Measured coefficient of consolidation field and laboratory values of very soft to soft marine Santos clay for wick drain design

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ABSTRACT

Very reliable compressibility parameters of the very soft marine clay of Santos, Brazil, have been obtained from good quality standard and special oedometer tests that can be used to estimate the total primary and secondary compressions of the clay under a containers terminal to be built close to Santos harbour. Although the geotechnical study of the site has been very thorough and has provided quite complete and reliable data about the soft layer properties, the drastic reduction of the coefficient of consolidation in the stress range around the preconsolidation stress makes it very difficult to define the representative value of the coefficient of consolidation needed to design the proper combination of “wick drains pattern/surcharge stress/preloading time” needed to bring the terminal operation settlements down to acceptable values taking into account the simultaneous occurrence of primary and secondary consolidations.

RÉSUMÉ

Des paramètres de compressibilité trés fiables de l’ argile marine très molle de Santos au Brésil, ont èté obtenus à partir d’ essais oedométriques conventionnels et spéciaux qui peuvent être utilisés pour estimer les compressions primaire et secondaire de l’ argile sous un terminal de conteneurs qui doit être construit prêt du port de Santos. Bien que l’ étude géotechnique du site ait été exhaustive et ait fourni des données bien complètes et fiables sur les propriétés de la couche molle, la réduction radicale du coefficient de consolidation dans l’ intervalle de contraintes proche de la contrainte de préconsolidation rend très difficile le choix de la valeur coreprésentative du coefficient de consolidation à utiliser pour le projet du trinôme “maille des drains verticaux/contrainte de surcharge/temps de préchargement” nécessaire pour amener les tassements du terminal pendant la phase d’ opération à des valeurs acceptables, prenant en compte l’ évolution simultanée des consolidations primaire et secondaire.

1. INTRODUCTION

Rémy et al (2010a, 2010b) report the results of good quality standard oedometer tests and special oedometer tests and the backanalysis of the measured layers compressions under a pilot embankment during a period of two years following beginning of construction, of a very soft to soft marine clay in Santos (Brazil). The backanalysis focused mainly on the measured compressions of the most critical soft clay layer under the three embankment areas, area 1 with wick drains on a 1.2 x 1.2m square mesh, area 2 with wick drains on a 2.4 x 2.4m square mesh and area 3 with no wick drains, and was based on the working hypothesis that secondary consolidation occurs simultaneously with primary consolidation.

The soil profiles under area 1 (with 1.2m x 1.2m wickdrains) and under area 3 (without wickdrains) are shown in figure 1. The most critical layer is layer number 6 described as silty clay, slightly sandy.

2. COMPRESSIBILITY PARAMETERS OF THE SOFT CLAY LAYER

The results of the standard oedometer tests are illustrated in figure 2 which shows the strain (ε) plotted against the logarithm of vertical effective stress (log σ'_v) for eight standard oedometer tests run on sample (6) of boring SRA-203 which position, in area 3, can be seen in figure

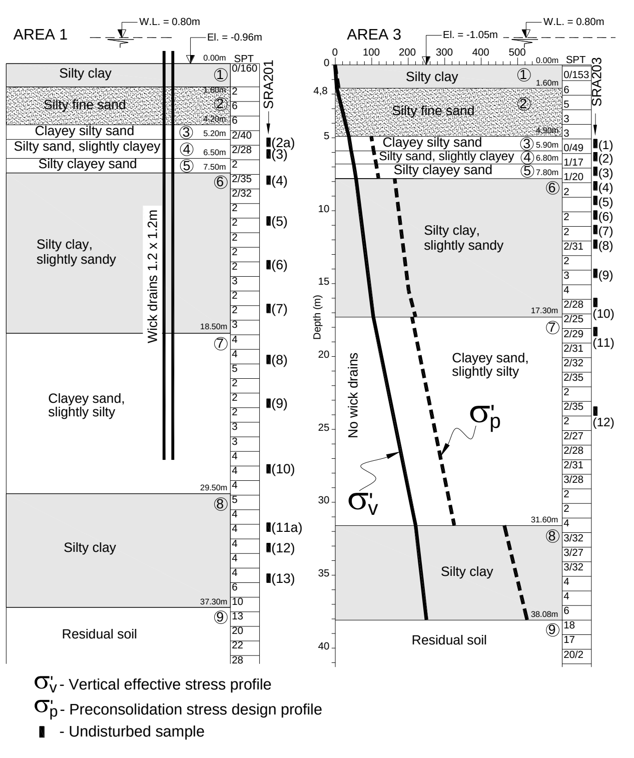

Figure 1. Soil profiles under area 1 (with wickdrains) and area 3 (without wickdrains)

