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## PVD-assisted consolidation of dredged sediments in a CDF: design of the test field

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### ABSTRACT

A confined disposal facility (CDF) was built at the Ancona Harbour (Italy) to host contaminated dredged sediments. The Port Authority intended to get the chance to use the reclaimed land as a storage area. Since dredged sediments typically have very high void ratios when disposed in a CDF, their consolidation is essential to achieve proper mechanical characteristics. Consolidation of sediments by prefabricated vertical drains (PVDs) and embankment preloading is being planned. Since large strains are expected, nonlinear constitutive relations for compressibility and hydraulic conductivity are required to model consolidation processes. For this purpose, sediments were characterised by laboratory tests to determine compressibility and permeability laws, followed by field investigations (CPTUs and dissipation tests). A full scale test field has been designed to calibrate modelling and set up the operating procedures. The paper illustrates and discusses the design criteria of the test field.

**Keywords:** dredged sediments, confined disposal facility, vertical drains, preloading, test field

### 1 INTRODUCTION

Continuous dredging works in ports and harbours are necessary to keep open access ways and carry out maritime operations in safe conditions. Huge amounts of sediments, sometimes contaminated, have to be frequently dredged.

Contaminated dredged sediments require proper and sustainable managing options. Their disposal in waste landfills is neither an environmentally friendly solution nor a cost effective one, given transport issues and volumes needed in suitable landfills. Among the options allowed by the Italian Regulation (DM 173/2016), their disposal in a confined disposal facility (CDF) is certainly a sustainable solution provided that the reclaimed land will be used. Given the typical high void ratios of dredged sediments, their consolidation is essential in order to achieve proper mechanical characteristics (shear strength and compressibility) for the reuse of the area.

The Central Adriatic Sea Port Authority (Ancona, Italy) combined the need of building a CDF to host both uncontaminated and contaminated dredged sediments and the need of land reclaiming to be used for storage of containers. The CDF has a surface area of about 9.5 ha and an overall capacity of almost 300,000 m<sup>3</sup>. It was completed in 2015 and it is currently being filled. The scheduling of sediments disposal had to take into account that the disposal would be staggered over

several months or years, therefore a partition in sectors of the CDF was planned for managing the filling steps and the subsequent sediment consolidation (Felici et al., 2019). Specific operating procedures have been planned for sediment disposal to rapidly consolidate them soon after filling each sector.

The first sector is currently completed, and a test field has been designed for full scale analysis and modelling calibration in order to optimize the consolidation technique to be applied to all the CDF sectors.

Given the fine grained nature of the sediments and the availability of coarse material at the CDF site, the improvement technique by preloading combined with vertical drains was selected (e.g. Stark et al., 2018; Müthing et al., 2018). The same material used for the preloading embankment in the first sector is planned to be reused to consolidate the sediments in the following sectors, as a “moving bank”.

Since large strains are expected, nonlinear constitutive relations for compressibility and hydraulic conductivity are required to model the consolidation processes (Gibson et al., 1981; de Lillis and Miliziano, 2016). Characterization by laboratory tests on the sediments allowed to determine compressibility and permeability laws for the dredged sediments (Felici et al., 2019). The results of subsequent field investigations by cone penetration tests and dissipation tests were used to model the consolidation process.

After a brief description of the CDF characteristics,

the paper illustrates and discusses the results of the geotechnical investigation (laboratory and in situ tests) to characterize the sediments of the first sector and the design criteria of the test field.

## 2 THE CONFINED DISPOSAL FACILITY AND TEST FIELD SECTOR

The CDF is located in the commercial dock of the Ancona Harbour. Its internal area (9.5 ha) includes sediments disposed soon after its construction, emerging 1 m above the sea level. The current mean water depth in the remaining part (4.5 ha) is 4 m, with a maximum of about 7 m.

The CDF was built by steel sheet piling, 20 m long, through a 6 m thick dense sandy layer to the underlying clayey layer (thickness > 12 m) that performs as the natural impervious base. The sheet pile interlocks were sealed by water expanding epoxy resin to ensure the tightness required by the Italian law (L.84/1994), that is, hydraulic conductivity,  $k$ , equivalent to 1 m of soil with  $k \leq 10^{-9}$  m/s. The hydraulic performance of the sheet walls was assured by special tests and quality controls during construction (Pasqualini et al., 2014).

