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CHALLENGES IN GROUND IMPROVEMENT TECHNIQUES FOR EXTREME CONDITIONS: CONCEPT AND PERFORMANCE

S. Varaksin[1] and K. Yee[2]

ABSTRACT: The development and advancement of ground improvement technology has gone through a fast track. Ground improvement has been used under extreme conditions. In Al Quo'a U.A.E., Dynamic Compaction was carried out to densify up to 28m thick of recently placed non-engineered sand fill. Using L. Menard's recommendation of a 3% induced strain which would double the soil's limit pressure, it was possible to predict the induced deformation and to obtain self-bearing conditions. In Hamburg, Germany the site for the Airbus A380 factory was built on extremely soft alluvium. Innovative solutions using geotextile confined columns and vacuum consolidation was used to provide the necessary stability and consolidation effects. A long term settlement monitoring program of 4 years has validated the successful applications of these innovation solutions despite the extreme conditions of ground improvement.

Keywords: Ground improvement, Dynamic compaction, Vacuum consolidation

INTRODUCTION

Ground improvement technology has shown great developments during the last decades. The combination of technological advancement in specialist equipment and advanced methods of analysis has opened up new avenues for applications of various ground improvement techniques. Ground improvement techniques are broadly categorised into various methods of improvement such as consolidation, compaction and reinforcement according to the nature of the soils to be improved and whether or not with the inclusion of foreign materials e.g. cement grout. Fig. 1 shows some of the more common ground improvement techniques used.

Although analytical designs based on numerical methods are fast gaining popularity, however in practice one should not forget to first establish a general concept which must be sound and economical. Design parameters for analyses which often are dramatically lacking and/or introduced in the design only based on correlations with other "indirect" parameters should be treated cautiously and these parameters should be verified for design.

This paper deals with the applications of ground improvement techniques under extreme types of soil conditions, such as in the case of very loose granular soils to great depths and extremely soft cohesive soils. This paper presents the concept and the long term performance of these ground improvement applications under extreme ground conditions.

CASE OF VERY LOOSE DUNE SAND TO GREAT DEPTH

In the 1950's Louis Menard introduced two new concepts based on his research works on pressuremeter. Since then, these concepts have become major interests for ground improvement design and control in man-made fills and under-consolidated soils.

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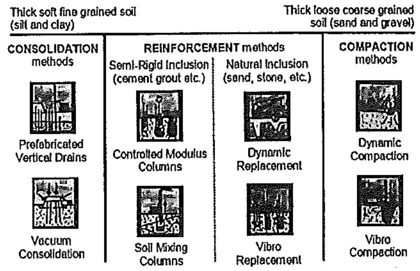

Fig. 1 Common ground improvement techniques

The first concept is self-bearing condition. The second concept is the relationship which quantifies the induced strain in loose granular fill as a function of the soil parameters measured by pressuremeter for self-bearing.

Self-Bearing Condition

On a new man-made fills or a natural under-consolidated soil deposits, it undergoes substantial deformations (settlements) with time even with very lightly loaded structures or even without any imposing load at all. This is often referred to as "creep". The rate of settlement can be accelerated by wetting such as in the case of fills above the ground water table; by vibrations or any other actions which tends to temporary reduce the shearing resistance between the particles (soil grains) contact points allowing the particles to settle into a more compact configuration (soil matrix orientation). Such deformations may be large over a period of time so much so that they are likely to jeopardise the serviceability of the structures erected on such fills or soil deposits. Hence, it is important to recognise and understand this behaviour. The analysis of self-bearing condition is critical to the development of a feasible and economical ground improvement concept which addresses the long term performance and serviceability of the structures on such ground conditions.

The relationship which quantifies the induced strain in such fills or soil deposits as a function of the soil parameter measured by the pressuremeter test was first postulated by Menard. The self-bearing

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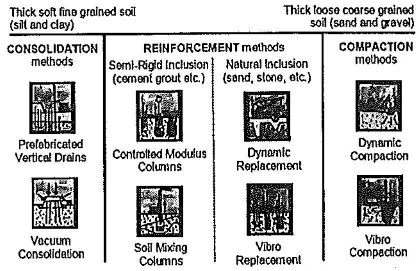

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Table 1: Minimum net limit pressure for self-bearing condition

	Self-bearing condition in terms of net limit pressure, P'_L (bars)
Clay	2.5 – 3
Silt	4
Sand	6
Sand and gravel	8

condition i.e. the level of the soil parameter that the fills or soil deposits must have so as not to settle under its own weight, can be related either to physical or mechanical properties of the fills or soil deposits. Menard (1975) has shown that the limit pressure (P_L) obtained from the pressuremeter test is a suitable characteristic parameter to determine the self-bearing condition. Table 1 indicates the self-bearing condition for different types of soil in terms of limit pressure (P_L).

Case History of Al Quo'a Housing Development, U.A.E.

A new township development project covering an area of 3,000,000m^2 was proposed. The project consisted of building 450 housing units and infrastructures in Al Quo'a in the Emirates of Abu Dhabi, U.A.E.

Realising the effect of self-bearing condition, the concept of ground improvement was to perform deep densification on the loose dune sand fill. The objective was to achieve the minimum limit pressure (P_L) for self bearing by Dynamic Compaction (Yee, 1997). Also, due to the enormous area of work, it was important to estimate reliably the amount of settlement and hence, the amount of additional imported fills to compensate for the induced settlement for cost control and to ensure project economy. Induced strains had to be predicted with reasonable accuracy as it was expected that the surface settlement would exceed 1m in certain parts of the project site.

Typical Ground Conditions.

The working platform as well as the final proposed finished elevations was made by levelling the natural sand dunes to their average elevation to balance the cut and fill volumes. No compaction was carried out during the filling and levelling process as shown in Fig. 2. This resulted in a non-engineered man-made loose fills with a maximum fill thickness of 28 m. Hence, significant long term creep deformation due to self-weight of the fill was expected.

Heavy Dynamic Compaction (HDC) was carried out on the filled areas. The total filled area was about 1,100,000m^2. This area was subjected to HDC treatment for the full thickness of loose fills up to 28m. The HDC works was carried out using a newly developed MARS (Menard Automatic Release System) accelerated impacts where free-fall impacts were made possible. Using this MARS system there was an increase in the effective compaction energy delivered and hence, the depth of treatment was also increased by 10 – 15%. The MARS system is shown in Fig. 3.

The cut-and-fill earthmoving operation was carried out in early 2003. In February 2004, about one year after the earthmoving works, a series of in-situ Menard pressuremeter tests (PMT) were carried out at the 28m thick filled area. The PMT results are shown in Fig. 4.

Based on the pressuremeter test results, the fills had registered low physical and mechanical properties almost throughout the full depth of filling (down to 28m). The mean limit pressure (P_L) was about 3.5 bars (equivalent to a relative density of about 30% - 40%; friction angle of about 30°) except for the upper 2m. The limit pressures at the upper 2 m (mean P_L = 12.6 bars) were significantly higher due to the traffic load of the earthmoving plant and equipment; and also the placement of an overlying blanket layer of compacted granular material locally known as "gatch". This blanket layer was constructed with the intention of providing a stable working platform and to facilitate manoeuvrability of site vehicle movement on site. Based on the results of P'_L < 4 bars, it is true that the loose fills is under-consolidated and subject to self-weight deformation.

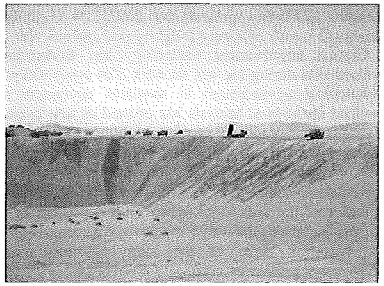

Fig. 2 Non-engineered sand filling process at Al Quo'a, U.A.E.