1. One of the tests was run on a remoulded sample and its results are very different from the results of the tests run on the seven undisturbed clay samples. From similar tests performed on all the undisturbed samples of boring SRA203, the profile of preconsolidation stress in area 3 which is shown in figure 1 was established.

The other properties of layer 6 obtained from the standard oedometers tests are γ = 15,0kN/m³, $C_c/(1+e_0)$ = 0,56 and C_r/C_c = 0,11.

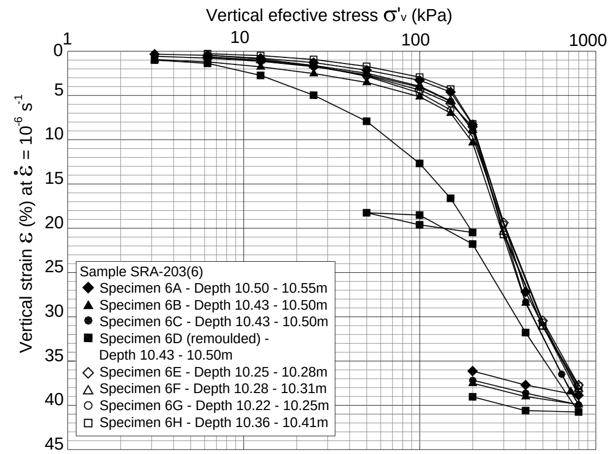

Figure 2. Results of oedometer tests in sample 6 of boring SRA203

Long term oedometer tests with up to 5 years duration performed by the COPPE rheology group during the last 20 years (Martins 2005) have shown that secondary compression is finite and ends after reaching a state equivalent to a constant OCR value for a given clay. Andrade (2009), following the procedure suggested by Feijó and Martins (1993), established that the end of secondary compression line for the Santos clay at the site of the terminal could be drawn as being the line equivalent to an OCR value of 2.1.

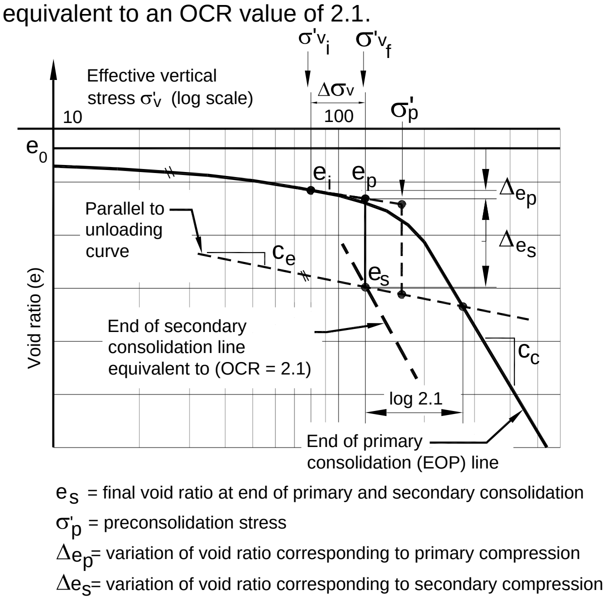

Figure 3. Primary compression and secondary compression

Based on these results Rémy et al (2010a, 2010b) presented the model used for their estimate of primary and secondary compression, illustrated in figure 3 (only for the case when the loading of the clay is all within the recompression range), and concluded *"High quality standard and special oedometer tests, such as the ones performed and presented by the authors, do achieve to provide trustworthy values of all primary and secondary compression … of the tested soft clay layer."*

3. CONSOLIDATION PARAMETERS OF THE SOFT CLAY LAYER

Rémy et al (2010a, 2010b), based on the values of the coefficient of consolidation (laboratory c_v) which plot against logarithm of vertical effective stress (log σ'_v) for all samples tested as illustrated in figure 4 by the results of the same eight tests of sample SRA203-6 already used in figure 2, also mentioned that *"High quality standard and special oedometer tests do achieve to provide trustworthy values of … consolidation parameters of the tested soft clay layer".*