The first sector was created in adjacency to one of the CDF walls by special geotextile bags (geo-tubes) filled with the sediments themselves, to avoid loss of the CDF capacity and create an effective sidewall confinement for the subsequent consolidation process. Placement and filling procedures of the geo-tubes were set up by a trial field in the CDF (Felici et al., 2019). This first sector (roughly rectangular, 50 m x 70 m) is confined at the open side by the geo-tubes (50 m long), directly laid and hydraulically filled with the sediments already placed in the CDF, by a submersible pump through the top openings of the geo-tubes.

The first sector of the CDF, that serves as test field too, was filled with dredged sediments from the Fano Harbor (70 km North of Ancona) and from two areas of the Ancona Harbor (“Isa-Palumbo” and “Fincantieri”).

## 3 LABORATORY CHARACTERIZATION OF THE SEDIMENTS

### 3.1 Classification

Table 1 shows the main physical and classification parameters of the three lots of sediments disposed into the first sector of the CDF. Both “Fincantieri” and “Fano” sediments essentially consist of fine fraction, of medium plasticity, whereas the “Isa-Palumbo” sediment resulted a silty sand.

### 3.2 Compressibility and consolidation parameters

Incremental load oedometric tests (ASTM D2435) on reconstituted samples (high void ratio), saturated with sea water, were carried out to study the sediment compressibility and consolidation. The applied effective vertical pressures ( $\sigma'_v$ ) ranged from 6.25 kPa to 400 kPa.

The mean values of the parameters relevant to the

design pressure range are listed in Table 2 for each of the sediments ( $c_v$  = vertical consolidation coefficient;  $c_c$  = compression index;  $E_{oed}$  = oedometric modulus).

The vertical hydraulic conductivity values,  $k_v$ , determined from the oedometric tests by the Terzaghi one-dimensional consolidation theory, were compared with those measured by consolidometer tests (ASTM D5856), in order to evaluate their reliability and possible use to design the consolidation technique without  $k$  testing. A detailed discussion of this comparison is given in Felici et al. (2019). It is worth highlighting that for void ratios,  $e$ , ranging from 1.3 to 2.0, a good agreement between the results of the two tests was found for the “Fincantieri” sediments, with  $k_v$  values from  $4 \times 10^{-10}$  m/s to  $1 \times 10^{-9}$  m/s, increasing with  $e$ .

Table 1. Classification and characterisation of the sediments disposed in the first sector of the CDF.

	Fincantieri	Fano	Isa-Palumbo
Gravel (%)	1-2	0-1	8-15
Sand (%)	6-7	0-3	51-59
Silt (%)	55-57	60-66	15-20
Clay (%)	36-38	30-35	5-11
Liquid limit (%)	44-50	53-58	20-26
Plasticity index (%)	18-22	25-28	n.p.*
USCS	ML-CL	MH	SM
Organic content (%)	4	6	0
Specific gravity (-)	2.67	2.72	2.61

\*non-plastic

ML: silt; CL: clay of low plasticity; MH: silt of high plasticity; SM: silty sand.

Table 2. Parameters from incremental load oedometric tests.

$\sigma'_v$ (kPa)	$E_{oed}$ (MPa)	$e$ (-)	$c_v$ (m <sup>2</sup> /s)	$k_v$ (m/s)	$c_c$ (-)
“Fincantieri”					
12.5	0.22	1.89	$2.4 \times 10^{-8}$	$3.1 \times 10^{-9}$	0.68
25	0.58	1.81	$4.4 \times 10^{-8}$	$1.0 \times 10^{-9}$	0.29
50	0.68	1.64	$3.1 \times 10^{-8}$	$5.1 \times 10^{-10}$	0.57
100	1.62	1.52	$3.9 \times 10^{-8}$	$2.9 \times 10^{-10}$	0.44
“Fano”					
12.5	0.20	1.60	$1.7 \times 10^{-8}$	$1.0 \times 10^{-9}$	0.23
25	0.26	1.50	$1.6 \times 10^{-8}$	$6.9 \times 10^{-9}$	0.34
50	0.40	1.36	$1.8 \times 10^{-8}$	$5.5 \times 10^{-10}$	0.46
100	0.87	1.23	$2.3 \times 10^{-8}$	$3.2 \times 10^{-10}$	0.43
“Isa-Palumbo”					
12.5	0.59	0.77	$2.1 \times 10^{-8}$	$4.8 \times 10^{-10}$	0.15
25	1.11	0.71	$1.8 \times 10^{-7}$	$5.8 \times 10^{-10}$	0.11
50	3.06	0.71	$9.6 \times 10^{-8}$	$3.3 \times 10^{-10}$	0.13
100	2.99	0.67	$1.4 \times 10^{-7}$	$3.8 \times 10^{-10}$	0.14

## 4 DESIGN OF THE TEST FIELD

### 4.1 General description

The improvement technique by preloading and vertical drains was selected to achieve consolidation settlements in a time of few months.

