JET GROUT STABILIZATION OF PEATY SOILS UNDER A RAILWAY EMBANKMENT IN ITALY

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ABSTRACT: The construction of two railway embankments on a peaty soil deposit up to 7m thick required a foundation improvement program meeting very strict specifications in terms of environmental protection, speed of execution and early effectiveness. The problem was solved by a jet grouting procedure, selected and optimized after exhaustive field testing. Design features and results of laboratory tests on samples recovered after the treatment are outlined, together with a statistical interpretation intended to estimate the in situ composition of the treated soil. Finally, the instrumentation installed for the long term monitoring of strain distribution in the ground is described, and the preliminary data collected are reviewed.

INTRODUCTION

The Gothard International Railway Line, connecting Milan in Italy to Basel in Switzerland, carries very heavy traffic: 130 daily trains with a yearly volume of 3.2 million passengers and 8 million tonnes of freight. The section from Milan to Chiasso, though more than a century old, can still cope efficiently with the traffic requirements. However, there is another stretch, close to Chiasso (Figure 1), where there are major restrictions imposed by the size of the 2000m long Monte Olimpino 1 tunnel, and by the steepness of the tracks from Como to Albate (1.8%) and towards Chiasso (1.38%).

The Italian State Railways therefore commissioned the construction of an additional line for the international traffic, leaving the existing line for commuter traffic to and from the city of Como. From Field Station 39+545m, the additional line would depart from the existing one, run 1500m with a 0.15% gradient and then enter the Monte Olimpino 2 tunnel, 7209m long with a 0.8% gradient. The problems involved in constructing the new tunnel, and the special techniques adopted (soil freezing and horizontal jet-grouting) have been described elsewhere (3).

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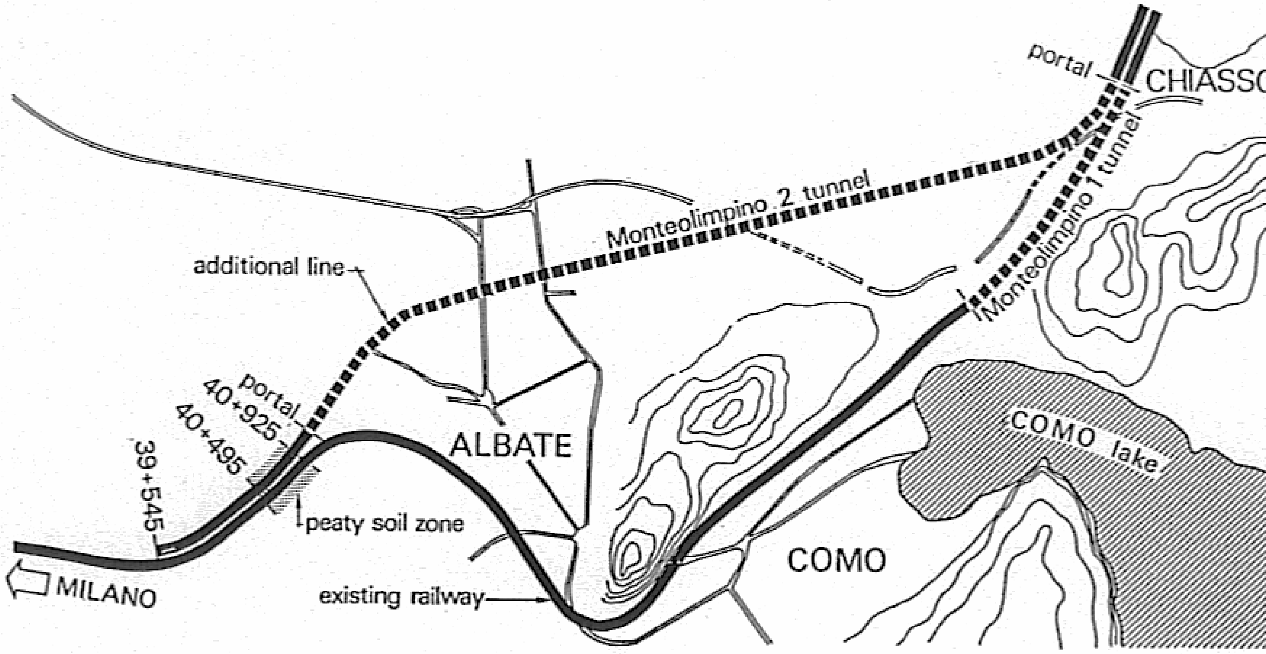

Figure 1. Layout of existing and additional railway lines, and location of peaty zone.

Near the new tunnel portal, the new route flanks the existing line running on highly compressible peaty soil mostly consisting of fibrous peat with a silty-clayey matrix. The thickness of this formation averages 5m. The old track, between Stations 40+495 and 40+925, built more than one century ago on similar soil, has required frequent maintenance works owing to continuous settlements of the embankment. At present, levelling surveys and remedial works are being carried out twice a year. For the new embankment foundation, the owner requested that the problem be resolved to ensure maintenance-free operation without even temporary interference to the adjacent operating line.

SOIL CONDITIONS

The peaty soil deposit has a thickness generally ranging between 4 and 6m, and overlies a sandy gravelly formation. Figure 2 shows the results of soil classification tests with depth (exploratory borehole S1). From data on absolute specific gravity, and assuming a specific gravity $G_m = 2.7$ for the mineral fraction, it was possible to calculate for the organic matter a specific gravity G_o between 1.3 and 1.5 as shown in Figure 3. The high compressibility was displayed by the range of oedometer modulus when plotted against effective pressures: 0.15 to 0.3 MPa at overburden pressure. The rate of secondary compression was between 0.02 and 0.03.

SELECTION OF THE FOUNDATION IMPROVEMENT METHOD

Potential improvement methods can be evaluated according to their capacity to:

a) prevent embankment failures,
b) control settlements and displacements,
c) allow settlements only during or immediately after construction in order to minimize maintenance charges,

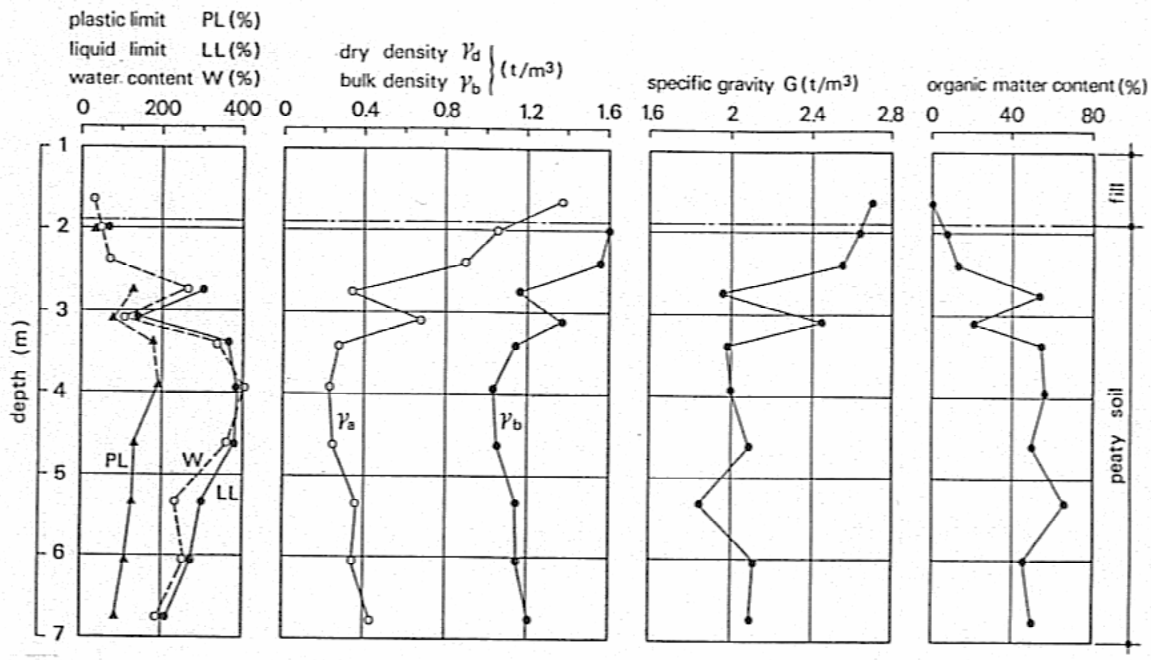

Figure 2. Characteristics of peaty soil with depth (borehole S1)

