DISCUSSION

Soil–foundation modelling in laterally loaded historical towers

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Contribution by G. Mesri

Pisanò *et al.* (2014) correctly state that in the seismic stability analysis of historical towers the choice of an appropriate value of undrained shear strength deserves special attention. Therefore, the 700-year aging of Modena clay underlying the Ghirlandina bell tower should not be neglected. Mesri *et al.* (1997), using the preconsolidation pressure resulting from some 500 years of post-construction aging of Pancone clay underlying the Leaning Tower of Pisa, explained why the preservation lead ingots stacked at the north rim, producing effective vertical stresses in the recompression range, did not significantly correct the lean of the tower.

Before examining the post-construction aging effect on undrained shear strength of Modena clay underlying the Ghirlandina bell tower, for reference the discussor has computed the undrained shear strength of the clay in the free field using the laboratory σ'_p and field q_c data in Fig. 5. The values of σ'_p and q_c in Table 5 were read at 5, 10, 15 and 20 m depths using a linear variation of both σ'_p and q_c with depth through the measurements. The mobilised undrained shear strength in the free field was computed using (Mesri, 1975, 1989; Terzaghi *et al.*, 1996)

$$s_{uo}(\text{mob}) = 0.22\sigma'_p \quad (14)$$

and using (Mesri, 2001)

$$s_{uo}(\text{mob}) = \frac{q_c - \sigma_{vo}}{16} \quad (15)$$

The computed values of $s_{uo}(\text{mob})$ in Table 5, according to equations (14) and (15), are quite comparable.

Under the slowly growing tower load between 1009 and 1319, following each construction load increment, Modena clay completed primary consolidation and some secondary compression, and then the clay aged for some 700 years by secondary compression and thixotropic hardening. In order to evaluate aging of the Modena clay underlying the Ghirlandina bell tower, the discussor has read the values of final effective vertical stress, σ'_{vf} , at 5, 10, 15 and 20 m depths in Fig. 7, computed $\sigma'_{vf}/\sigma'_{vo}$, and compared this to pre-construction σ'_p/σ'_{vo} in Table 6. In all depths at $x = 0$ and $x = \pm 9.2$ m (except at $z = 5$ m and $x = \pm 9.2$ m), $\sigma'_{vf}/\sigma'_{vo}$ exceeds the pre-tower construction σ'_p/σ'_{vo} , and therefore, these subsoil elements were loaded to the compression range. In all depths, at $x = \pm 27.2$ m, the values of $\sigma'_{vf}/\sigma'_{vo}$ are less than σ'_p/σ'_{vo} ; therefore, these elements of Modena clay were loaded to the recompression range. All elements have experienced some 700 years of aging by secondary compression and thixotropic hardening. For the elements in the compression range, a preconsolidation pressure by aging, σ'_c , has developed according to (Mesri, 1993)

$$\frac{\sigma'_{vc}}{\sigma'_{vf}} = \left(\frac{t}{t_p}\right)^{\frac{C_a/C_c}{1-C_r/C_c} + \beta} \quad (16)$$

where t is the duration of consolidation (in this case about 700 years), and t_p is the duration of primary consolidation under the last construction load increment. (The value t_p is expected to be quite small because during the 310-year slow construction period, secondary compression followed each construction load increment, and therefore, the last construction load increment was in the recompression range, and t_p was likely to have been less than 10 years and probably close to 1 year.) In the absence of data for Modena clay, the discussor has used $C_a/C_c = 0.034$ of the Pancon clay (Mesri *et al.*, 1997), $C_r/C_c = 0.05$, and parameter of thixotropic hardening $\beta = 0.02$. According to equation (D3), $\sigma'_{vc}/\sigma'_{vf} = 1.44$ for $t_p = 1$ year, and $\sigma'_{vc}/\sigma'_{vf} = 1.27$ for $t_p = 10$ years, leading to a representative value of the aging factor $\sigma'_{vc}/\sigma'_{vf} = 1.35$. Thus, for the values of $\sigma'_{vf}/\sigma'_{vo}$ equal or greater than pre-construction σ'_p/σ'_{vo}

$$s_u(\text{mob}) = (0.22) \frac{\sigma'_{vc}}{\sigma'_{vf}} \sigma'_{vf} \quad (17a)$$

$$s_u(\text{mob}) = (0.22)(1.35)\sigma'_{vf} \quad (17b)$$

As an example, at $x = 0$, $z = 5$ m, $\sigma'_{vf} = 600$ kPa, and according to equation (17a), $s_u(\text{mob}) = 132$ kPa, whereas equation (17b) predicts $s_u(\text{mob}) = 178$ kPa.