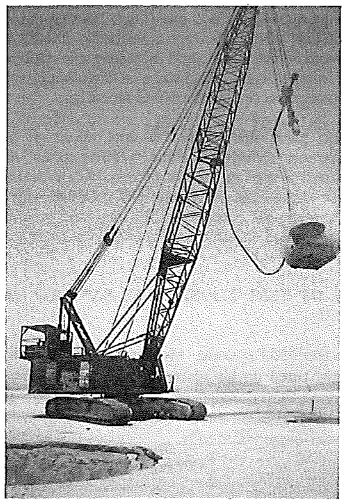

Fig. 3 Heavy Dynamic Compaction using the MARS accelerated impact system

Estimation of Potential Creep Deformation

For the estimation of creep deformation due to self-weight, the following properties have been assumed for the loose fills:

- Density: γ = 18 kN/m^3
- Internal friction angle: ϕ' = 30°

The basic equation for estimating the one-year creep deformation is given below (Menard 1975):

$$w_{(t=1)} = \frac{h}{1000} \cdot \frac{1-\left(\frac{\alpha P'_L}{2}\right)}{\left(\frac{\alpha P'_L}{2}\right)} \tag{1}$$

where $w_{(t=1)}$ is the one-year creep deformation (in mm); h is the layer thickness (in cm); P_L' is the net limit pressure (in bars) given by Eq. (2) and it is valid for $P'_L \geq 2/\alpha$ for self-bearing deformation.

$$P'_L = P_L - P_o \qquad P'_L \geq \frac{2}{\alpha} \tag{2}$$

with P_L being the measured in-situ limit pressure; P_o being the at-rest horizontal pressure of the soil and α is the Menard rheological factor given in Table 2.

Table 2 Menard rheological factor α

	Clay	Silt	Sand	Sand and Gravel
Over-consolidated	1	2/3	1/2	1/3
Normally Consolidated	2/3	1/2	1/3	1/4
Weathered or Remoulded	1/2	1/2	1/3	1/4

The 1-year creep deformation for the 28m thick fill is estimated using Eq. (1) based on the pressuremeter test results of Fig. 4. The computation is given in Table 3. The estimated creep deformation is about 46mm during the 1-year period.

It is assumed that creep deformation follows a decreasing logarithmic law. Hence, deformation during year *n* is given by the following equation:

$$w_{(t=n)} = w_{(t=1)} \cdot \frac{\ln\left(\frac{t+1}{n}\right)}{\ln(t+1)} \tag{3}$$

$$w_{(t=n)} = w_{(t=1)} \cdot T_c \tag{4}$$

where t is the total expected duration of deformation (in years); $W_{(t=1)}$ is the deformation for one-year period and T_c is the time factor for creep deformation.

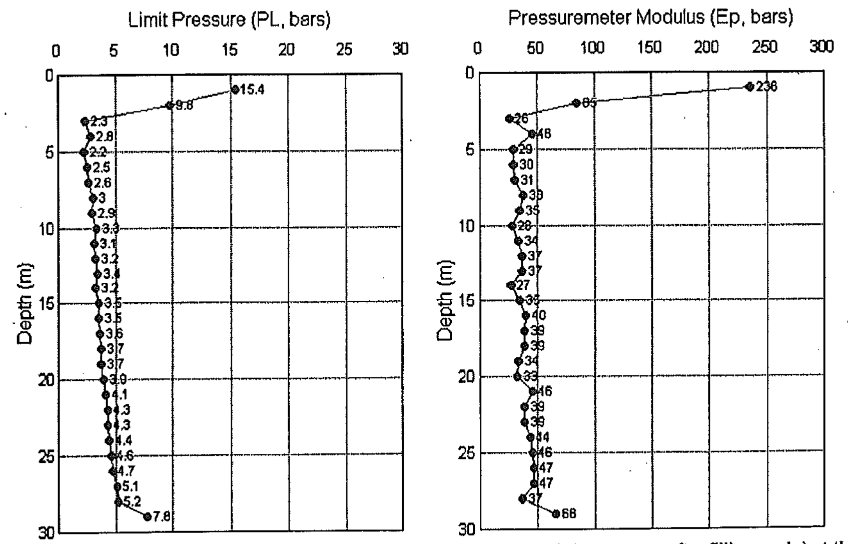

Fig. 4 Results of Menard pressuremeter tests carried out in February 2004 (i.e. one year after filling works) at the filled areas

Table 3 Creep settlement for one-year period

Depth (m)	Over burden σ_o (bars)	$\sigma_h = P_o$ (bars)	P_L (bars)	P'_L (bars)	Settlmnt $W_{(t=1)}$ (mm)
1	0.18	0.09	15.4	15.31	-
2	0.36	0.18	9.8	9.62	-
3	0.54	0.27	2.3	2.03	1.96
4	0.72	0.36	2.8	2.44	1.46
5	0.90	0.45	2.2	1.75	2.43
6	1.08	0.54	2.5	1.96	2.06
7	1.26	0.63	2.6	1.97	2.05
8	1.44	0.72	3.0	2.28	1.63
9	1.62	0.81	2.9	2.09	1.87
10	1.80	0.90	3.3	2.40	1.50
11	1.98	0.99	3.1	2.11	1.84
12	2.16	1.08	3.2	2.12	1.83
13	2.34	1.17	3.4	2.23	1.69
14	2.52	1.26	3.2	1.94	2.09
15	2.70	1.35	3.5	2.15	1.79
16	2.88	1.44	3.5	2.06	1.91
17	3.06	1.53	3.6	2.07	1.90
18	3.24	1.62	3.7	2.08	1.88
19	3.42	1.71	3.7	1.99	2.02
20	3.60	1.80	3.9	2.10	1.86
21	3.78	1.89	4.1	2.21	1.71
22	3.96	1.98	4.3	2.32	1.59
23	4.14	2.07	4.3	2.23	1.69
24	4.32	2.16	4.4	2.24	1.68
25	4.50	2.25	4.6	2.35	1.55
26	4.68	2.34	4.7	2.36	1.54
27	4.86	2.43	5.1	2.67	1.25
28	5.04	2.52	5.2	2.68	1.24
1-year creep deformation due to self weight $\sum W_{(t=1)}$					46.02

For thick fills, the long term deformation will take place over a long period. Two hypotheses have been considered for the total expected duration of creep deformation. They are 50 years and 70 years. The time factor, T_c is given in Fig. 5. Table 4 tabulates some of the values taken from Fig. 5.

Since the pressuremeter test was carried out about one year after the filling process had taken place; hence the 1-year deformation calculated above corresponds to the deformation occurring during the second year after filling. Thus, the time factor T_c needs to be adjusted such that the time factor T_c for year 2 should be 1 instead of 0.824 for 50-yr period and similarly 1 instead of 0.837 for 70-yr period. This is carried out by equating the time factor in year 2 to be unity and apply the adjustment accordingly for the rest of the years. The results are as given in Table 5.