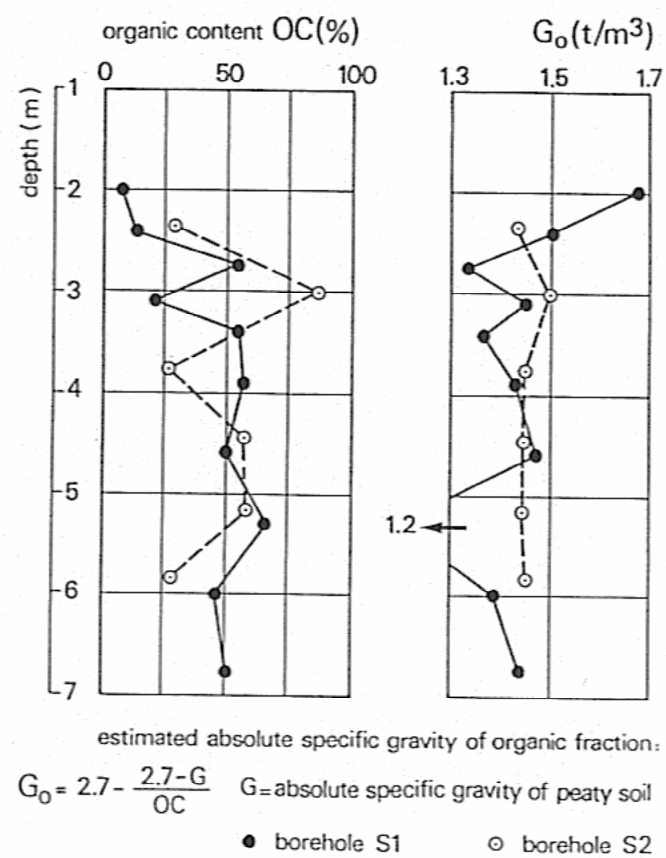

Figure 3. Content and estimated specific gravity of organic matter.

d) reduce vibratory effects of traffic,

e) keep the effects on adjacent structures within allowable limits, during and after construction.

In this case, condition a) was not significant due to the embankment height (less than 2m above ground level). However, condition c) was critical and the presence of the existing railway line close to the new embankment emphasized the importance of conditions b) and e). In addition, a very short construction time was specified for the whole scheme, involving soil improvement, embankment and ballast laying, and commissioning.

Various improvement methods were studied, including removal and replacement, pre-loading by surcharge, sand drains, compaction piling, and in-place mixing with cement or lime. None of these solutions met fully the requirements of environmental protection, rapid execution and short term effectiveness, except, perhaps, the last.

In comparison with methods based on mechanical deep mixing with lime or cement, jet grouting was finally selected owing to its potential for forming columns of larger diameter (0.6 to more than 2m), reducing therefore the working time per unit volume of treated soil, and providing a more homogeneous and effective improvement of mechanical properties. To explore the effectiveness of the jet gouting techniques, an extensive field trial was carried out.

The principles of the various types of jet grouting have been described in detail elsewhere (2, 4, 5, 6). In summary, the soil may be fractured and simultaneously mixed in place with a cement grout, or alternatively removed to a certain extent and partly replaced by the grout. The processes involve the use of one fluid (grout), two fluids (air jetting to enhance the facturing effect of the grout, or three fluids (air and water as fracturing and washing media, the grout as replacement.)

All three procedures were tested in the trial:

RODINJET 1: cement grout injected at a nozzle pressure of 40 MPa.

RODINJET 2: compressed air at 1 MPa around cement grout at 40 MPa through coaxial nozzles.

RODINJET 3: water at 50 MPa inside a cone of compressed air at 1 MPa through upper coaxial nozzles, and cement grout at 8 to 10 MPa through lower nozzles.

Holes were drilled with a 124mm diameter tricone bit, 89mm diameter rods and water flush.

A total of 9 trial columns were formed (Table 1): two by Rodinjet 1, two by Rodinjet 2 and five by Rodinjet 3. During drilling, a "pre-washing" treatment by coaxial air-water jet was carried out in five columns in order to improve both fracturing effect and partial replacement. In addition, two grout mixes were used (w/c = 0.7 and 0.85 by weight). The main grout properties were: bulk density 1.57 - 1.64 t/m3; bleed capacity 2-5%; Marsh viscosity 32-37 sec.; unconfined compressive strength 25-30 MPa after 28 days.

Table 1 details the operational parameters used for each column. Figure 4 presents the discharge/pressure correlations of the grouting equipment related to the type of fluid (water and cement grouts) and to the diameter and number of injection nozzles.

Approximately 15 days after grouting, the columns were exposed to a depth of 3.5m to observe their dimensions. The measured diameters of columns within two depth ranges are reported in Table 2. In general, the diameter was large in the upper parts (fill) while the important effect of pre-washing in the peaty soil is highlighted by the larger diameters of columns C2 to C4 in comparison with C0 and C1.

Procedure	Column number	Column depth (m)	Prewashing		Grouting					Cement mix C/W
			Pressure (MPa)		Pressures (MPa)			Injection quantities		
			water	air	water	air	cement mix	dry cement (t/m)	cement mix (m3/m)	
RODINJET 3	C_0	8,50	-	-	50	1	8	1,4	1,633	1,2
	C_1	7,50	-	-	50	1	8	1,4	1,633	1,2
	C_2	6,50	20	1	50	1	8	1,6	1,867	1,2
	C_3	6,50	20	1	50	1	12	1,4	1,467	1,4
	C_4	6,50	20	1	50	1	12	1,6	1,676	1,4
RODINJET 2	C_5	6,50	20	1	-	1	40	0,6	0,628	1,4
	C_6	6,50	30	1	-	1	40	0,6	0,700	1,2
RODINJET 1	C_7	6,50	-	-	-	-	40	0,2	0,210	1,4
	C_8	6,50	-	-	-	-	40	0,2	0,233	1,2

Table 1. Operational parameters used for 9 trial columns.