For elements with $\sigma'_{vf}/\sigma'_{vo} = 1$, $s_u(\text{mob}) = s_{uo}(\text{mob})$ which is obtained commonly from equation (14) (Terzaghi *et al.*, 1996). For the soil elements loaded to the recompression range, the aging effect on undrained shear strength depends on σ'_{vf}/σ'_p , and aging factor increases to 1.35 as σ'_{vf}/σ'_p approaches one. For the data in Table 6, at $x = \pm 27.2$ m, the values of σ'_{vf}/σ'_p are near 0.60 for the depth range of 5–20 m, and the aging effect on undrained shear strength is negligible.

In summary, for the Modena clay elements underlying the Ghirlandina bell tower that were loaded to the compression range beyond pre-construction σ'_p , an aging factor of 1.35 should be included in the mobilised undrained shear strength that is used in a seismic stability analysis.

Authors' reply

The interest of Professor Mesri in the authors' original paper is much appreciated, because it offers the opportunity to debate the relevant issue of time effects in geotechnical engineering.

The first aspect concerns the influence of aging on the preconsolidation stress (Mesri, 1993) and, as a consequence, on the evaluation of the mobilised, undrained, shear strength (Mesri, 1975; Jamiolkowski *et al.*, 1985). In fact, when assessing the seismic vulnerability of historical buildings, such as the Ghirlandina tower, a sound estimation of the soil's undrained strength is of paramount importance, so that the conceptual path suggested by Professor Mesri should certainly be taken into proper account.

Further, creep phenomena affect not only the soil strength,

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Table 5. Mobilised undrained shear strength of Modena clay in the free field, $s_{uo}(\text{mob})$, computed using preconsolidation pressure or cone tip resistance

Depth: m	σ'_{vo} : kPa	σ_{vo} : kPa	σ'_p : kPa	q_c : MPa	$s_{uo} = 0.22\sigma'_p$: kPa	$s_{uo} = (q_c - \sigma_v)/16$: kPa
5	50	90	170	0.7	37	38
10	100	190	240	1.0	53	51
15	150	290	310	1.3	68	63
20	200	390	400	1.7	88	82

Table 6. Consolidation pressure on Modena clay underlying Ghirlandina bell tower

Freefield				Underlying Ghirlandina bell tower					
Depth: m	σ'_{vo} : kPa	σ'_p : kPa	σ'_p/σ'_{vo}	σ'_{vf} : kPa	$\sigma'_{vf}/\sigma'_{vo}$	σ'_{vf} : kPa	$\sigma'_{vf}/\sigma'_{vo}$	σ'_{vf} : kPa	$\sigma'_{vf}/\sigma'_{vo}$
				$x = 0$		$x = \pm 9.2$ m		$x = \pm 27.2$ m	
5	50	170	3.40	600	12.0	130	2.60	100	2.00
10	100	240	2.40	610	6.10	260	2.60	145	1.45
15	150	310	2.07	580	3.87	330	2.20	195	1.30
20	200	400	2.00	510	2.55	375	2.55	250	1.25

but also the pre-failure compliance of the material. As was discussed by Nova (1982), this aspect can be readily grasped and interpreted in the framework of work-hardening elastoplasticity. The occurrence of drained creep processes can in fact be associated with the expansion of the local elastic domain, whereas the stress state remains unaltered. When substantial creep has taken place, the current stress point lies within the (expanded) yield locus, and an elastic stiffness is exhibited by the soil upon loading (until the condition for further plastifications is newly fulfilled). The time-dependent soil behaviour at the constitutive level plays an apparent role in determining the mechanical response of geo-structural systems, as was recently shown by Marchi *et al.* (2011) in the context of macro-element modelling, and experimentally corroborated by Lancellotta & Sabia (2014). In particular, the experience gained by Lancellotta & Sabia (2014) on the basis of identification techniques, clearly demonstrates the importance of soil–structure interaction in the dynamic response of the Ghirlandina tower; it was proven that the soil stiffness mobilised by environmental vibrations is actually the elastic stiffness.

Following from the above premises, the authors would like to point out how the issue raised by Professor Mesri has even broader implications, deserving further research efforts to be undertaken by the whole geotechnical community. However, it also worth remarking on the genuinely methodological focus of the paper being discussed: how to exploit finite-element results to lump the effect of inhomogeneous, undrained strength distributions into a macro-element model, which is believed to be particularly suitable for soil–structure interaction analyses.

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