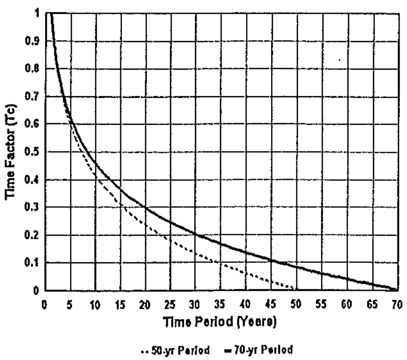

Fig. 5 Time factor T_c for creep deformation

Table 4 Time factor T_c (taken from Fig. 5)

Year (n)	50-yr Period	70-yr Period
1	1.000	1.000
2	0.824	0.837
3	0.721	0.742
4	0.647	0.675
5	0.591	0.622
6	0.544	0.580
7	0.505	0.544
8	0.471	0.512
9	0.441	0.485
10	0.414	0.460
15	0.311	0.365
20	0.238	0.297
25	0.181	0.245
30	0.135	0.202
35	0.096	0.166
40	0.062	0.135
45	0.032	0.107
50	0.005	0.082
55		0.060
60		0.039
65		0.021
70		0.003
$\sum T_{c\,(t=1-n)}$	12.237	15.940

Table 5 Adjusted time factor T_c

Year (n)	50-yr Period	70-yr Period
1	1.214	1.194
2	1.000	1.000
3	0.875	0.886
4	0.786	0.806
5	0.717	0.743
6	0.661	0.692
7	0.613	0.649
8	0.572	0.612
9	0.536	0.579
10	0.503	0.549
15	0.378	0.436
20	0.289	0.355
25	0.220	0.292
30	0.164	0.241
35	0.116	0.198
40	0.075	0.161
45	0.039	0.128
50	0.006	0.098
55		0.072
60		0.047
65		0.025
70		0.004
$\sum T_{c\,(t=1-n)}$	14.856	19.036

Hence, the total creep deformation over a period of 50 years and 70 years is about 68cm and 88 cm respectively. Since one year has elapsed after filling, the remaining creep deformation is expected to be about 63 and 82cm respectively (Table 6).

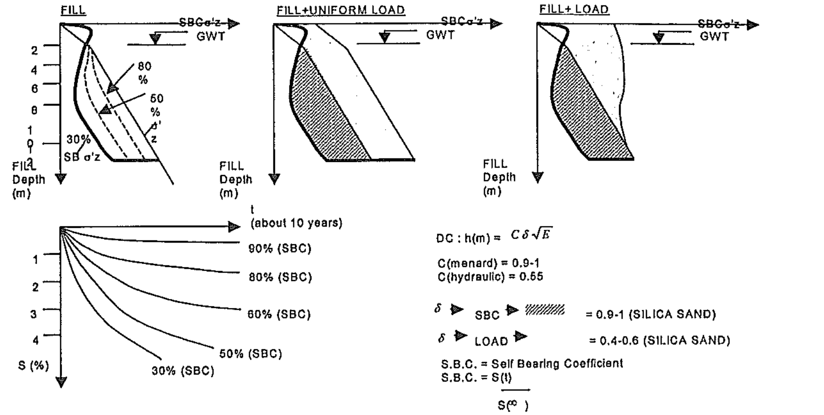

Fig. 6 Concept of self-bearing conditions

Table 6 Creep settlement

	50-yr Period	70-yr Period
Total creep settlement from Year 1	14.856 * 46.02 = 684mm	19.036 * 46.02 = 876mm
Remaining creep settlement	(14.856 – 1.214) * 46.02 = 628mm	(19.036 – 1.194) * 46.02 = 821mm

The settlement rate can be accelerated under external factors such as ground water table variation, seasonal water content variation, vibration due to traffic and other vibrational loadings. Soils with organic content will exhibit larger creep deformation. As a rule of thumb, one percent of organic content will increase the self bearing creep deformation by 20%.

General Concept and Definition of Target Parameters

The acceptance limit pressure (P_L) must be greater than 6 bars ($P_L > 6$ bars) for the deep densification work to eliminate creep deformation due to self bearing of the fills (Table 1). The principles of self-bearing conditions are as shown in Fig. 6.

Prediction of Vertical Strain to be Induced to Reach Acceptance Limit Pressure (P_L)

For an area of treatment of about 1,000,000m^2, it was important to estimate the deformation that would occur during the deep densification work and to determine the required quantity of settlement compensating fill needed to bring the working platform to its final finished elevation.

This estimation was made based on L. Menard's hypothesis that a 3% strain in the granular fill material would double the soil's characteristics defined by the limit pressure, P_L.

Based on the above hypothesis, ε_o is defined as the ratio of the final height (h_{final}) to the initial height ($h_{initial}$) of fill and the improvement ratio is defined as the ratio of the final limit pressure (P_{Lfinal}) to the initial limit pressure ($P_{Linitial}$). When subject to 3% strain (hence, ε_o = 97%), the improvement ratio will be 2. This is represented below:

$$\varepsilon_o = \left(\frac{h_{final}}{h_{initial}}\right) = 0.97 \quad (\text{or } 97\%) \tag{5}$$

and

$$\frac{P_{Lfinal}}{P_{Linitial}} = 2 \tag{6}$$

We call this "phase n = 1". When a further strain of 3% is applied (which is equal to 3% of ε_o = 97% which is 2.91%), the "new" ε_o becomes 94.09% (i.e. 97% minus 2.91%). The improvement ratio is now double to 4. We now call this "phase n = 2". This can be repeated for a number of phases denoted by "n". Thus, the relationship between the induced strain and the improvement of limit pressures can then be established for successive "phases" following the hypothesis as shown in Table 7.

Table 7 Relationship between induced strain and improvement ratio of limit pressure (P_L)

A	B	C	D
Phases n	Improvement Ratio $= P_{L\,final} / P_{L\,initial}$	Height Ratio $= h_{final} / h_{initial}$	Induced Strain $= \Delta h / h$
0	1	100%	0%
1	2	97%	3%
2	4	94.09%	5.91%
3	8	91.27%	8.73%
4	16	88.53%	11.47%
5	32	85.87%	14.12%
6	64	83.30%	16.70%

By generalizing these relationships as shown in Table 8, the following equations can be established.

From column B: $$\frac{P_{Lfinal}}{P_{Linitial}} = 2^n \qquad (7)$$

$$n = \frac{\ln\left(\frac{P_{Lfinal}}{P_{Linitial}}\right)}{\ln(2)} \qquad (8)$$

From column C: $$\frac{h_{final}}{h_{initial}} = (\varepsilon_o)^n \qquad (9)$$

when n = 1; $\varepsilon_o = 0.97$

From column D: $$\frac{\Delta h}{h} = 1 - \left(\frac{h_{final}}{h_{initial}}\right) = 1 - (\varepsilon_o)^n \qquad (10)$$

$$\Rightarrow n = \frac{\ln\left(1 - \frac{\Delta h}{h}\right)}{\ln(\varepsilon_o)} \qquad (11)$$

From Eqs. (8) and (11); we have

$$n = \frac{\ln\left(\frac{P_{Lfinal}}{P_{Linitial}}\right)}{\ln(2)} = \frac{\ln\left(1 - \frac{\Delta h}{h}\right)}{\ln(\varepsilon_o)} \qquad (12)$$

$$\Rightarrow \frac{\Delta h}{h} = 1 - \left(\frac{P_{Lfinal}}{P_{Linitial}}\right)^{\left(\frac{\ln(\varepsilon_o)}{\ln(2)}\right)} \qquad (13)$$

Equation 13 can be used to determine the deformation that will occur for each 1m thick fill layer in order to achieve the acceptance limit pressure defined for self-bearing condition. Table 9 shows the computation of the induced deformation needed by deep densification of the fills to achieve the acceptance limit pressure (P_L = 6 bars). The estimated deformation is about 84cm.

In-situ Verification of the ε_0 Value

The Al Quo'a project provided an opportunity to verify the ε_o value (i.e. the ratio of the final height (h_{final}) to the initial height ($h_{initial}$) of fill) as proposed by L. Menard using the post-treatment pressuremeter tests and the measurement of the deformation.