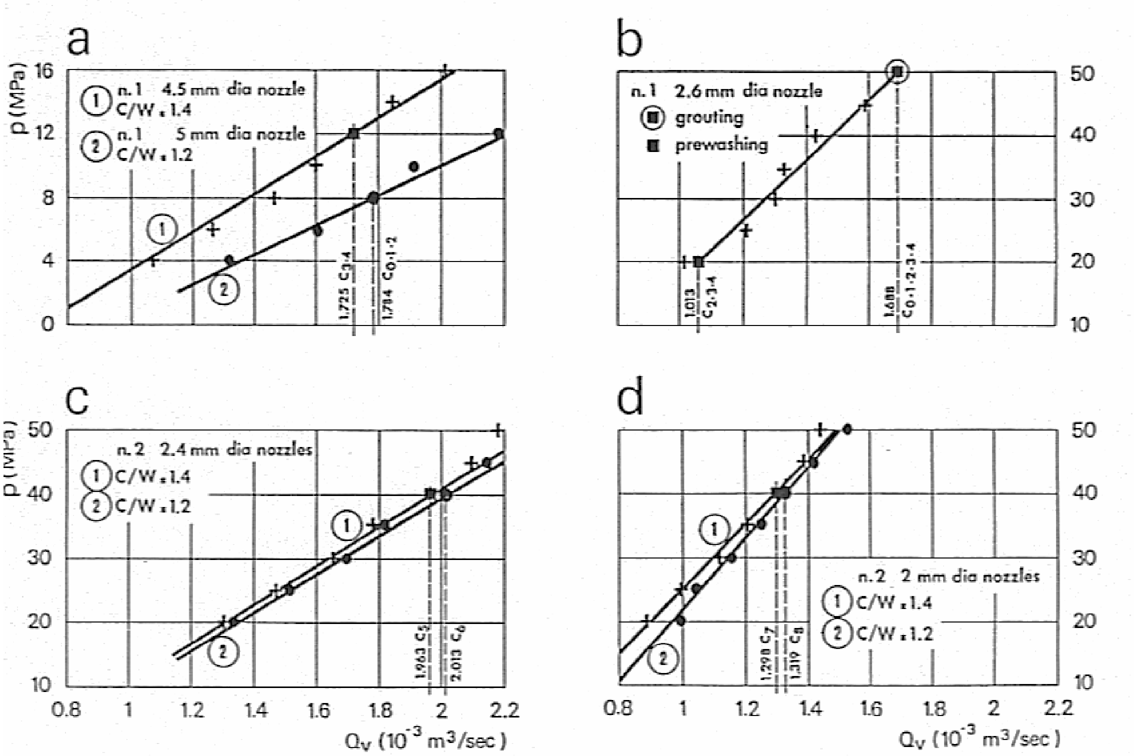

Figure 4. Grouting pressure (p) and rate of discharge (Qv) as a function of the fluid, and the nozzles for a) Rodinjet 3 grouts, b) Rodinjet 3 water, c) Rodinjet 2 grouts, d) Rodinjet 1 grouts.

Procedure	Column number	Column diameter (m)	
		depth (m)	
		from 0 to 1.50	from 1.50 to 3.50
RODINJET 3	C_0	2,00	1,50
	C_1	2,20	1,50
	C_2	2,20	2,10
	C_3	2,40	2,10
	C_4	2,50	2,20
RODINJET 2	C_5	1,80	1,60
	C_6	1,80	1,50
RODINJET 1	C_7	0,80	0,70
	C_8	not recorded	not recorded

Table 2. Average diameters of trial columns.

The actual diameters obtained in the peaty soil were correlated with the following parameters:

- <u>injection pressure</u> (p): mean ejection pressure (MPa) applied to the fracturing fluid (and then expended for erosion, cavitation, mixing and expulsion). MPa
- <u>discharge</u> (Qv): rate of fluid ejection (m^3/sec).
- <u>power</u>(P): mean power delivered by the fluid exiting the nozzle, expressed by the product Qvp (kW).
- <u>specific energy:</u>
  per lineal meter of column: Esl = P.T (MJ/m)
  per cubic meter of column: Esc=P.T/S (MJ/m3)

where T is the specific injection time (sec./m and S the cross sectional area of the column (m^2)

(Note on Units: 1 W = 1 joule per second
1 MJ = 10^6 Joules
1 KW = 1.34 HP)

Figure 5 represents Esl as a function of diameter while Figure 6 shows the amounts of energy expended for the different columns. The data in both these figures emphasize the influential role of pre-washing:

- the amount of energy required for columns C0 and C1 was almost double that for columns C2 to C6, where pre-washing was performed.
- for applied energy of the same order, the column diameter D obtained with pre-washing increased from 1.5m (C0 and C1) to 2.2m (C2 to C4).
- fairly linear correlations are evident between energy and diameter for these two groups. In addition, the Rodinjet 3 technique clearly gave the largest column diameters.

The mechanical properties were evaluated by unconfined compression tests on cylindrical specimens cored by rotary drilling and trimmed from blocks recovered during excavation. Figure 7 shows the average strength values obtained as a function of the amount of cement

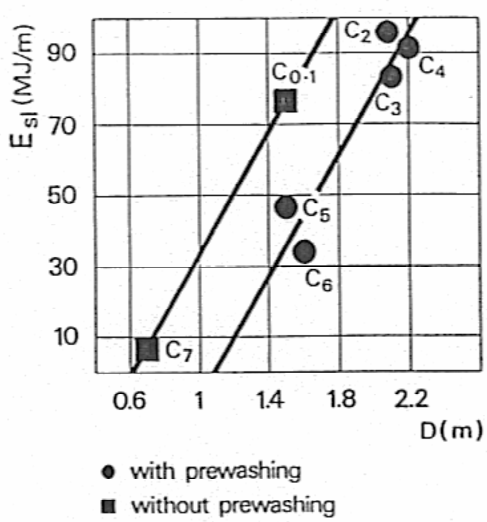

Figure 5. Relationship of specific energy per column meter (Es1) and column diameter (D).

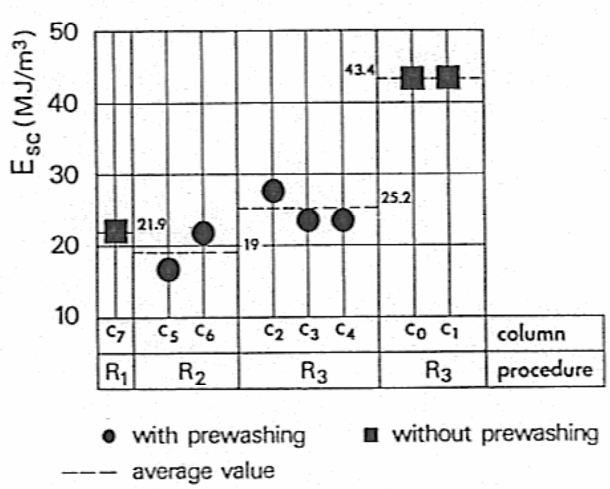

Figure 6. Specific energy per cubic meter of jet grouted soil (Esc) per column.

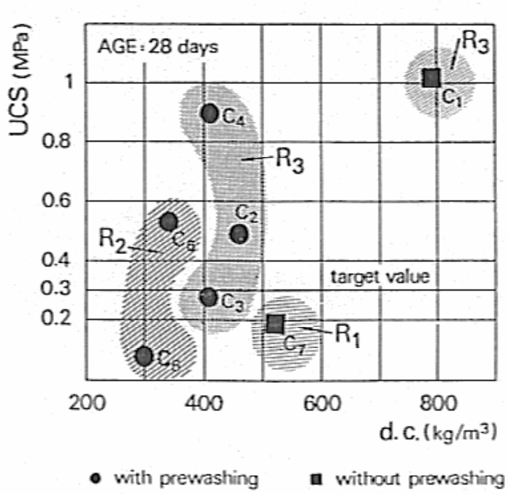

Figure 7. Relationship of column unconfined compressive strength, and dry cement injected per cubic meter of jet grouted soil (d.c.)

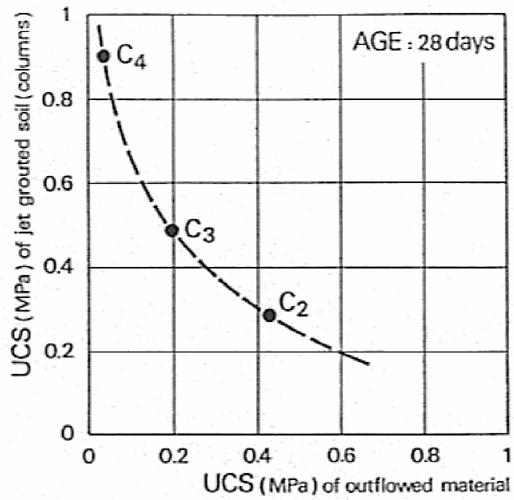

Figure 8. Comparison between strength of jet grouted soil in situ and the expelled material.

injected per unit volume of treated soil. Despite the scatter of results, only columns C5 and C7 were far from the target strength of 0.3MPa. The different mean strengths of columns C2, C3, and C4 executed by the same procedure (Rodinjet 3 with pre-washing) were likely due to field variations in discharge characteristics. This is illustrated in Figure 8 which shows an inverse relationship between the strength of the grouted material in situ and the strength of the material ejected during grouting.