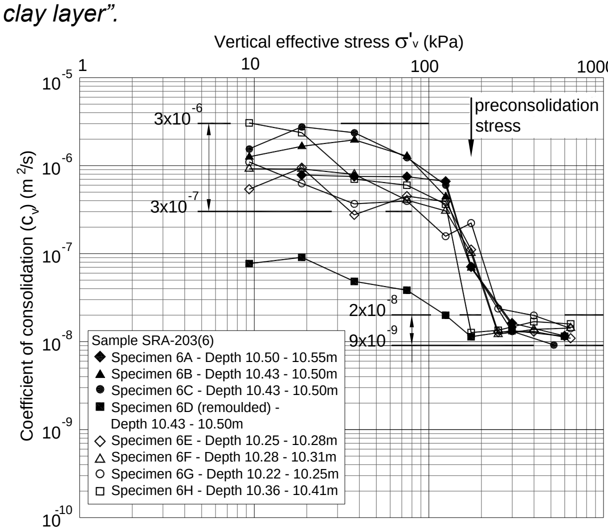

Figure 4. Results of oedometer tests in sample 6 of boring SRA203

As can be seen in figure 4, there is a the drastic reduction of the coefficient of consolidation in the stress range around the preconsolidation stress.

To account for the simultaneous occurrence of primary and secondary consolidations in their backanalysis of the settlements of the test embankment, Rémy et al (2010b) made direct use of Taylor and Merchant's theory for the calculations of the evolution of the layer compression with time in the area with vertical drainage, that is the one with no wickdrains, and made use of a procedure which they devised and named *"primary Barron + pseudo secondary Taylor and Merchant procedure"* for the calculations of the evolution of the layer compression with time in the areas with radial drainage, that is the ones with wickdrains. All wickdrains were 5mm x 100mm (r_w = 3.25cm), and two possible situations were considered for the smear zone, one with a radius r_s equal to 6.5cm and a second one with a radius r_s

equal to 19.5cm, and, in both cases, a permeability coefficient 5 times smaller than the one of the undisturbed soil was assumed.

Figure 5 which shows the backanalyzed values of field c_v (and c_h with c_h considered to be equal to c_v) versus mean effective vertical stress in the soft clay layer led Rémy et al (2010b) to conclude that the range of field values of the coefficient of consolidation obtained from their backanalysis agrees remarkably well with the range of the laboratory values and that, as expected, the transition of higher values of c_v (or c_h) in the recompression range to lower values of c_v (or c_h) in the virgin compression range occurs for values of the effective vertical stresses which are smaller in the field than in the laboratory. As shown in figure 5, the field c_v value is about 8 times smaller than the c_v lab value for σ'_v = 120kPa and gets to be about 7 times smaller for σ'_v = 180kPa. One peculiarity of the results shown in figure 5 is that the backanalyzed c_v values under the embankment without vertical drain drop much faster than the lab values and than the c_h values under the embankment with vertical drains. Rémy et al (2010b) comment as follows: *"Close to the top and to the bottom of layer 6 under area 3, the local consolidation ratio is above 80% with σ'_v close to the "pseudo final effective stress σ_v - u_{hid}". At this stress level the c_v value is close to the small virgin compression range values. The very fast decrease of the c_v value under area 3 is, therefore, believed to be due to this "sealing" effect of the top and bottom sublayers".*

A simple look at figure 5 shows that the drastic reduction of the coefficient of consolidation in the stress range around the preconsolidation stress and the marked difference between field values and lab values make it very difficult to define the representative value of the coefficient of consolidation needed to design the proper combination of "wick drains pattern/surcharge stress/preloading time" needed to bring the terminal operation settlements down to acceptable values.