From Eq. 12, an approximation yields the following expressions:

$$n = \frac{\ln\left(\frac{P_{Lfinal}}{P_{Linitial}}\right)}{\ln(2)} = \frac{\ln\left(1 - \frac{\Delta h}{h}\right)}{\ln(\varepsilon_o)} \qquad (13)$$

$$\ln\left(1 - \frac{\Delta h}{h}\right) \approx -\frac{\Delta h}{h} \qquad (14)$$

and thus,

$$\frac{\frac{\Delta h}{h}}{\ln(\varepsilon_o)} = -\frac{\ln\left(\frac{P_{Lfinal}}{P_{Linitial}}\right)}{\ln(2)} \qquad (15)$$

By choosing the same thickness h_o for all layers, one can sum up the limit pressure improvement ratio over the full depth of the test as defined by Eq. 16 which is derived from Eq. 15 above:

Note: This approximation yields an error of less than 5% for values of n ≤ 3 as computed in the Table 8 below.

Table 8 Generalized relatioships

n	$\frac{\Delta h}{h}$ (*from Table 8*)	$n = \frac{\ln\left(1 - \frac{\Delta h}{h}\right)}{\ln(\varepsilon_o)}$	$n = \frac{-\frac{\Delta h}{h}}{\ln(\varepsilon_o)}$
0	*0%*	*0*	*0*
1	*3%*	*1.0*	*0.99*
2	*5.91%*	*2.0*	*1.94*
3	*8.73%*	*3.0*	*2.87*
4	*11.47%*	*4.0*	*3.77*
5	*14.12%*	*5.0*	*4.64*

Table 9 Required deformations to achieve acceptance (target) limit pressure (P_L)

Depth (m)	Original P_L (bars)	Target $(P_L)^{\text{see \#1}}$ (bars)	Increase Ratio	Strain Δh/h (%)$^{\text{see \#2}}$	Settlmt w (cm)
1	15.4	6.09	-	-	-
2	9.8	6.18	-	-	-
3	2.3	6.27	2.73	4.31	4.31
4	2.8	6.36	2.27	3.54	3.54
5	2.2	6.45	2.93	4.62	4.62
6	2.5	6.54	2.62	4.14	4.14
7	2.6	6.63	2.55	4.03	4.03
8	3.0	6.72	2.24	3.48	3.48
9	2.9	6.81	2.35	3.68	3.68
10	3.3	6.90	2.09	3.19	3.19
11	3.1	6.99	2.25	3.51	3.51
12	3.2	7.08	2.21	3.43	3.43
13	3.4	7.17	2.11	3.23	3.23
14	3.2	7.26	2.27	3.54	3.54
15	3.5	7.35	2.10	3.21	3.21
16	3.5	7.44	2.13	3.26	3.26
17	3.6	7.53	2.09	3.19	3.19
18	3.7	7.62	2.06	3.12	3.12
19	3.7	7.71	2.08	3.17	3.17
20	3.9	7.80	2.00	3.00	3.00
21	4.1	7.89	1.92	2.84	2.84
22	4.3	7.98	1.86	2.68	2.68
23	4.3	8.07	1.86	2.73	2.73
24	4.4	8.16	1.85	2.68	2.68
25	4.6	8.25	1.79	2.53	2.53
26	4.7	8.34	1.77	2.49	2.49
27	5.1	8.43	1.65	2.18	2.18
28	5.2	8.52	1.64	2.15	2.15
Total deformation (cm)					83.92

Note 1: The target P_L is obtained by adding P_o to P'_L of 6 bars for self-bearing conditions of sand.

Note 2: The strain (Δh/h) is given by Eq. 13

$$\frac{\frac{1}{h_o}\sum_{layers}\Delta h}{\ln(\varepsilon_o)} = -\frac{\sum_{layers}\ln\left(\frac{P_{Lfinal}}{P_{Linitial}}\right)}{\ln(2)}$$

$$\Rightarrow \varepsilon_o = e^{\left(\frac{-\ln(2)\sum_{layers}\Delta h}{h_o\sum_{layers}\ln\left(\frac{P_{Lfinal}}{P_{Linitial}}\right)}\right)} \qquad (16)$$

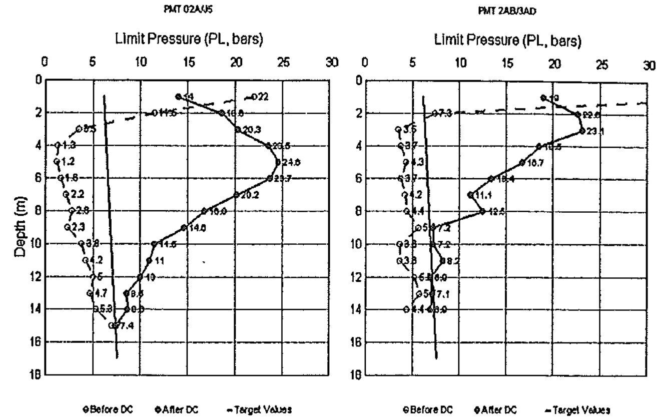

Fig. 7(a) Limit pressures (PL) before and after Dynamic Compaction

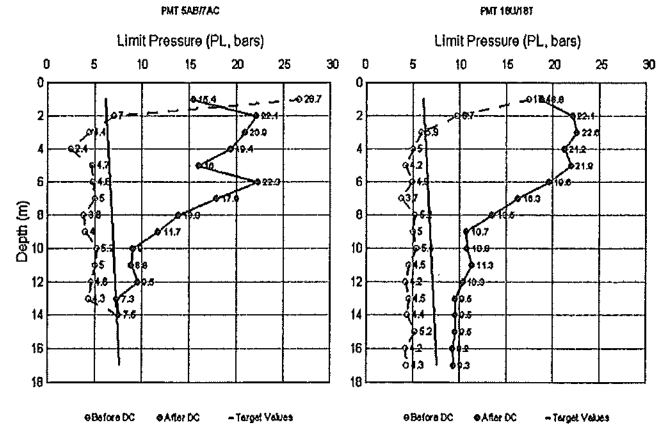

Fig. 7(b) Limit pressures (P_L) before and after Dynamic Compaction

Four pressuremeter test results (Figs. 7(a) and (b)) are used in the analysis for self-bearing conditions. These tests were carried out before and after Dynamic Compaction at different filled areas having thickness (depth) of loose fills ranging from 14 to 17m.

From these results, the settlement, n, ε (=Δh/h) and $(\varepsilon_o)^n$ values are computed as shown in Table 10. These results of ε (=Δh/h) based on calculation and pressuremeter test results are plotted in Fig. 8. It can be shown that the measured values agreed well with the calculated values. Hence, based on this analysis it may be concluded that a 3% strain in the granular fill material would double the limit pressure, P_L as proposed by L. Menard.

A simple equation can be obtained to compute for the induced settlement based on the improvement ratio of the limit pressures as follow:

From Eq. (10): $$\varepsilon = \frac{\Delta h}{h} = 1 - (\varepsilon_o)^n$$

An approximation of $1 - (\varepsilon_o)^n \approx n(1 - \varepsilon_o)$ yields the following equation:

$$\varepsilon = n.(1 - \varepsilon_o) \qquad (17)$$

This approximation yields an error of less than 5% for n ≤ 3.