The main conclusions drawn from the trial were:

- a design diameter of 2.1m could be reliably assumed for columns formed by Rodinjet 3 with pre-washing.
- a water/cement ratio of 0.7 and a cement content between 1.4 and 1.6 tonnes/m^3 were suggested.
- since a fairly wide range of strengths might be possible, control tests on cored samples were required during the course of the work.
- a long term strength target of 0.3 MPa was realistic.

DESIGN AND EXECUTION

The design was consequently based on columns 2.1m in diameter, formed by the Rodinjet 3 technique with pre-washing, and arranged as shown in Figure 9. The distance between centers was 2.75m along five longitudinal rows 2.15 to 2.80m apart. The columns, embedded 0.5m into the underlying sandy gravelly formation, involved the treatment of about 60% of the peaty soil foundation. The general working schedule was planned in three stages:

<u>Stage 1</u> - construction of a railway embankment for the diversion of the two tracks in service at that time.

<u>Stage 2</u> - traffic diversion onto this new embankment, permitting the dismantling of the corresponding stretch of the existing line.

<u>Stage 3</u> - construction of the new permanent embankment and line.

For the construction of both embankments the working sequence was:

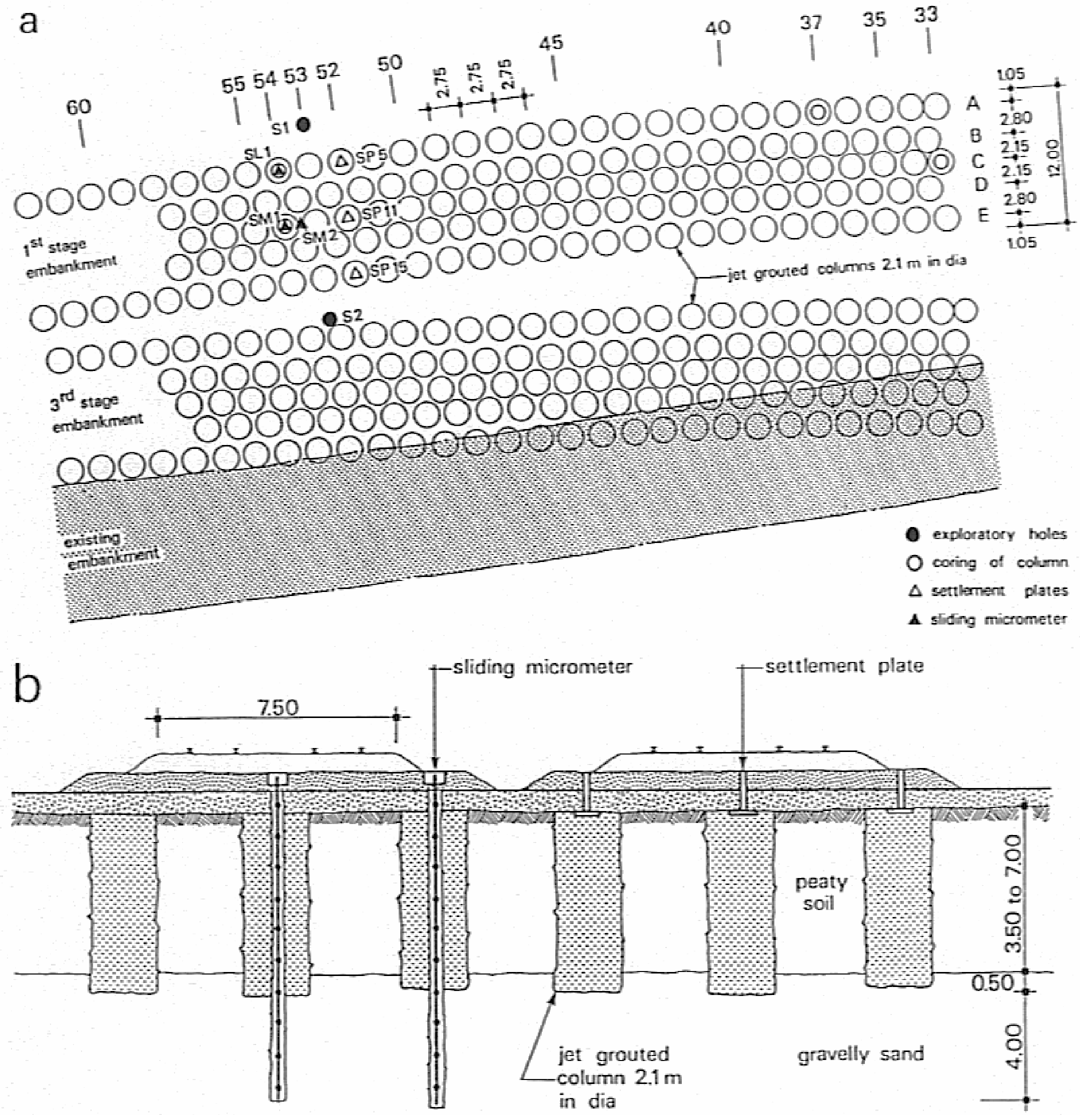

Figure 9. Typical plan (a) and cross section (b) of embankments, soil improvement, and instrumentation.

- formation of a platform for working equipmnent, consisting of a layer of granular material about 1.5m thick.
- soil improvement by jet grouting.
- removal of the temporary platform, levelling of the top of columns and formation of the subgrade.
- construction of the embankment, laying of ballast and railway equipment.

Stage 1 was completed in four months without any inconvenience, as checked by continuous levelling during the course of works. It involved 800 lineal meters of embankment underlain by about 1300 columns, totalling 7000m, and treating about 25000 cubic meters of soil. Instrumentation for long term monitoring of strain distribution in the ground was installed as shown in Figure 9.

QUALITY ASSURANCE

Drilling and Sampling. Boreholes were cored through the centers of four columns, after one to three months, allowing continuous sampling by means of double tube barrels. Figure 10 shows the treated soil profiles summarized in terms of strength range (from unconfined compressive strength testing on 38 selected samples), the recovery ratio and RQD. The treated peaty soil ranged in thickness from 3.5m (column 37/A) to nearly 5m (column 54/C). The natural gravels in the lower sections also contained some of the fill material placed to create the working platfom and which had dropped during the pre-washing operations.

Laboratory Tests. All samples appeared to be uniformly cemented with a stiff to very hard consistency.