A full scale test field has been planned both to set up the construction procedures and to calibrate the consolidation modeling, after installing a monitoring

system. The design of the test field will allow for a proper selection and location of the monitoring tools.

A layer of coarse material, about 0.7 m thick, has already been placed above the sediments to allow the construction vehicles to operate and to act as a top drainage for the vertical drains. The bottom drainage for the sediment layer is the natural sandy layer under it.

Prefabricated vertical drains (PVDs) were chosen because of their easy availability and of using light vehicles for their installation. In particular, Colbondrain®, of width,  $a = 0.1$  m and thickness,  $b = 0.005$  m, will be used in the test field. They are made by a polyethylene core on which a needle-punched polypropylene geotextile filter is pasted.

The choice of PVDs is compatible with the time required for sediment consolidation (few months) considering that the drain efficiency tends to decrease with time, particularly in the marine environment and in presence of chemical and microbiological contamination (Rollin and Lombard, 1988), as in the case of concern.

The preloading embankment will be built with coarse material (available at the site) on a square area of 30 m side (Figures 1 and 2). For preventing instability, the embankment will be gradually built, and the final load will be reached in three steps applied in an overall time of at least 3 weeks. A schematic section of the test field is given in Figure 1.

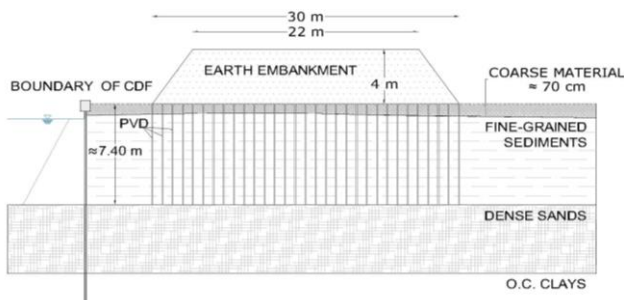


Fig. 1. Schematic cross-section of the test field.

#### 4.2 Parameters from laboratory tests

To evaluate the expected consolidation time and settlements, laboratory parameters related to the sediments “Fincantieri” and “Fano” have been considered since they are about 85% of the volume of the sector. It is reasonable assuming that the small amount of the “Isa-Palumbo” sandy sediments would not significantly affect the compressibility and consolidation process when considering the entire sediment layer.

The values of the design parameters for the test field have been calculated as the mean values of the “Fincantieri” and “Fano” sediments (Table 2) related to the pressure range (i.e. from 25 kPa to 100 kPa) that includes the initial and final vertical pressure levels in the middle of the sediment layer.

In order to take into account the scale effect (laboratory vs site), the coefficient of vertical consolidation from oedometer tests was amplified by a

factor of 5, as suggested in the literature (Burghignoli and Calabresi, 1975; Robertson et al., 1992; de Lillis and Miliziano, 2016).

The geostatic vertical effective pressure has been calculated on the basis of the stratigraphy resulted from the in situ tests (§ 4.3), with hydrostatic pressure starting from the top of the sediment layer (-0.7 m from the ground level) and volume weight of the coarse material,  $\gamma_{cm} = 17$  kN/m<sup>3</sup>. All the laboratory parameters assumed for calculation are listed in Table 3.

Table 3. Parameters for test field design.

