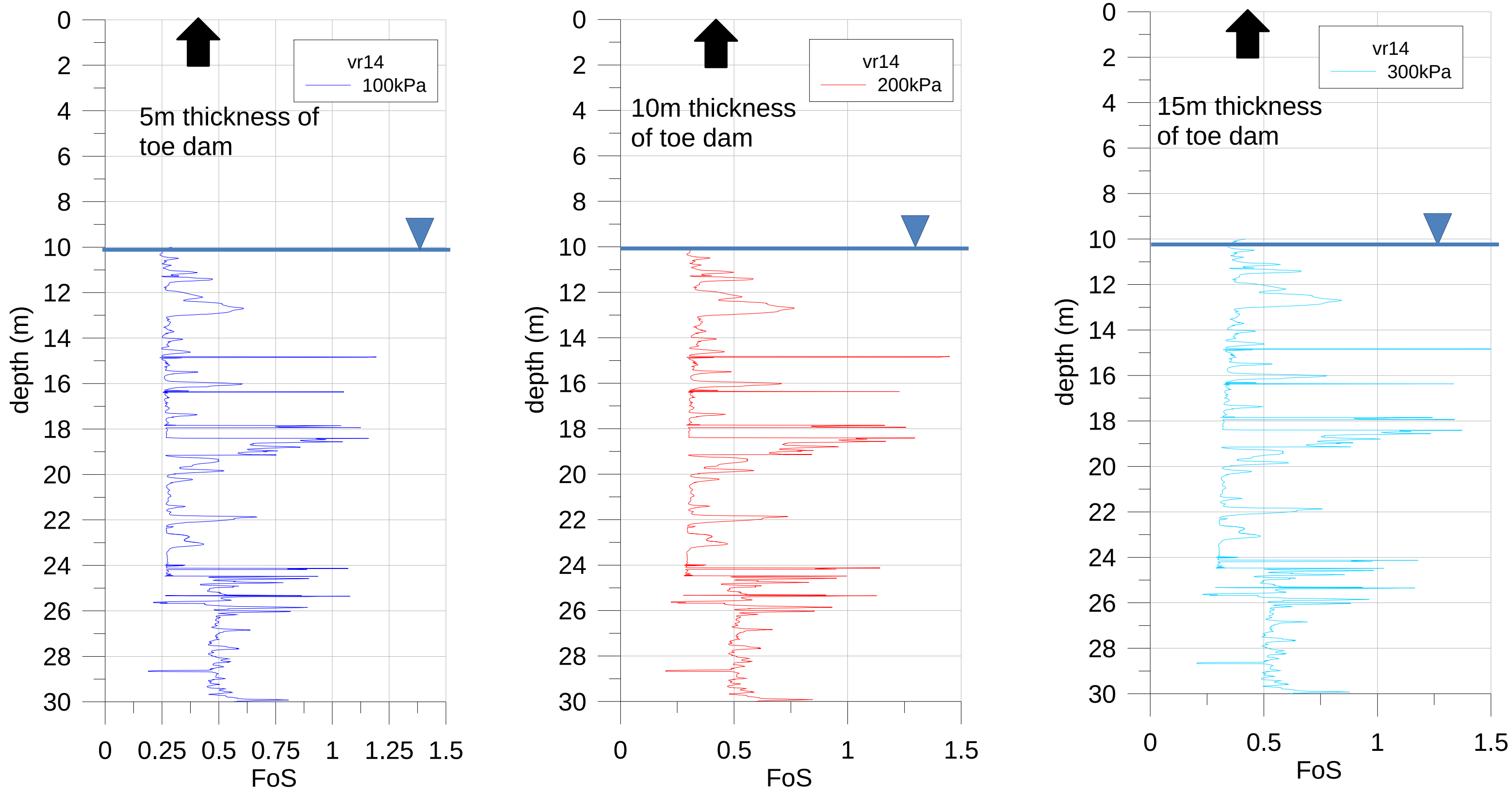


Figure 10. VR-10 CPTu, (a) Soil index behaviour, (b) Factor of safety, (c) vertical stresses for CSR calculations, (d) CRR & CSR.



(a) (b) (c)

Figure 11. VR-14 CPTu, Factors of safety (FoS) computed assuming a dam height of (a) 5m (100 kPa), (b) 10 m (200kPa), (c) 15 m (300 kPa).