Bulk and dry density, water content, and unconfined compressive strength were evaluated on specimens having the same diameter of cores (about 90mm) and a height/diameter ratio equal or close to

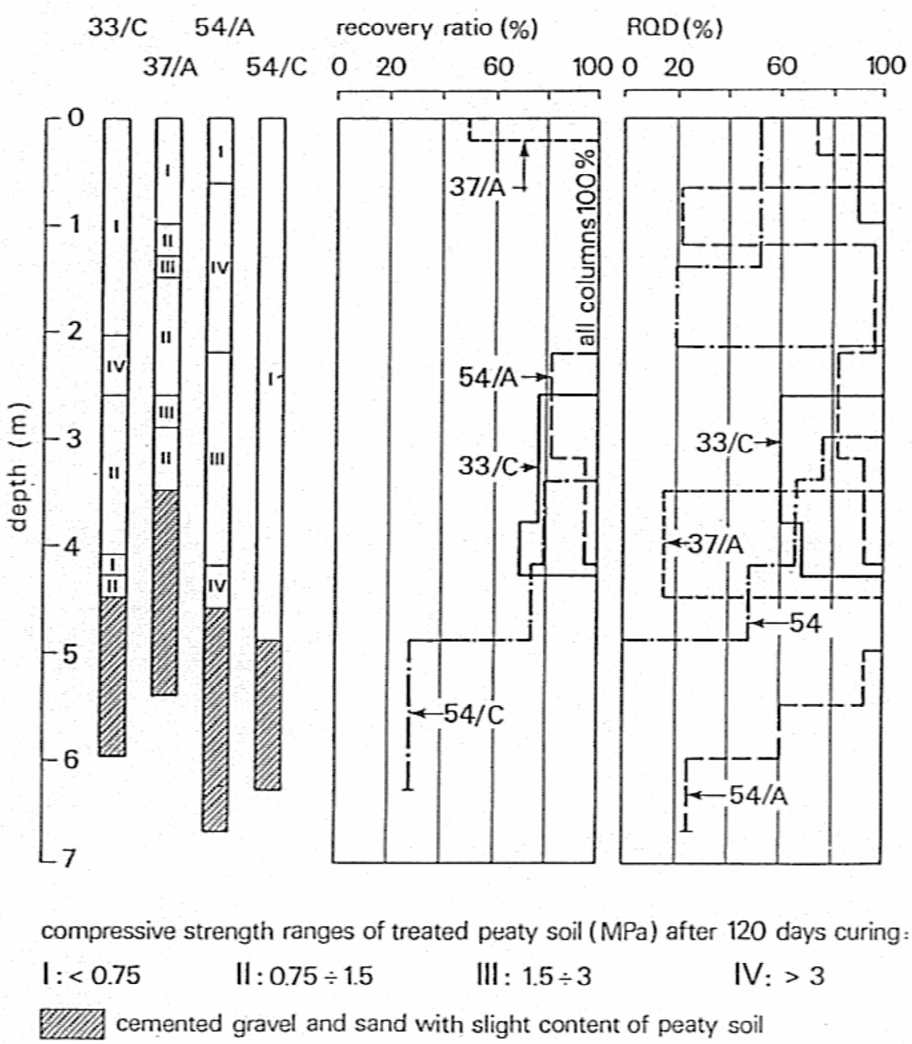

Figure 10. Core hole data through centers of four production columns

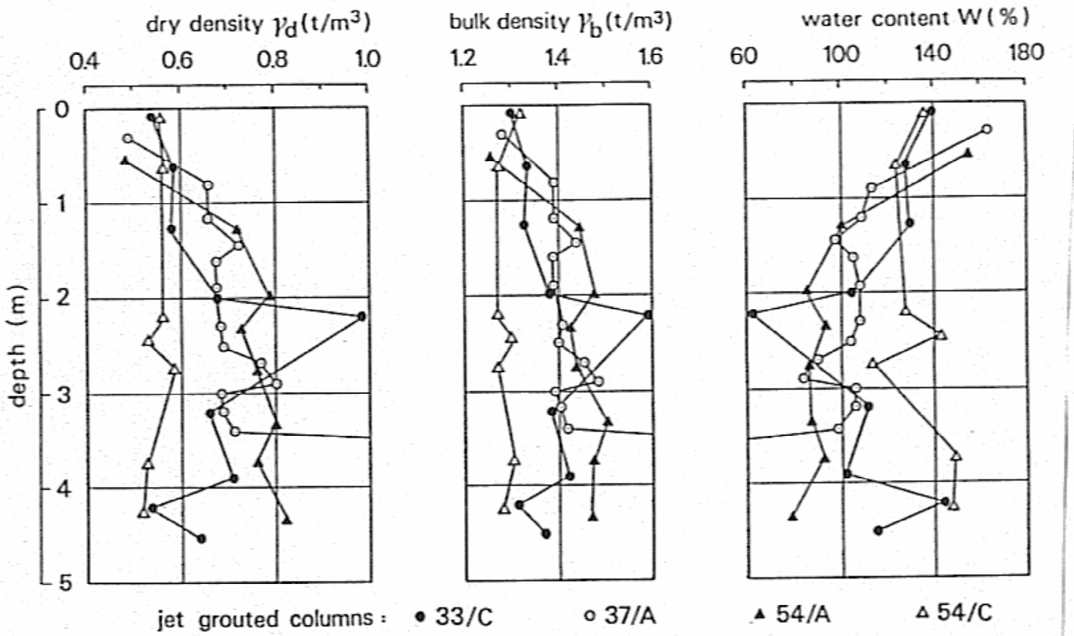

Figure 11. Density and water content of jet grouted peaty soil, with depth.

two. Figure 11 shows for each column the plots of the volumetric parameters versus depth, within the treated peaty soil. As regards the experimental data obtained by standard oven drying, the overestimate of dry density and a consequent underestimate of water content were ascribed to the incomplete evaporation of water, bonded in the grout (1). As described below, an alterative approach to this problem leads to more realistic results, in good agreement with calibration tests on oven dried soil-cement samples with known composition. On average, the experimental values of dry density were 9 percent higher and those of water content 13 percent lower than the alternative method.

The unconfined compression tests were performed 120 days after grouting. The values of compressive strength (R) and elastic modulus (E) are plotted versus depth in Figure 12. As shown in Figure 13, strength generally increased with bulk density to a value of about 2 MPa. Considering the mean values in each column and within four strength ranges, the correlation of Figure 14 is obtained. The peak value of 7 MPa is included but the extrapolation of the correlation in the range between 4 and 7 MPa without intermediate data is tenuous.

As regards deformability, Figure 15 shows the mean values of elastic modulus (E) versus strength (R) within the same strength ranges as above.

Estimated Composition of Jet Grouted Peaty Soil. The composition may be estimated in terms of cement (C), dry soil (S), and total water (W) contained in the unit volume of treated soil according to the general expression of bulk density:

$$\gamma_b = \frac{1 + C/W + S/W}{1 + \frac{C/W}{G_C} + \frac{S/W}{G_S}} \qquad (1)$$