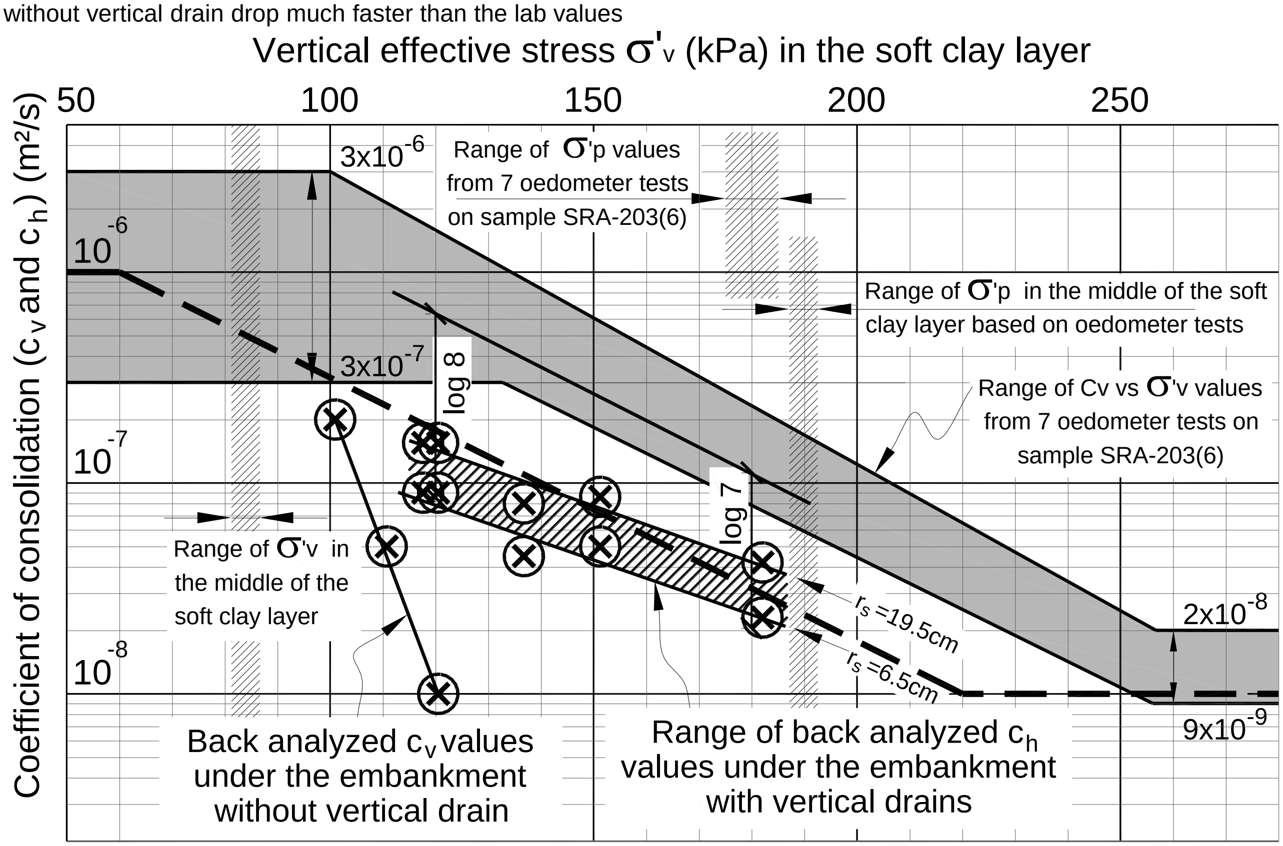

Figure 5. Backanalyzed values of field c_v and c_h versus mean effective vertical stress in the soft clay layer.

4. CONSIDERATIONS ABOUT THE FINAL DESIGN PREVISIONS OF TOTAL SETTLEMENTS EVOLUTION WITH TIME

The pilot embankment was built in the area where the fill thickness was expected to be maximum in the whole area of the Terminal. The average ground level under the area of the terminal embankment stands at elevation +0,50m. At the final design stage, settlements will have to be calculated individually at all borings existing in the area. Extrapolating the geotechnical model to each boring location, where the soil profile is identical to the one shown in figure 1, but with both ground level and water level, at elevation +0,50m shall be taken as an example to illustrate the challenge to be faced in the final design.

Total settlement under the embankment built up to elevation +6,00m calculated as defined in figure 3 would be 3,81m with 1,30m total primary settlement and 2,51m total secondary settlement. Total compression of layer 6 would amount to 1,93m with 0,24m of primary compression and 1,69m of secondary compression.