Table 10 Empirical calculations to determine $(\varepsilon_o)^n$

	PMT 02A/J5				PMT 2AB/3AD				PMT 5AB/7AC				PMT 18U/18T			
Depth (m)	P_{Li} (bars)	P_{Lf} (bars)	Ln. P_{Lf}/P_{Li}	n	P_{Li} (bars)	P_{Lf} (bars)	Ln. P_{Lf}/P_{Li}	n	P_{Li} (bars)	P_{Lf} (bars)	Ln. P_{Lf}/P_{Li}	n	P_{Li} (bars)	P_{Lf} (bars)	Ln. P_{Lf}/P_{Li}	n
1	22.0	14.0	-	-	36.0	19.0	-	-	26.7	15.4	-	-	17.4	18.8	0.08	0.1
2	11.5	18.6	0.48	0.7	7.3	22.6	1.13	1.6	7.0	22.1	1.15	1.7	9.7	22.1	0.82	1.2
3	3.5	20.3	1.76	2.5	3.5	23.1	1.89	2.7	4.4	20.9	1.56	2.2	5.9	22.6	1.34	1.9
4	1.3	23.5	2.90	4.2	3.7	18.5	1.61	2.3	2.4	19.4	2.09	3.0	5.0	21.2	1.44	2.1
5	1.2	24.6	3.02	4.4	4.3	16.7	1.36	2.0	4.7	16.0	1.23	1.8	4.2	21.9	1.65	2.4
6	1.6	23.7	2.70	3.9	3.7	13.4	1.29	1.9	4.8	22.3	1.54	2.2	4.9	19.6	1.39	2.0
7	2.2	20.2	2.22	3.2	4.2	11.1	0.97	1.4	5.0	17.9	1.28	1.8	3.7	16.3	1.48	2.1
8	2.8	16.8	1.79	2.6	4.4	12.5	1.04	1.5	3.8	13.8	1.29	1.9	5.2	13.5	0.95	1.4
9	2.3	14.6	1.85	2.7	5.6	7.2	0.25	0.4	4.0	11.7	1.07	1.5	5.0	10.7	0.76	1.1
10	3.8	11.5	1.11	1.6	3.6	7.2	0.69	1.0	5.2	9.0	0.55	0.8	5.4	10.8	0.69	1.0
11	4.2	11.0	0.96	1.4	3.6	8.2	0.82	1.2	5.0	8.8	0.57	0.8	4.5	11.3	0.92	1.3
12	5.0	10.0	0.69	1.0	5.2	6.9	0.28	0.4	4.6	9.5	0.73	1.0	4.2	10.3	0.90	1.3
13	4.7	8.5	0.59	0.9	5.7	7.1	0.22	0.3	4.3	7.3	0.53	0.8	4.5	9.5	0.75	1.1
14	5.3	8.6	0.48	0.7	4.4	6.9	0.45	0.6	7.5	7.5	0.00	0	4.4	9.5	0.77	1.1
15	7.0	7.4	0.06	0.1									5.2	9.5	0.60	0.9
16													4.2	9.2	0.78	1.1
17													4.3	9.3	0.77	1.1
	$\Sigma P_{L,f}/P_{Li}$			20.15				12.01				13.57				16.11
	Av. n			2.1				1.3				1.5				1.4
	Settlement (m)			0.87				0.51				0.58				0.69
	Strain ($\Delta h/h$) %			5.8				3.7				4.1				4.1
	$(\varepsilon_o)^n$ (%)			94.2				96.3				95.9				95.9

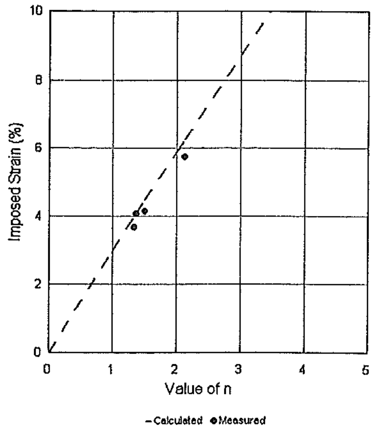

Fig. 8 Imposed strain (= $\Delta h/h$): calculated vs. measurement

Let's define $a = (1 - \varepsilon_o) = 0.03$ (i.e. 3% strain)

$$\varepsilon = n.a \tag{18}$$

From Eq. (8): $n = \dfrac{\ln\left(\dfrac{P_{Lf}}{P_{Li}}\right)}{\ln(2)}$

Substitute n in Eq. 18 will give the following:

$$\varepsilon = \frac{\ln\left(\dfrac{P_{Lf}}{P_{Li}}\right)}{\ln(2)} a \tag{19}$$

The induced layer settlement s will be

$$s = \varepsilon h \tag{20}$$

where h is the layer thickness.

The summation of the layer strains will give the induced ground settlement (s). Hence, it can be written as follow:

$$s = \sum_{k=1,m} h_k \varepsilon_k = \sum_{k=1,m} h_k \frac{\ln\left(\dfrac{P_{Lj}}{P_{Li}}\right)_k}{\ln(2)} a \tag{21}$$

same test location within the improvement zone (i.e. the depth where P_L has increased).

If the ratio of the post treatment over the pre treatment limit pressure ratio is denoted by r, then

$$s = \sum_{k=1,m} h_k \frac{\ln(r)_k}{\ln(2)} a \tag{22}$$

Substituting the value of a = 0.03 and ln (2) in Eq. 22 will yield

$$s = 0.0433 \sum_{k=1,m} h_k \ln(r)_k \tag{23}$$

Similarly, if $\log_{10}$ is used instead of natural logarithmic (ln) Eq. (23) becomes:

$$s = 0.1 \sum_{k=1,m} h_k \log(r)_k \tag{24}$$

The above equations are for $r > 1$.

Summary for Al Quo'a Project

The ground improvement works at Al Quo'a has offered the opportunity to predict creep behaviour in recent young sand fills. It has also offered the opportunity to derive an optimal acceptance criterion based on limit pressure values to avoid over-compaction.

Potential creep of the fill material has been evaluated. Further research based on long time observations should be performed to validate this approach.

Regarding L. Menard's proposal to equate improvement of soil characteristics in terms of limit pressure P_L with the percentage of strain, the actual ground improvement induced strain obtained from the treatment of 1,100,000 m^2 and 140 full treatment depth pressuremeter tests have shown that the recommended value of 3% strain, corresponding to the doubling of the limit pressure is relatively accurate for sandy fills above ground water table.

The concept of using self bearing to determine parameters governing the acceptance of ground improvement works has lead to an economical method of ground improvement controlled by measurable parameters.

Also the empirical relationship between the pre and post treatment limit pressures could allow for the determination of the deformation of the ground surface and thus, allow for estimation of the required amount of settlement compensating fills.

At this stage the township has been completed and the monitoring measurements do not exhibit any measurable deformation.

CASE HISTORY OF THE CONSTRUCTION OF AN INDUSTRIAL PLATFORM ON EXTREMELY SOFT ALLUVIUM

This is a case history on the construction of the Airbus A380 industrial platform in Hamburg, Germany. It illustrates an innovative concept of applied ground improvement techniques under extreme soil conditions and within the particular contractual conditions. Indeed, at the negotiation stage the performance criteria of long term deformation behaviour were not yet established by the owner and a method had to be defined to contractually allow for variations in time schedule and deformation guarantees as a function of time. The only initial parameters known at that particular moment were the ground conditions, nature of fill and final finished elevations.

Project Description and Environmental Constraints.

Airbus Industries had selected a site adjacent to their existing factory and the existing runway for the unloading, assembly, painting and final delivery of the future mega plane, the A380.

This project site covering approximately 140 hectares was a former sand quarry mined in the first half of the 20th century and since then, it was abandoned. It was heavily silted by the Elbe River and thus heavily polluted.

Environmental considerations only allow construction methods which cause minimum lateral displacement of the mud deposits, and provided that all water discharge from the consolidation of the mud must be treated and cleaned due to the presence of large concentration of ammonium and heavy metals.