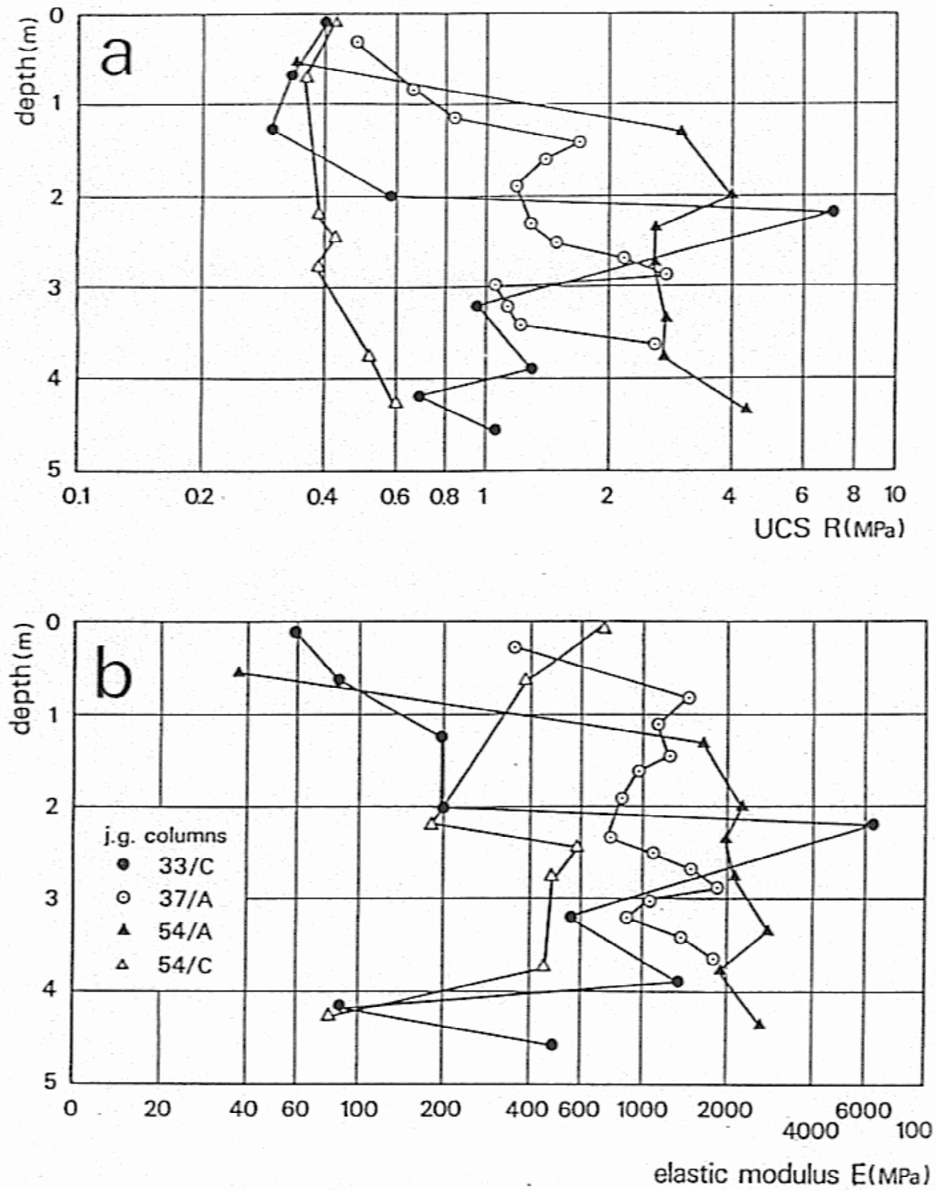

Figure 12. Unconfined compressive strength (a), and elastic modulus (b) of treated soil, after 120 days, with depth.

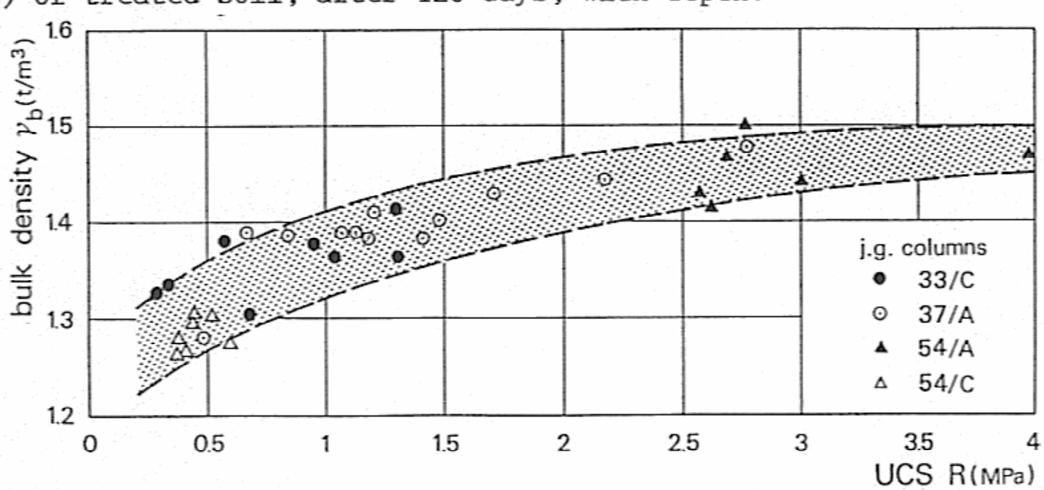

Figure 13. Relationship between bulk density of treated soil and unconfined compressive strength, after 120 days.

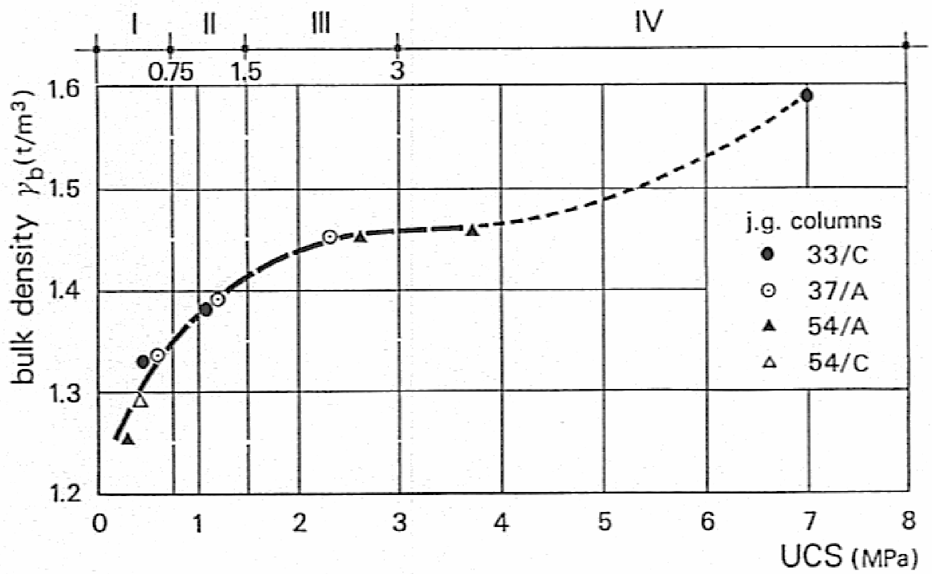

Figure 14. Relationship between bulk density of treated soil and averaged 120 day unconfined compressive strength values for each strength range (I to IV).

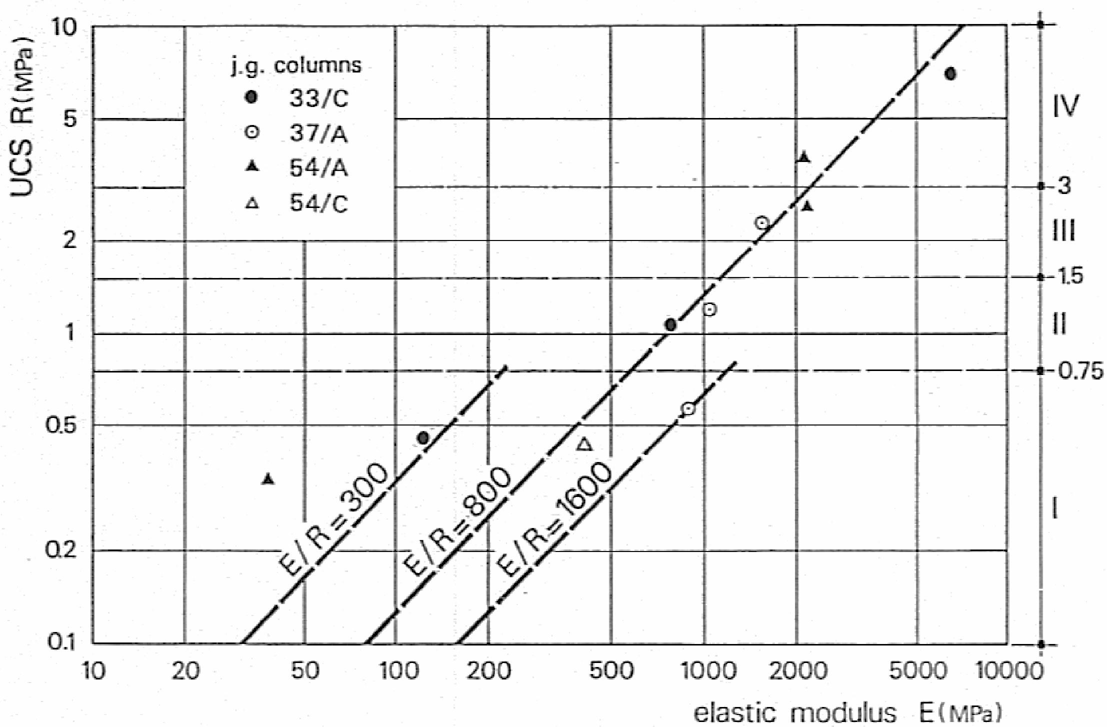

Figure 15. Relationship between elastic modulus and averaged 120 day unconfined compressive strength values (in strength ranges I to IV) of treated soil.