Parameter	Value
Sediment layer thickness, $H_0$ [m]	6.7
Volume weight of the top layer, $\gamma_{cm}$ [kN/m <sup>3</sup> ]	17
Volume weight of the sediments, $\gamma_{sed}$ [kN/m <sup>3</sup> ]	16.5
Volume weight of sea water, $\gamma_w$ [kN/m <sup>3</sup> ]	10.1
Initial void ratio, $e_0$ [-]	1.65
Geostatic vertical effective stress, $\sigma'_{v,0}$ [kPa]	33.5
Compression index, $c_c$ [-]	0.42
Oedometer modulus, $E_{oed}$ [kPa]	730
Coefficient of vertical consolidation, $c_{v,lab}$ [m <sup>2</sup> /s]	$2.8 \times 10^{-8}$
Amplificative scale factor for $c_{v,lab}$	5
Assumed $c_{v,lab}$ [m <sup>2</sup> /s]	$1.4 \times 10^{-7}$
$c_h$ [m <sup>2</sup> /s]	$1.9 \times 10^{-7}$

#### 4.3 Parameters from in situ tests

In situ characterization of the test area was performed with the purposes to detect the thickness of the sediment layer, to verify its homogeneity, to measure the horizontal consolidation coefficient,  $c_h$ , and to get reference values before consolidation, in order to quantify the preloading effectiveness on the basis the same final investigations.

The in situ investigation (Figure 2) consisted in 6 piezocone tests (CPTUs) and 8 mechanical cone penetration tests (CPTs), all performed at the standard rate (2 cm/s). CPTs were distributed across the footprint of the embankment to investigate the sediment mechanical characteristics and stratigraphy. CPTUs were located in the central area of the test field to estimate hydraulic parameters (for modeling) where future consolidation process will not be much affected by boundary effects.

Since the stratigraphy of the seabed was known from previous investigations (§2), the maximum depth of CPTs was just below the sediment layer, in the dense sands, easily detected on the basis of the measured values of the tip resistance,  $q_c$ . Pre-holes of depth of about 0.7 m were necessary to go across the coarse material placed on the surface of the sediment layer.

The sandy layer was found at a depth ranging from a minimum of 6.5 m to 7.6 m (Figure 3). The average thickness of the sediments layer in the test field area,  $H_0$ , resulted equal to 6.7 m.



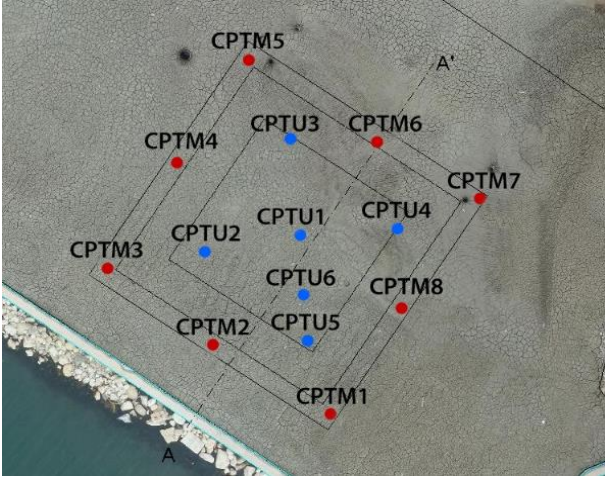


Fig. 2. Preload embankment print and in situ investigations: CPTUs and mechanical CPTs (CPTM).

As expected, the measured tip resistance through the sediment layer resulted to be very low: 0.4 MPa on average, with values of  $q_c$  locally lower than 0.1 MPa (Figures 3 and 4). In 4 out of the 6 CPTUs, 2 lenses with a higher tip resistance were detected, one of them close to the upper boundary of the sediment layer (1.1 m thick, average  $q_c = 1$  MPa) and the other one in the middle of the layer (thickness of about 0.8 m,  $q_c = 3.2$  MPa). The  $q_c$  log of the CPTU.1 (close to the center of the test area) is shown in Figure 4, in which the lens is evident in the middle of the sediment layer.

The CPTUs were interpreted by the unified approach proposed by Robertson and Wride (1998) on the basis of the measured values of  $q_c$ , sleeve resistance,  $f_s$ , and excess pore pressure,  $u_2$ . The sediment layer resulted to be a clayey silt or silty clay (with small lenses of peat), according to the laboratory classification. The two lenses resulted to be sandy (silty sand or sandy silt) as well shown by the values of the excess pore pressure that tend to decrease in correspondence with each lens (Figure 3).