Calculating the evolution of the above mentioned compression of layer 6 with time through the same *"primary Barron + pseudo secondary Taylor and Merchant procedure"* used by Remy et al (2010b), for a soft clay layer treated with wickdrains on a triangular pattern with 1.0m spacing, and considering the load to be applied instantaneously, the following curves are obtained for various c_h values within the possible range between $c_h = 10^{-6}$m²/s and $c_h = 10^{-8}$m²/s.

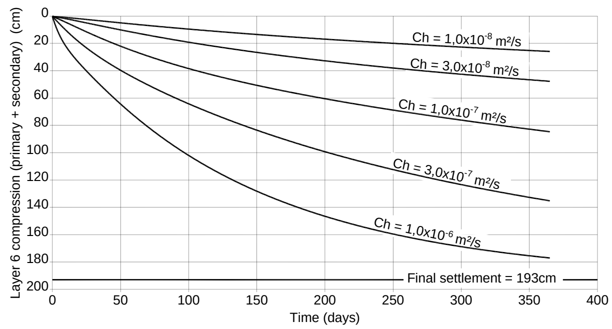

Figure 6. Evolution of primary + secondary compression of layer 6 with time.

It is quite clear, just as would be expected, that the percentage of layer 6 compression reached after 360 days of preloading varies drastically with the assumed c_h value as shown in table 1.

Table 1. % of layer 6 compression reached after 360 days

c_v value	% of layer 6 compression reached after 360 days
1×10^{-6} m²/s	91.7%
3×10^{-7} m²/s	70.1%
1×10^{-7} m²/s	43.9%
3×10^{-8} m²/s	24.7%
1×10^{-8} m²/s	13.4%

These results clearly show that the remaining settlements after preloading, vary widely even for a change in c_h as small as one to three. This means that the proper representative c_h value has to be rather closely assessed for a correct final design, and to avoid big surprises during preloading.

5. A POSSIBLE PATTERN OF THE VARIATION OF THE COEFFICIENT OF CONSOLIDATION WITH VERTICAL EFFECTIVE STRESS FOR DESIGN

Based on figure 3, it might, perhaps, be easier to use a representative law of variation of the coefficient of consolidation with vertical effective stress than to assess a correct representative mean value of c_h.

One possible pattern of the variation of the coefficient of consolidation with vertical effective stress could be such as the one illustrated in figure 7. This pattern considers that c_h in the field is equal to 10^{-6} m²/s for all values of σ'_v up to σ'_p - 120kPa, that c_h in the field is equal to 10^{-8} m²/s for all values of σ'_v higher than σ'_p + 40kPa and that log c_h decreases linearly with σ'_v when σ'_v increases from σ'_p - 120kPa up to σ'_p + 40kPa, as illustrated by the dashed line in figure 5 in order to compare this pattern with the measured lab and field values.

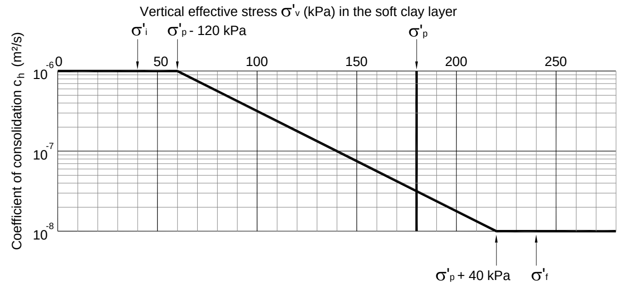

Figure 7. Assumed variation pattern of log c_h (field value) versus vertical effective stress σ'_v

The mathematical formulation for Taylor and Merchant's theory (Carvalho 1997) supposes that the c_v value be constant. For this reason any calculation of evolution of layer compression with time should be done in steps, taking the suitable value of c_v for the range σ'_v covered by each calculation step, in the same way as done by Rémy et al (2010b) for the backanalysis. Of the compression of layer 6.

6. SOME CONSIDERATIONS ABOUT THE EVOLUTION OF PRIMARY COMPRESSION WITH A COEFFICIENT OF CONSOLIDATION VARYING WITH VERTICAL EFFECTIVE STRESS

To analyze the influence of the variation of the c_v value with σ'_v according to the chart shown in figure 7, a situation where the increase of vertical effective stress covers the whole stress range where c_v decreases is assumed. This situation considers that the initial stress in the middle of the layer (σ'_i) is equal to σ'_p - 140kPa and that the final stress (σ'_f) is equal to σ'_p + 60kPa, as illustrated in figure 7.