This assumes:
- full saturation,
- strength (R) mainly depending on C/W ratio,
- a reliable correlation of C/W and experimental R values,
- known absolute specific gravity of cement (G_C) and soil particles (G_S).

According to preliminary tests on trial cement/peaty soil mixes with variable composition, the following mean relation for unconfined compressive strength at 120 days can be adopted:

$$R = 15 \cdot (C/W)^3 \text{ (MPa)} \qquad (2)$$

Hence:

$$C/W = 0.405 \cdot R^{1/3} \qquad (3)$$

Introducing (3) into (1) and assuming:

$$G_C = 3 \text{ tonnes/m}^3 \text{ and } G_S = 2.3 \text{ tonnes/m}^3$$

the only unknown, S/W, of relation (1) can be determined, and therefore, the weight of components C, S, W or any pair of R – γb data.

The sum (C+S) and the ratio W/(C+S) gives more reliable values of dry density and total water content than direct experimntal data on oven dried samples as outlined previously. Moreover, the volumetric cement grout content can be estimated, introducing the cement/water ratio C/Wg = 1.4 :

Vg = C/Gc + C/(C/Wg) = C/3 + C/1.4 = 1.048.C (litres), with C in kg.

The statistical results of the analysis are plotted versus strength in Figure 16 and summarized in the lower part of Table 3 with reference to the general mean values within each of the four strength ranges I to IV. The mean values related to single columns are shown in Figure 17, including for comparison the trial columns C2, C3, C4 executed by the same jet grouting procedure, and summarized in the upper part of Table 3. The 28-day data have been extrapolated to 120 days by introducing a multiplying factor of 1.44. It is noteworthy that 3 of the 4 production columns have mechanical properties and compositions within the same range recorded for the trial columns. As shown in Table 4, the mean values of the two groups of columns are almost identical, with the exception of column 54/A, likely to be statistically abnormal in terms of strength. This leads to the following conclusions based on a conservative analysis of available date:

- the mean strength of columns is likely to range mostly between 0.4 and 1.2 MPa. Excluding column 54/A the general mean decreases from 1.3 (Table 3) to 0.8 MPa (Table 4).
- the cement content ranges between 230 and 340 kg/m^3. The mean value of 290 kg/m^3 corresponds to about 70% of the injected cement, assuming the average diameter of 2.1m verified in the trial columns.
- the jet grouted soil contains about 300 liters of grout per cubic meter and 260 kg of dry soil on average.

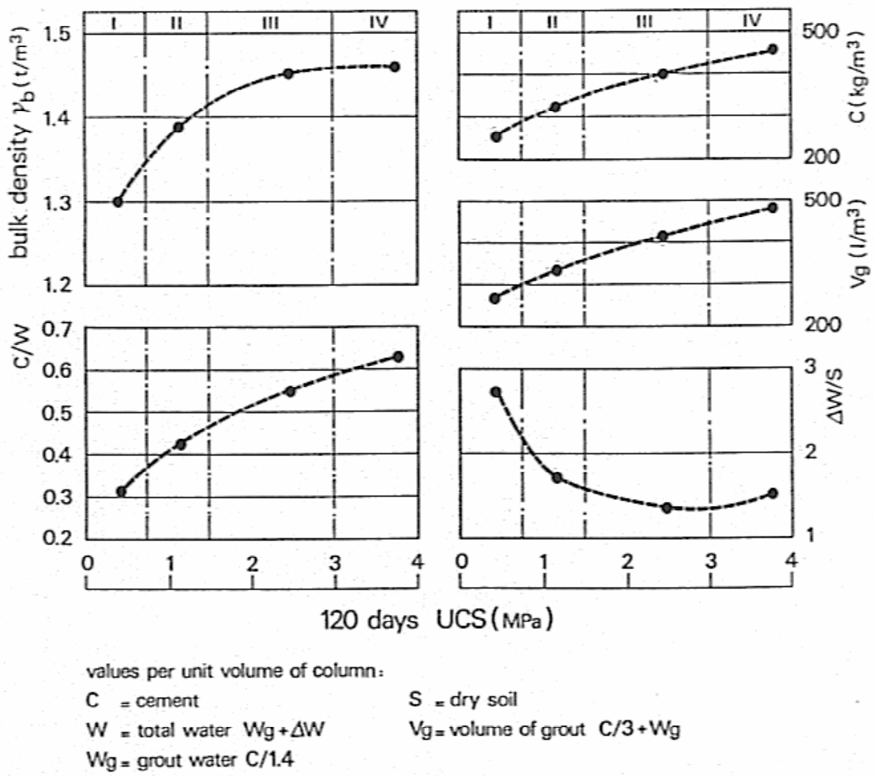

Figure 16. Estimated composition of treated soil according to experimental data (in strength ranges I to IV).

Column n.	Strength Range	γ_b kg/m^3	R MPa	C/W	C kg/m^3	S kg/m^3	W l/m^3	Wg l/m^3	ΔW l/m^3	Vg l/m^3
33/C		1358	0.77	0.371	288	293	777	206	571	302
37/A		1385	1.18	0.429	327	296	762	234	528	342
54/A		1429	2.83	0.574	428	254	747	306	441	449
54/C		1283	0.44	0.308	255	200	828	182	646	267
	I	1300	0.45	0.311	253	232	815	181	634	265
	II	1390	1.14	0.424	321	311	758	229	529	336
	III	1453	2.47	0.548	296	334	723	211	512	415
	IV	1452	3.76	0.631	460	267	730	329	401	482
General weighted mean values		1363	1.34	0.447	350	229	784	250	534	367

<u>Experimental mean values</u>

γ_b = Bulk density = C+S+W

R = 120 d. U.C. strength

<u>Estimated mean values per unit volume of column</u>

C = cement content

S = dry soil content

W = total water content = Wg + ΔW

Vg = volume of grout

Wg = grout water = C/1.4

Table 3. Results of statistical analysis of data from 4 production columns.