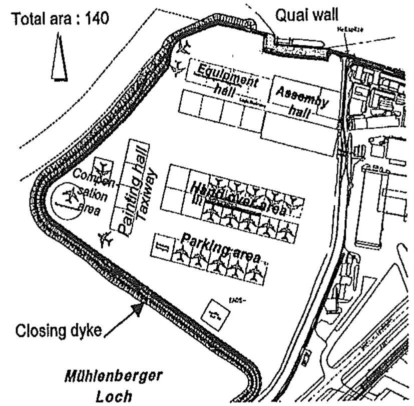

Fig. 9 General site layout plan

Subsoil Conditions.

The site was under tidal influence. Average variation of the tide movement was from -2m to +2m elevation. In certain periods with high tides and East winds, the highest water elevation can reach +6.50m. Table 11 summarises the subsoil properties.

The thickness of the compressible layer varied from 5 m to 14m with very soft surface mud of 3m to 12m thick rendering inaccessibility on site. The only accessibility was by floating flat bottom barges and it was limited to 1½ hour accessibility per tidal movement.

The shear strength of the surface mud with $C_u \approx 0.5$ kN/m² hardly allowed any equipment or any traffic movement on it. Fill in excess of 30cm would not be stable.

Table 11 Subsoil engineering properties

Soil type	Water content	Density	Shear strength		Deformation modulus (under σ_z = 100 kN/m²)	Coefficient of vertical consolidation	Coefficient of secondary consolidation
	W (%)	γ / γ' (kN/m³)	$\delta'(°)/c'$ (kN/m²)	C_u (kN/m²)	E_S (MN/m²)	C_V (m²/year)	$C\alpha$
Mud	142	13 / 3	20 / 0	0.5 - 5	0.8	0.35	0.03
Young clay	119	14 / 4	20 / 0	2 - 10	0.9	0.35	0.03
Clay	70	15 / 5	17.5 / 10	5 - 20	1.5	0.5	0.02
Peaty clay	139	14 / 4	20 / 5	5 - 20	0.9	0.4	0.03
Peat	240	11 / 1	20 / 0	5 - 15	0.5	≥ 0.4	0.04

Considering a final finished elevation at +5.5m and excluding lateral displacement, the calculated vertical deformation under the fill load ranges from 2.5m to 4m. This excludes secondary compression where organic deposits were present.

General Concept of the Engineer

The call for tenders was made by the developer which is a subsidiary of the Port of Hamburg, Germany. However, the basic design of the Engineer had resulted in 3 different tenders.

Tender No.1 - This tender called for a permanent quay wall construction for the purpose of unloading plane elements transported by barge and a peripheral temporary sheetpile wall to contain the muds and isolate the site from tidal influence.

Tender No.2 -The works consisted in raising the water level inside the sheetpile wall to elevation +4.0m then followed by sprinkling some 3,000,000 m² of sand in thin layers. The sand was to be obtained from an island in the Elbe River about 5km away and a site in the North Sea at 180 km distance from the works site.

The sand sprinkling operations were to be performed in maximum of 30 cm thick per single layer to avoid any possible mudwave. This process was to build successive layers up to the final elevation of +3.0m. The next phase of works would consist of lowering the ground water level to elevation +0.7m allowing a suitable working platform to support very light construction equipment. Vertical drains were to be installed in non structural areas and vacuum consolidation (Menard type) in the structural areas measuring some 204,000 m².

The aim of the vacuum consolidation process was to allow easy filling operations and to reach the deformation criteria in a very short time without any risk of failure.

Tender No.3 - The works would consist of the construction of a permanent dyke on the consolidated grounds within the closing dyke and the removal of the temporary sheetpiles.

Alternative Concept by the Specialist Contractor

In 1994, the harbour of Lübeck which is located approximately 80 km North East of Hamburg had a successful experience of backfilling an old harbour using cohesive polluted dredge spoil with a sand cover deposited using the sprinkling method. The consolidation of the dredge spoil was then carried out within a short time period using the vacuum consolidation process.

The engineer of the Airbus Project has since adopted a similar concept in Hamburg based on this successful experience.

The specialist contractor had presented an alternative method that would avoid the construction of the temporary sheetpile wall. This alternative method required the construction of the permanent dyke of tender no. 3 within the tender no.1 period which was 8 months and thus ahead of the annual high tide period that would eventually destroy the works if the dyke was not closed. This alternative method is shown in Fig. 10.

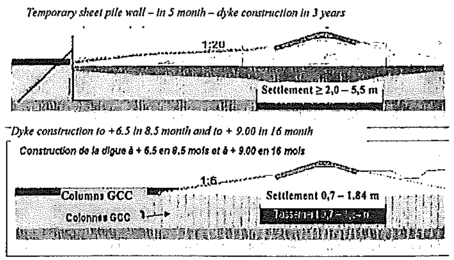

Fig. 10 Basic design and alternative concept of the specialist contract

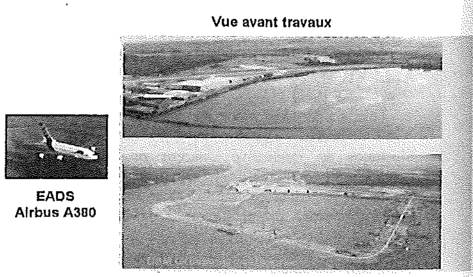

Fig. 11 Dyke construction for the polder formation

The technique of using geotextile confined columns (GCC) was proposed. These columns were constructed from a floating barge. This technique was used to ensure the stability of the closing dyke; to avoid lateral mud displacement in the adjacent Elbe River; and to reduce and accelerate settlement of the subsoil with time (Fig. 11)

The use of GCC was demonstrated previously on a test site within the Port of Hamburg and it had allowed construction and raising of the closing dyke in 3 successive stage constructions over a period of 3 months.

The vertical drains and vacuum consolidation would allow the subsoil inside the dyke to consolidate within the allocated schedule and to achieve the performance criteria.

Since the leasing of the land by Airbus Industries to the Port of Hamburg (the owner) was not finalized by then and as such, the alternative method was accepted and awarded to the specialist contractor based on quantities to be defined by an established calculation method (TARAO) defined in the contract as the works progress and taking into account the actual quantity of sand delivery, the residual settlement criteria and their time of validity only to be defined at a later stage.

Some General Aspects of the Geotextile Confined Columns.

This technique consists of driving or vibrating a 80cm diameter steel casing to the bearing soil followed by placing a seamless cylindrical closed bottom geotextile "sock" with tensile strength ranging from 200 to 400 kN/m. This followed by filling it with sand to form a sand column (Fig. 12). Raitel and Kemfert (1999) and Raithel et al. (2002) have reported the analytical and numerical methods of design for this scheme.

Some General Aspect of the Vacuum Consolidation Process

The vacuum consolidation is an alternative to surcharge with vertical drains. Details are found in Yee and Tan (2002) and Yee et al. (2004). The general principle following the French Method (Menard Technique) is presented in Fig. 13.

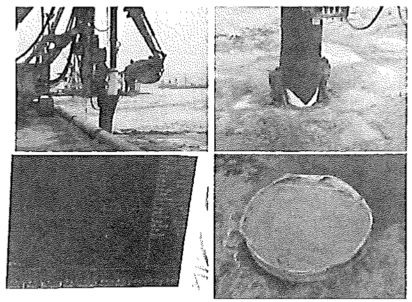

Fig. 12 The construction of the Geotextile Confined Columns

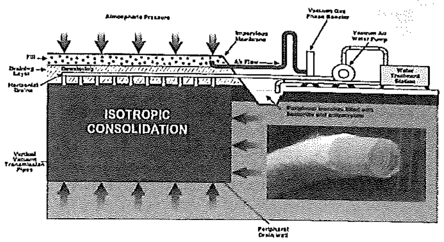

Fig. 13 Vacuum consolidation process

The particularity of the vacuum consolidation process as developed by JM Cognon (Cognon 1991; Cognon et al., 1994) is the dewatering below the membrane which permanently keeps a gas phase between the membrane and the lowered water level.