Dissipation tests have been performed in CPTU.1 and CPTU.6 (Figure 2), at depths of 5.1 m and 2.8 m, respectively, to estimate  $c_h$  using the equation proposed by Teh and Houlsby (1991):

$$c_h = \frac{T_{50}^* r^2 I_r^{0.5}}{t_{50}} \quad (1)$$

where:  $T_{50}^*$  is the modified time factor, for a given probe geometry and porous element location, at 50% of consolidation (for  $u_2$  pressure,  $T_{50}^* = 0.245$ );  $r$  is the radius of the piezocone tip ( $r = 1.785$  cm);  $I_r$  is the rigidity index,  $I_r = G/c_u$ , with  $G$  = shear modulus and  $c_u$  = undrained shear strength;  $t_{50}$  is the measured time at 50% of the initial excess pore pressure.  $I_r$  has been estimated equal to 130 on the basis of the study by Keaveny and Mitchell (1986), assuming normally consolidated sediments with a plasticity index,  $PI = 25$ .

From the dissipation test of CPTU.1 it was found  $t_{50} = 5400$  s and  $c_h = 1.6 \times 10^{-7}$  m<sup>2</sup>/s; from the CPTU.6:

$t_{50} = 4200$  s and  $c_h = 2.1 \times 10^{-7}$  m<sup>2</sup>/s. The resulted values of the coefficient of horizontal consolidation are very similar and higher than the  $c_v$  values from laboratory tests (Table 2). The average value of the  $c_h$  coefficient has been used for design (Table 3).

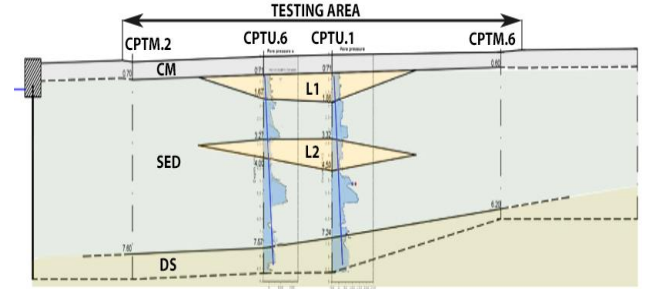


Fig. 3. Section A-A' with  $u$  log (in light blue) of CPTU tests (CM = coarse material; L1 = Lens 1; L2= Lens 2; SED = Sediments; DS = Dense Sand).

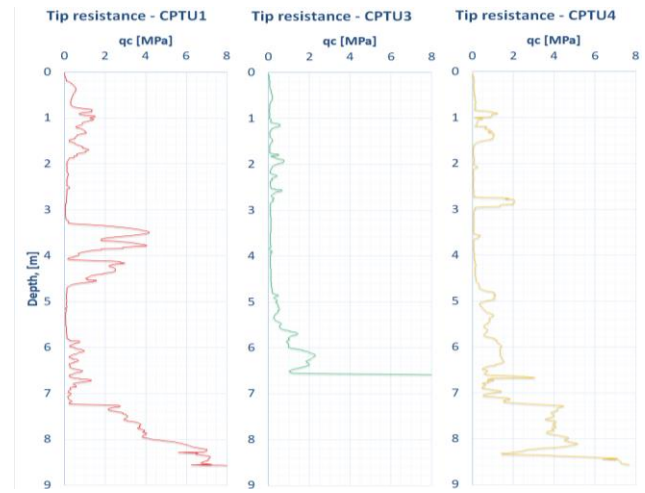


Fig. 4. Tip resistance of CPTU.1 (in the centre of the test area), CPT.3 (at a corner) and CPTU.4.

#### 4.4 Embankment and drains design

The service load for the foreseen container storage area of the CDF is 47 kPa. Starting from laboratory results, the expected consolidation settlement,  $\delta_E$ , was estimated considering a one-dimensional deformation:

$$\delta_E = \frac{H_0}{1 + e_0} c_c \log \left( \frac{\sigma'_{v,0} + \Delta q_E}{\sigma'_{v,0}} \right) \quad (2)$$

where:  $c_c$  is the average compression index;  $\sigma'_{v,0}$  is the geostatic effective vertical pressure and  $e_0$  is the void ratio, both at the middle of the layer;  $\Delta q_E$  is the service load;  $H_0$  is the compressible layer thickness.

The variables considered for the test field design are the pressure to be applied by the embankment ( $\Delta q_R$ ), its residence time ( $t_R$ ) and the spacing of the drain mesh ( $S$ ).

We proceeded by establishing the overburden pressure to be applied and estimating the residence time by means of the theory of consolidation with vertical drains under time-depended loading conditions (Tang

and Onitsuka, 2000), for different spacing of the PVD.