Figure 8 shows the variation of mean primary consolidation ratio $\overline{U}$ with time for five c_v values constant throughout the effective stress range, as calculated through Terzaghi's one dimensional consolidation theory for layer 6 with 9,5m thickness with pure vertical drainage, that is without wickdains, drained both on top and at bottom. Figure 8 also shows a sixth curve, calculated through finite differences with the c_v value recalculated at each time step, in function of σ'_v according to the chart of figure 7. Figure 8 clearly shows that with c_v varying, in fact decreasing, as consolidation takes place, the process of consolidation is obviously much different than for c_v constant and as can be seen, consolidation rapidly slows down with time. Calculating the mean c_v value which would lead to the same beginning and ending point of the curve for c_v varying with σ'_v for each interval of 10% $\overline{U}$, the mean c_v values shown in table 2 can be obtained.

CONCLUSIONS

Furthering the following conclusions presented by Rémy et al (2009b):

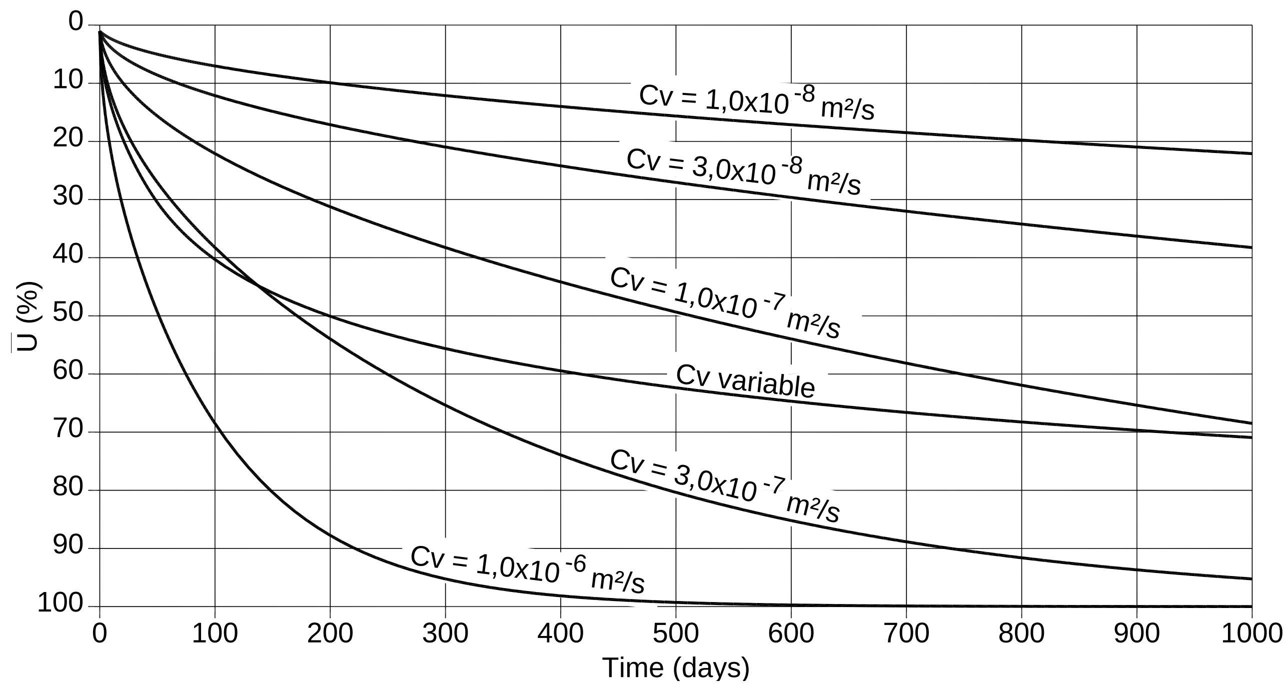

Figure 8. Variation of mean primary consolidation ratio $\overline{U}$ with time for various values of c_v

Table 2. Variation of the coefficient of consolidation ratio with mean consolidation ratio