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Appendix 1

Liquefaction assessment method (Robertson, 2009)

Computation of the Cyclic Stress Ratio (CSR)

The peak cyclic stress ratio $(CSR)_{peak}$ induced by an earthquake is the maximum tangential stress, τ_{peak}, divided by the effective vertical stress (Seed and Idriss 1971).

$$(CSR)_{peak} = \frac{\tau_{peak}}{\sigma'_v} \quad [1]$$

CSR_{peak} is modified by two factors r_e and r_d to obtain $(CSR)_M$, the average CSR. r_e is the required to obtain a representative percentage of the earthquake shear stress, usually 65% of the τ_{peak} (r_e=0.65). r_d is a stress reduction factor dependent on depth.

$$(CSR)_M = r_d \cdot r_e \cdot \frac{\tau_{peak}}{\sigma'_v} \quad [2]$$

Several approaches are present in the literature to obtain r_d; the most commonly used is the approach of Liao and Withman (1986b) that evaluates r_d dependsing on depth (z).

$$r_d = 1 - 0.00765\ z \qquad z \leq 9.15\ \text{m} \quad [3]$$

$$r_d = 1.174 - 0.0267\ z \qquad 9.15\ \text{m} < z \leq 23\ \text{m} \quad [4]$$

$$r_d = 0.744 - 0.008\ z \qquad 23\ \text{m} < z \leq 30\ \text{m} \quad [5]$$

$$r_d = 0.5 \qquad z > 30\text{m} \quad [6]$$

τ_{peak} is derived from the vertical and horizontal forces of a soil column according to:

$$\tau_{peak} = \sigma_v \frac{a_{max}}{g} \quad [7]$$

where: σ_v is total vertical stress, a_{max} is the maximum possible acceleration of the study area due to an earthquake and g is the gravity acceleration.

Equation[8] summarizes the above calculation as:,.

$$(CSR)_M = \frac{\tau_{av}}{\sigma'_v} = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_v}{\sigma'_v}\right) \cdot r_d \quad [8]$$

and corresponds to a M_w=7.5 earthquake. Therefore this expression is not corrected as design magnitude for Bisri Dam is 7.3.

Computation of the Cyclic Resistance Ratio (CRR)

Cyclic resistance ratio (CRR) is defined as the quotient between the resistance shear stress and the vertical effective stress. A methodology was developed by Robertson (1998) and (2009) in order to obtain CRR from CPTu parameters: cone resistance (q_c), sleeve friction (f_s) and water pore pressure (u), frequently measured at the cone shoulder (u_2).

q_c have to be corrected to total cone resistance (q_t) using Baligh et al. (1980) expression :

$$q_t = q_c + (1-a)\cdot u_2 \qquad [9]$$

where: a is net area ratio of the cone apparatus.

A number of ratios are used to evaluate soils: normalized resistance parameter (Q), friction ratio (F) and pore pressure ratio (B_q) (Wroth 1984, Senneset and Janbu (1985).

$$Q = \frac{(q_c - \sigma_{v0})}{\sigma'_{v0}} \qquad [10]$$

$$F = \frac{f_s}{(q_t - \sigma_{v0})}\cdot 100 \qquad [11]$$

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \qquad [12]$$

where σ_v is the total vertical stress and σ'_v is the vertical effective stress.

Subsequently, Roberston and Wride (1998) proposed a normalizzed equivalent clean sand applicable to all soils:.

$$Q_{tn} = q_{c1N} = \left[\frac{(q_t - \sigma_v)}{p_a}\right]\cdot\left[\frac{p_a}{\sigma'_v}\right]^n \qquad [13]$$

where: p_a is the atmospheric pressure and n is the stress exponent which varies with soil behaviour type.

Robertson (2009a), after the suggestion of Zhang et al. (2002), proposed the following equation for n .

$$n = 0.381(I_c) + 0.05(\sigma'_v/p_a) - 0.15 \qquad [14]$$

where $n \leq 1.0$.

where the soil behaviour type index (I_c) is given by (Jefferies and Davies 1993):

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \qquad [15]$$

The computation of CRR depends on whether the soils is clay-like, sand-like or transitional. They are defined in the bases of the index I_c (Boulanger and Idriss 2007, Robertson 2012):

(i) Clay-like soils when $I_c > 2.7$

(ii) Sand-like soils when $I_c \leq 2.5$

(iii) Transitional soil when I_c lies on the range between 2.5 andto 2.7.

CRR, for an earthquake of M=7.5, is computed using the resistance ratio ofrclean sands according to:

Clay like soils,

$$CRR_{7.5} = 0.053 \cdot Q_{tn} \cdot K_{\alpha} \qquad [16]$$

Sand-like and transitional soils like soils,

$$CRR_{7.5} = 93 \cdot \left[\frac{Q_{tn,cs}}{1000}\right]^3 + 0.08 \qquad [17]$$

where $50 \leq Q_{tn,cs} \leq 160$.

$Q_{tn,cs}$ is computed as:

Sand-like soils, $Q_{tn,cs} = K_c \cdot Q_{tn}$ [18]

where:

$K_c = 1$, for $I_c \leq 1.64$

$K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63\ I_c^2 + 33.75\ I_c - 17.88$, for $1.64 < I_c \leq 2.5$

$K_c = 1$, for $1.64 < I_c \leq 2.36$ and $F_r < 0.5$

Transitional soils, $Q_{tn,cs} = K_c \cdot Q_{tn}$ [19]

where: $K_c = 6 \cdot 10^{-7} . (I_c)^{16.76}$ [20]