The embankment was designed as a truncated pyramid with a square base of 30 m side and a height of 4 m. The side slopes are 45°, therefore the square upper surface is of 22 m side. This geometry allows to assume that the sediment layer below most of the embankment is in conditions of prevented lateral deformation. Due to the impossibility to use a compactor, a volume weight of the embankment,  $\gamma_R = 16.5 \text{ kN/m}^3$  has been considered. Therefore, the applied pressure is  $\Delta q_R = 66 \text{ kPa}$ .

Equation (2) can be used to estimate the final consolidation settlement due to the embankment,  $\delta_R$ , by replacing the service load,  $\Delta q_E$ , with the embankment load,  $\Delta q_R$ . The overall average degree of consolidation,  $U_{ref}$ , to be obtained by preloading was set equal to the ratio between the settlements  $\delta_E$  and  $\delta_R$ :

$$U_{ref} = \frac{\delta_E}{\delta_R} \quad (3)$$

The consolidation process has been modeled by the theory for vertical drains by Tang and Onitsuka (2000), that combines radial and vertical flow (in our case with double pervious boundaries) under time-dependent loading, in equal strains conditions. The pressure applied by the embankment versus time,  $q(t)$ , is shown in Figure 5 (the embankment is planned to be built by three lifts, each of them constructed in 2 days waiting a week before construction of the subsequent one).

According to the theory by Tang and Onitsuka (2000) the average degree of consolidation of the sediment layer with time,  $U(t)$ , for the whole layer can be evaluated as:

$$U(t) = 1 - \frac{1}{q_u H} \int_0^H u dz - \int_0^t \frac{dq}{dt} \sum_{m=0}^{\infty} \frac{2}{M^2} e^{-\beta_m(t-\tau)} d\tau \quad (4)$$

where:  $q_u$  is the final loading ( $= \Delta q_r$ );  $q$  is the time dependent loading,  $H$  is the half of the thickness of the compressible layer;  $\tau$  is the time of application of any load;  $u(z,t)$  is the average excess pore water pressure at a given depth,  $z$ . It is expressed as:

$$u(z,t) = \int_0^t \frac{dq}{dt} \sum_{m=0}^{\infty} \frac{2}{M^2} \sin \frac{Mz}{H} e^{-\beta_m(t-\tau)} d\tau \quad (5)$$

where:

$$\beta_m = \beta_m + \beta_{zm} \quad (6)$$

$$\beta_m = c_h \frac{2}{r_e^2} \frac{1}{F + D_m} \quad (7)$$

$$\beta_{zm} = c_v \frac{M^2}{H_0^2} \quad (8)$$

$$M = \frac{2m+1}{2} \pi \quad (9)$$

with  $m = 0, 1, 2, \dots$ ,  $r_e = 0.564 \times S$  = radius of the influence

zone of the vertical drain (for square grids), with  $S$  = drain spacing.  $D_m$ ,  $G$  and  $F$  are defined as follows:

$$D_m = \frac{8}{M^2} \frac{n^2 - 1}{n^2} G \quad (10)$$

$$G = \frac{k_h}{k_w} \left( \frac{H}{2r_w} \right)^2 \quad (11)$$

$$F = \left( \ln \frac{n}{s} + \frac{k_h}{k_s} \ln s - \frac{3}{4} \right) \frac{n^2}{n^2 - 1} + \frac{s^2}{n^2 - 1} \left( 1 - \frac{k_h}{k_s} \right) \left( 1 - \frac{s^2}{4n^2} \right) + \frac{k_h}{k_s} \frac{1}{n^2 - 1} \left( 1 - \frac{1}{4n^2} \right) \quad (12)$$

where:  $k_h$  = horizontal hydraulic conductivity;  $n = r_e/r_w$  = ratio of influence radius to equivalent drain radius,  $r_w = (a+b)/\pi$ ;  $s = r_s/r_w$  = smear ratio, with  $r_s$  = radius of the smear zone;  $k_s$  = horizontal hydraulic conductivity in the smear zone;  $k_w = q_w/A_w$  = hydraulic conductivity of the drain, with  $q_w$  = discharge capacity of the drain and  $A_w = a \times b$  = section area of the drain.

The drain discharge capacity was assumed = 8.4 l/min according to the technical sheet by the producer. Regarding the smear ratio, values in the range of 1÷8 are proposed in the literature (Hansbo, 1981; Indraratna and Redana, 1998; Bo et al., 2000). The value of  $s = 6$  has been selected, as a significant smear effect was observed during trials for the PVD installation at the test field.