The p'-q' diagram (Fig. 14) illustrates the difference between surcharge and vacuum process. The vacuum process moves away from the failure line, K_f and stays below the K_o line. The surcharge process moves towards the K_f line and stays above the K_o line. This explains how vacuum consolidation prevents lateral movement and even creates the opposite phenomenon of inward movement of the treated area.

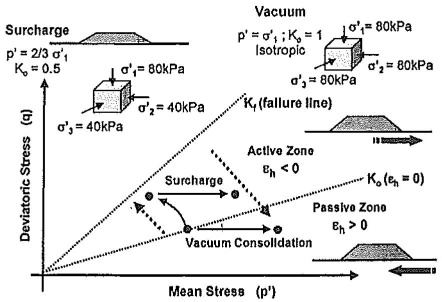

Fig. 14 Stress path for vacuum process (Yee *et al.,* 2004)

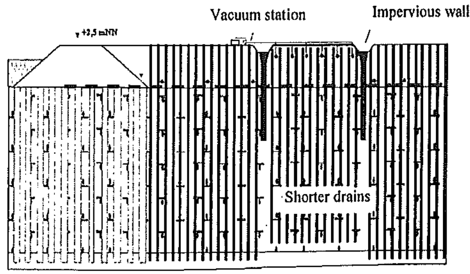

Fig. 15 Installation of the "vacuum" corset

Particular Application of Vacuum as Confinement "Corset" in the Assembly Hall Area

The engineer's request to accelerate the works program and early hand-over of the assembly hall area with a revised period of 8 months (even before the completion of the closing dyke) required special treatment and adaptation to the 130,000 m² area.

A "fast-track emergency scheme" was launched and the method of construction implemented is as shown in Fig.15.

Construction of a "mini" dyke on four rows of G.C.C. covered with sand bags was undertaken to resist a water pressure of 2.5m height. This was followed by filling the basin with water to elevation +2.5m and also sprinkling of sand to elevation +2.5m. Installation of a corset with vacuum consolidation inside the "mini" dyke providing the required stability for the hydraulic fills that had to be placed over a period of few weeks.

For this purpose, the combination of the pre consolidation of the mud in the vacuum process and the shear strength increment, including the apparent cohesion in the sand layer under the vacuum were taken into consideration for the stability analysis.

The analysis of deformations (including horizontal movement) and stability by finite element method has demonstrated that sand filling from elevation +2.5 to +9.5, with slopes of 1V:4H could be stabilised by the vacuum corset and also consolidated according to the client's specifications.

Figure 16 illustrates the behaviour of the different subsoil layers as function of time as compared to the theoretical analysis based on laboratory obtained parameters and the contractually defined "TARAO" calculation method (see Table 12).

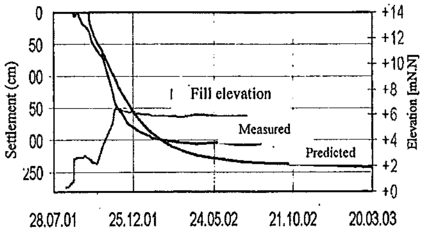

Fig. 16 Calculated settlements against measured values for the North Area

Table 12 Table of soil parameters used in TARAO calculations

Hambourg A380
DS 59 - XI1017-B62 - (4.19 u.4.20) - +5,50mNN - Raster: 0,60m
MVC-Block Rollwege in FS

Fill parameters :	
Density (KN/m³):	18
Cohesion (KPa):	0
Friction angle (°):	27,5
Initial elevation :	3,50
Initial fill thickness above GWT :	0,50
Initial fill thickness below GWT :	3,10

LOAD PARAMETERS										
Parameter	unit		Step1	Step2	Step3	Step4	Step5	Step6	Step7	Step8
Fill density	KN/m³	γ_t	18	18	18	18	18	18	18	18
Fill height above GWT (incl. exist. fill, begining of step, no settlement considered)	m		0,50	1,00	1,00	2,00	5,50	5,50	5,23	5,23
Maintain elevation (yes=1, no = 0) ?			0	0	0	0	0	0	0	0
Fill height above GWT (incl. exist. fill, end of step)	m	H_f	-0,28	-0,38	-1,03	-0,46	2,30	2,26	2,00	1,86
Fill thickness below GWT (incl. settl, end of step)			3,88	4,48	5,13	5,56	6,30	6,34	6,33	6,47
Fill settlement (end step)	m	W_t	0,78	1,38	2,03	2,46	3,20	3,24	3,23	3,37
Fill width and length	mxm	(Lxl)	Infinite	Infinite	Infinite	Infinite	Infinite	Infinite	Infinite	Infinite
Vacuum Pressure	KPa	V	0	0	60	80	80	25,6	25,6	35,2
Coefficient of Vacuum		α_V	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
Duration of loading	days	t	25	74	62	57	221	149	39	1825
Time at beginning of step	days		0	25	99	161	218	439	588	627
Date at beginning of step			22/08/2002	16/09/2002	29/11/2002	30/01/2003	28/03/2003	04/11/2003	01/04/2004	10/05/2004
Fill elevation from GWT (begining step)	m		0,50	1,00	1,00	2,00	5,50	5,50	5,23	5,23
Fill elevation from GWT (end step)	m		-0,28	-0,38	-1,03	-0,46	2,30	2,26	2,00	1,86
Elevation at end of step	m		3,22	3,12	2,47	3,04	5,80	5,76	5,50	5,36

INITIAL SOIL PARAMETERS										
Layer	unit		1	2	3	4	5	6	7	8
Thickness	m	H	5,10	2,50	1,00	0,30	1,00	0,80		
Void ratio Tarao		e_t	3,7	2,7	3,6	3,6	1,4	2,7		
Void ratio		e_0	3,3	2,7	2,7	2,7	1,4	2,7		
Primary consolidation Tarao		C_{ct}	0,95	0,83	0,90	0,90	0,45	0,83		
Primary consolidation		C_c	1,15	0,95	1,00	1,00	0,45	0,95		
Secondary consolidation Tarao		$C_{\alpha t}$	0,141	0,110	0,184	0,184	0,024	0,110		
Secondary consolidation		C_α	0,030	0,030	0,040	0,040	0,010	0,030		
Density	KN/m³	γ_s	13,00	14,00	12,00	12,00	16,00	14,00		
Cohesion	KPa	C	0,00	0,00	0,00	0,00	0,00	0,00		
Cohesion increase		$\Delta C_u/\Delta\sigma'$	0,20	0,20	0,20	0,20	0,20	0,20		
Internal friction angle	°	ϕ	20,00	20,00	20,00	20,00	20,00	20,00		
Effective stress	KPa	σ_0'	7,65	20,30	26,30	27,60	30,90	35,50		
Influence factor of surcharge	%	I	100	100	100	100	100	100		
Calibration coefficient		β	1	1	1	1	1	1		

DRAINAGE PARAMETERS										
Parameter	unit		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Layer 7	Layer 8
Drain spacing (square)	m	D	0,6	0,6	0,6	0,5	0,5	0,5		
Drain diameter (MCD)	m	d	0,05	0,05	0,05	0,05	0,05	0,05		
Coefficient of horizontally drained consolidation	m²/yr	C_h	0,35	1,6	1,2	0,4	0,7	0,4		

Application of Vacuum Consolidation at the Structural Areas (Taxiway, Parking Areas)

The stringent criteria of time-deformation behaviour for the taxiway and apron areas of less than 10cm allowable post construction settlements had limited the possible choice of ground improvement techniques. Vacuum consolidation was selected to solve the stability problem associated with the filling operations and to satisfy the required long-time settlement criteria.