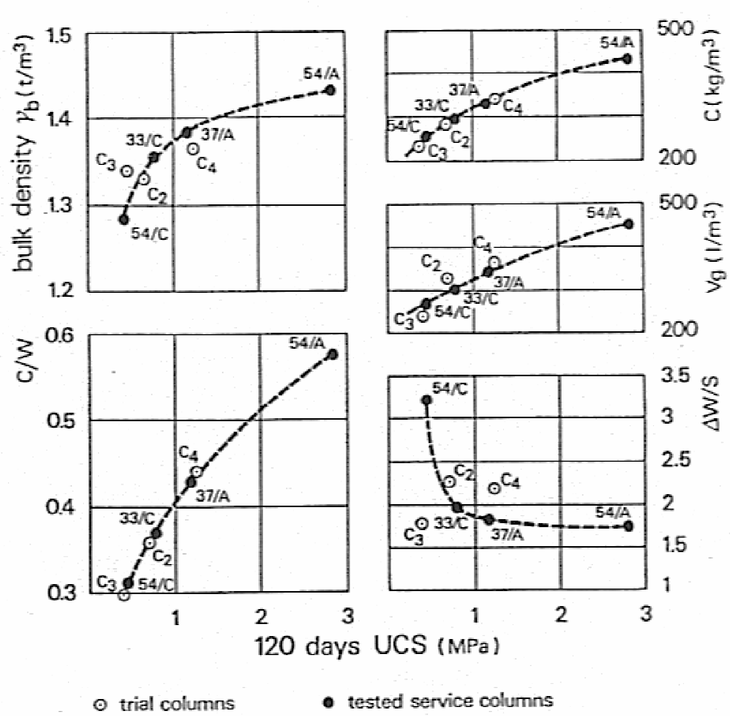

Figure 17. Estimated composition of treated soil according to experimental data (mean values of test and production columns).

Columns of jet grouted peaty soil		γ_b kg/m³	R MPa	C/W	C	S	W	Wg	ΔW	ΔW/S	Vg 1/m³
					kg/m³		1/m³				
Trial C_2 C_3 C_4	min	1330	0,39	0,296	229	243	774	164	536	1,78	240
	max	1365	1,26	0,438	342	344	797	244	610	2,27	358
	mean	1347	0,78	0,373	293	268	786	209	577	2,15	307
Service 33/C 37/A 54/C (*)	min	1283	0,44	0,308	255	200	762	182	528	1,79	267
	max	1385	1,18	0,429	237	296	828	234	646	3,23	342
	mean	1342	0,80	0,376	297	255	790	212	578	2,27	311

(*) except Column 54/A   Symbols : see Table 4

Table 4. Comparison of data from trial and production columns (symbols as for
Table 3).

- the retained soil is likely to have lost part of the purely organic fraction druing pre-washing and water jetting during the treatment.

- this assumption agrees with the estimated content of water $\triangle$W exceeding the grout water Wg. In fact, the mean $\triangle$W/S ratio (Table 4) is lower than the mean water content of native peaty soil.

- overall, no dilution effect seems to be introduced by water jetting, since the total retained water is less than the sum of water contents related to grout and native soil.

Instrumentation. Some cross sections of the embankment have been monitored with:

- strain meters (sliding micrometers) located both within and between columns of jet grouted peaty soil.
- settlement plates on top of columns.
- inclinometers.

With reference to the typical section of Figure 9, Figure 18 shows:

- the vertical strains recorded by sliding micrometers SL1, SM1 and SM2 for columns 54/A, 54/C and peaty soil between 53/C and 54/C respectively.
- the total settlements of columns 52/A/C/E recorded by plates SP5/11/15 respectively.

About 130 days after completion of the first stage embankment, the maximum settlements recorded in the columns by sliding micometers ranged from3mm (54/C) to more than 6mm (54/A), whilst almost 9mm was reached between columns 53/C and 54/C. The plates indicated settlements ranging between 2.4mm (SP11) and 7.5mm (SP15), in good agreement with the sliding micrometer readings.

Thus, it appears that short term compressibility is negligible within the columns and far reduced in the peaty soil between, fully vindicating the choice of the solution. Long term monitoring continues.

Data from the inclinometers indicate no significant horizontal displacements to date.

CONCLUSION

Although special jet grouting systems are reported to have been developed in Japan (3) for the treatment of soft, organic soils, the authors believe that this case history is one of the first commercial applications in the world. The lack of previous practical experience regarding the execution and performance had therefore to be compensated for by executing an exhaustive field trial, and by timely and meticulous controls during and after the main construction phase. The whole program again emphasizes the potential and reliability of the technique of jet grouting when conducted in a systematic and controlled fashion by an experienced contractor.

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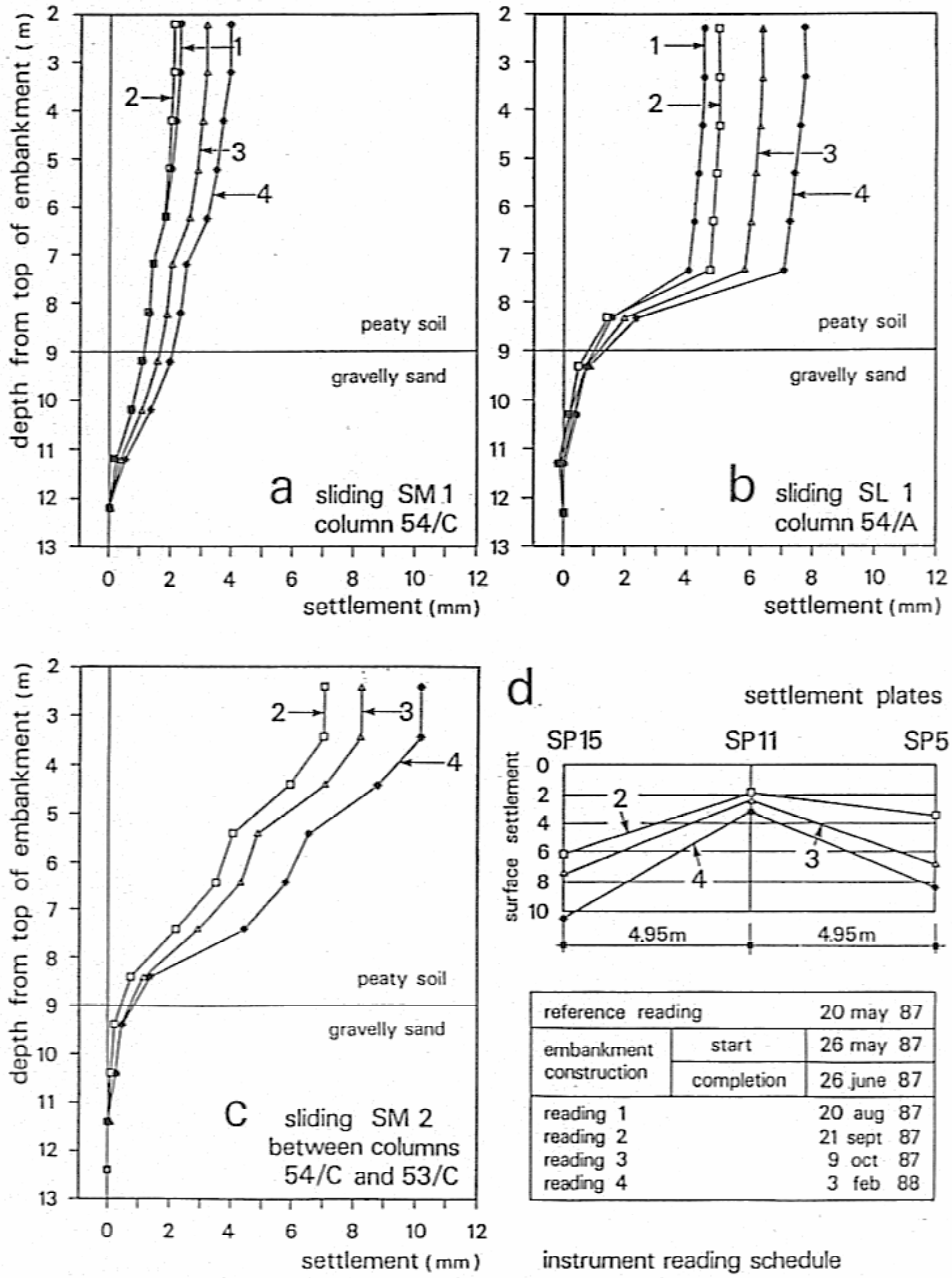

Figure 18. Preliminary results from settlement monitoring.