The ratio between the horizontal hydraulic conductivity of the sediments and that of the smear zone was assumed equal to 2 (Terzaghi et al., 1996). The drain length,  $L$ , was considered equal to the thickness of the compressible layer,  $L = H_0 = 6.7 \text{ m}$ .

All the values of the modeling parameters are included in Table 4, together with the results of the embankment residence time evaluated for two attempts of drain spacing:  $S1 = 1 \text{ m}$  and  $S2 = 1.5 \text{ m}$ .

Table 4 – Parameters and results of modelling of consolidation process with two different drain squared meshes,  $S1$  and  $S2$ .

Parameters	$S1 = 1 \text{ m}$	$S2 = 1.5 \text{ m}$
$\Delta q_E$ [kPa]		47
$\Delta q_R$ [kPa]		66
$\delta_E$ [m]		0.42
$\delta_R$ [m]		0.52
$U_{ref} = \delta_E / \delta_R$ [-]		0.81
$k_h/k_s$		2
$s$ [-]		6
$q_w$ [l/min]		8.4
$L$ [m]		6.7
$r_e$ [m]	0.56	0.85
$r_w$ [m]		0.033
$n$ [-]	16.9	25.3
$F$	3.8	4.2
$t_R$ [days]	60	117

Figure 5 shows the computed excess pore pressures and degree of consolidation with time, resulting from the supposed load sequence, for the two different drain spacing. For the  $S1$  spacing the reference average degree of consolidation is reached in about 2 months, whereas for  $S2$  in about 4 months. Square grid with 1 m spacing has been selected, in order to reduce test field duration and to avoid possible loose of efficiency of the drains. Therefore, the residence time of the embankment is estimated in 2 months.

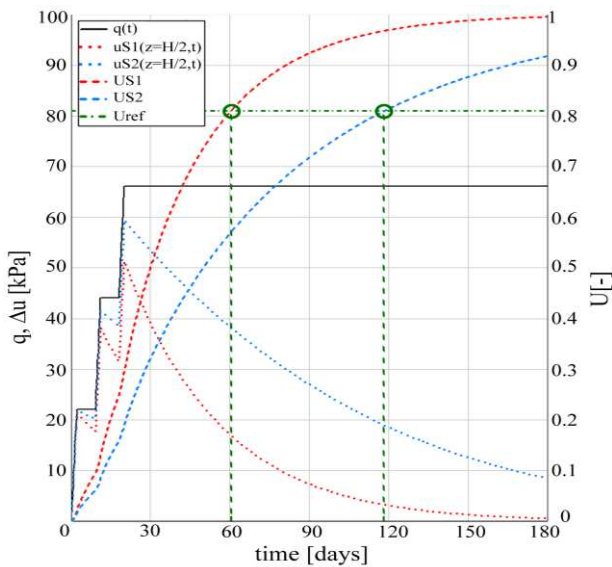


Fig. 5. Consolidation process with  $S1$  and  $S2$  spacing of the PVD;  $q(t)$  = pressure applied versus time;  $uS1$  and  $uS2$  = excess pore pressure induced;  $US1$  and  $US2$  = average degree of consolidation;  $U_{ref} = 0.81$  = target value of the average degree of consolidation.

## 5 CONCLUDING REMARKS

A full scale test field was designed to calibrate modelling and set up operating procedures for consolidation of dredged sediments in a CDF by PVD and preloading. The theory by Tang and Onitsuka (2000) was used for modelling consolidation to consider smear effect, well resistance, horizontal and vertical flows and time-dependend load. Modeling a gradual preloading (necessary to prevent instability) was important since hypothesis of instantaneous loading would have underestimated the consolidation time. Contribution of the vertical flow in consolidation is not negligible for small layer thickness, as in the case of concern.

Aim of the case study was to illustrate the design of a sustainable solution to manage contaminated dredged sediments. The main purpose of the analysis was to evaluate a residence time of preloading compatible with the CDF filling procedure and a solution with a suitable cost-effectiveness ratio. It allowed a proper selection and location of the monitoring tools. A simplified model of the sediment layer has been considered. Modeling of the

lenses, their effects on the consolidation time and compressibility of the sediment layer will be performed on the basis of the monitoring data as well as a parameter analysis of smear effect.

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