Furthermore, the structures were to be constructed at areas of thick compressible soils extending to greater depth and it contained organic matters.

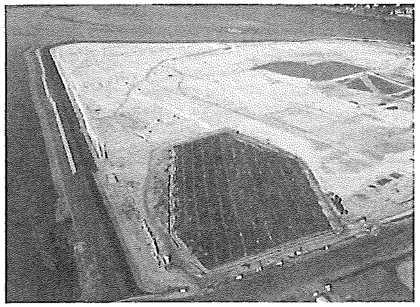

Fig. 17 Vacuum application on the taxiway area

Fig. 18 Construction of the impervious wall at the vacuum treatment area

To reduce the secondary compression to meet the required post construction residual settlement of 10 cm under the fill, structures and live loads, substantial over-consolidation and "aging" effects had to be induced. Hence, the vacuum consolidation is used to provide the necessary surcharge effect by vacuum depressurization without inducing excessive lateral deformation and instability.

The installation of the vacuum system required the utilisation of high capacity trenching machines, capable to install the geo-membrane and bentonite wall to 8m depth (Fig. 18). Indeed, the upper layers consisting of "sprinkled" previous sand had to be totally isolated by a cut-off geomembrane. The deep seated sand seams were trenched, mixed with clay from the mud layers and a bentonite injection rail equipped with a trenching arm to create a 40 cm thick impervious cut-off wall.

Fourteen pumping stations, each with one air-water, and one air-air vacuum pump were installed on a 8m high and 2.5m diameter casing, placed on a concrete pad equipped with a sump pit. This was to facilitate the air-water pumps to be close to the water table and protected from the fill operation during their hydraulic filling process (Fig. 19).

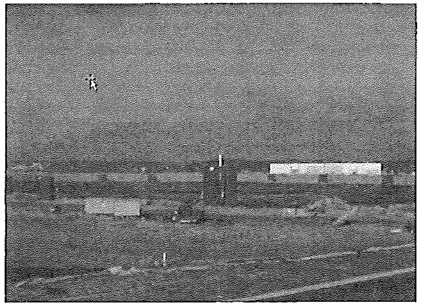

Fig. 19 Protection casings for the vacuum pumps

The principle of calculation for deformation is based on the "target void ratio" to be reached in each layer in order to meet the specifications under full effective stress in each layer concerned. Table 12 shows the main parameters used in the calculation using TARAO program. It takes into account the water table variation due to tidal action, and the variation of fill stress due to buoyancy effect as the fill settled below water.

Figure 20 shows the settlements obtained from the surface settlement plates located at the vacuum treated area plotted against the height of filling. The settlement records included a 3-month settlement monitoring period after the end of the vacuum process.

Figure 21 presents the settlement monitoring data over a period of 3 years and 10 months after the works. The deformation included additional load from the infrastructure construction and some live loads. Settlement measurements were taken until 17th of March 2004 by the specialist contractor Menard and from this period until today, the Engineer (IWB) in charge of the project continues with the settlement monitoring.

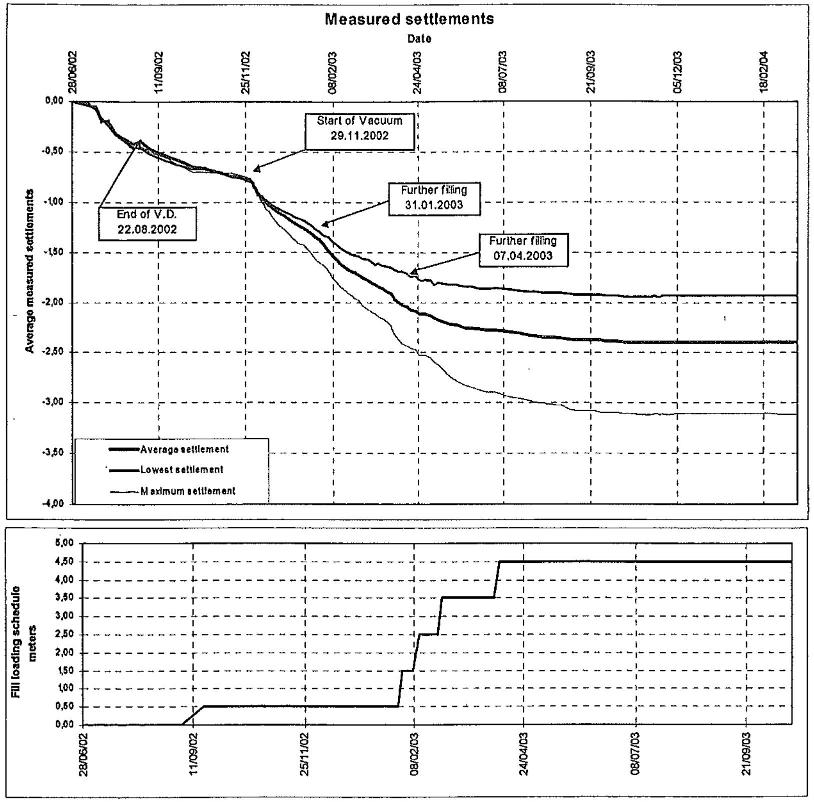

Fig. 20 Records of deformation (settlement) and fill heights

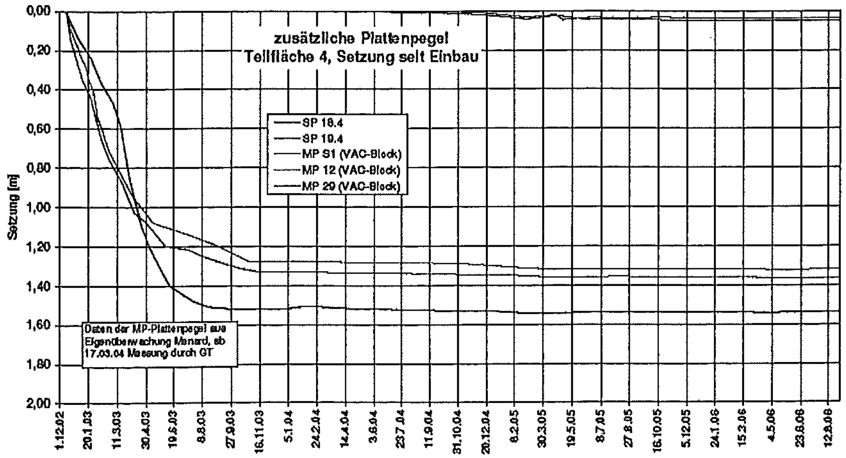

Fig. 21 Long term post construction settlements records

CONCLUSIONS

The two case histories covering different types of ground improvement techniques applied under extreme soil conditions exhibit a common point i.e. the optimisation of practitioners' capabilities of developing a concept including the necessary tools and method of analysis to obtain the desired result.

For the deep granular soils fill at Al Quo'a site, self bearing concept and the necessary dimensioning (optimisation of compaction energy) of the compaction works was adopted and a verification of the strain as function of the limit pressure could be concluded.

For the extremely soft cohesive soils in Hamburg, geotextile confined columns and vacuum consolidation were applied by their practitioners and worked in a general concept with tools and available dimensioning methods.

A detailed monitoring program covering the ground improvement works from its inception until today for well over four years has validated the soundness of the concept and the dimensioning methods utilised.

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