$\overline{U}$ range	Mean c_v value
0 to 10%	4.2×10^{-7} m²/s
10 to 20%	3.9×10^{-7} m²/s
20 to 30%	3.5×10^{-7} m²/s
30 to 40%	2.9×10^{-7} m²/s
40 to 50%	1.9×10^{-7} m²/s
50 to 60%	1.1×10^{-7} m²/s
60 to 70%	0.6×10^{-7} m²/s

- "The literature versing on the field behaviour of soft clay taking into account the undeniable simultaneous occurrence of primary and secondary compressions is very abundant. This is much more so than when Leroueil et al (1985) wrote that "this abundant literature has modified neither the common practice based on the Terzaghi theory nor the way of thinking on clay behaviour". However no practical tool has emerged from all this literature which can readily be used in common practice. This lack of tool has been, in the authors' understanding, the main hindrance to changes in common practice. This situation has much to do with the very little communication and lack of joint endeavour between practicing engineers and researchers".

- "In the lack of better practical available methods, the Taylor and Merchant's formulation allows for reasonably reliable modeling of soft clay field behavior, taking into account the simultaneous occurrence of primary and secondary consolidations".

- "Trial embankments back-analysis have repeatedly provided erroneous values of primary compressibility and consolidation parameters for being performed through Asaoka's method which does not take into account the simultaneous occurrence of primary and secondary consolidations"

The writers conclude from the considerations presented above that:

- even in the case presented in which the geotechnical study of the site has been very thorough and has provided quite complete and reliable data about the soft layer properties, the drastic reduction of the coefficient of consolidation in the stress range around the preconsolidation stress makes it very difficult to define the correct coefficient of consolidation value needed to design the proper combination of "wick drains pattern/surcharge stress/preloading time" needed to bring the terminal operation settlements down to acceptable values taking into account the simultaneous occurrence of primary and secondary consolidations,

- the fact that *"the literature has modified neither the common practice based on the Terzaghi theory nor the way of thinking on clay behaviour"* has led to the present situation when the practicing engineer still finds little useful help from the technical literature to meet his challenge when trying to face the reality of primary and secondary compression occurring simultaneously with a coefficient of consolidation which varies throughout the effective stress range to which the soft layer is to be submitted.

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References

Andrade, M. E. S. 2009 “Contribuição ao estudo das argilas moles da cidade de Santos” *M.Sc. Thesis, COPPE/UFRJ, Rio de Janeiro, RJ, Brazil.* (in Portuguese).

Carvalho, S. R. L. 1997 “Uma teoria de adensamento com compressão secundária” *Ph D thesis, COPPE-UFRJ, Rio de Janeiro, RJ, Brasil (in portuguese).*

Feijó, R. L. and Martins, I. S. M. 1993 “Relação entre a compressão secundária, OCR e K_0” COPPEGEO 93, In: *Simpósio Geotécnico Comemorativo dos 30 anos da COPPE/UFRJ, Rio de Janeiro, RJ, Brazil, pp. 27-40 (in Portuguese).*

Ladd, C. C. 2008 “Commentary: Soft ground geotechnics”, *Geo-strata, march/april, pp. 10-11.*

Martins, I. S. M. 2005 “Algumas considerações sobre adensamento secundário”*, Conference given at Clube de Engenharia, Rio de Janeiro, Brazil (Personal communication, in Portuguese).*

Massad, F. 2009 “Solos marinhos da baixada santista, características e propriedades geotécnicas” *São Paulo: Oficina de Textos.*

Rémy, J.P.P.; Martins, I.S.M; Santa Maria, P.E.L.; Aguiar, V.N. and Andrade, M.E.S. (2010a) The Embraport pilot embankment – primary and secondary consolidations of Santos soft clay with and without wick drains – Part 1 In: *Symposium on New Techniques for Design and Construction in Soft Clays,* Guaruja, SP, Brazil 289-309.

Rémy, J.P.P.; Martins, I.S.M; Santa Maria, P.E.L.; Aguiar, V.N. and Andrade, M.E.S. (2010b) The Embraport pilot embankment – primary and secondary consolidations of Santos soft clay with and without wick drains – Part 2 In: *Symposium on New Techniques for Design and Construction in Soft Clays,* Guaruja, SP, Brazil 311-330.