

Case stories, Harbours - Mass stabilisation of contaminated dredging mud in Sörnäinen, Helsinki

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ABSTRACT: The coastal neighbourhood of Sörnäinen in Helsinki is an old city sector of industrial and harbour area. Nowadays these activities have been replaced by residential building and other city activities, and the infrastructure of the area has been improved after the coastal construction with mass stabilisation in 1998. Many coastal constructions have been carried out by dredging mud from the sea bottom and replacing it with frictional materials (e.g. blasted rock). Dredged mud has been considered unsuitable for any kind of construction purposes and transported to deposit areas. In the Sörnäinen case this relocation of dredged mud was not allowed due to its contamination. Instead, it was decided to reuse the contaminated dredged mud in the coastal structure of the new shoreline of Sörnäinen. For two major reasons it was necessary to stabilise and solidify the mud. Bearing capacity had to be improved in order to avoid stability problems and settlements. Also, any future transport of contaminants to the environment had to be avoided. The chosen method for soil improvement and remediation was mass stabilisation.

1. INTRODUCTION

Mass stabilisation has been carried out mostly in order to improve the geotechnical properties of soft soils, so far. This paper presents a case where the purpose of mass stabilisation is to remediate contaminated sediments in situ, without relocating them to any other place involving potential environmental problems somewhere else.

In addition to the solidification also the stabilisation of contaminated masses is an important and desired effect of mass stabilisation nowadays. Treatment of dredged mud and contaminated soils are both relatively recent applications of the mass stabilisation technology. Both applications are good examples about the use of the method in very bad soil conditions.

In the coast of Helsinki, in Sörnäinen (Figure 1) a new park area was constructed by filling sea within an area of about 30 metres x 150 metres. The filling was initially planned to be made by dredging mud and clay away and replacing them with blasted rock. The dredged mud and clay were planned to be transported to Taulukari relocation area in the sea outside Helsinki. However, in the design phase it was found out that about 1.5 metres of the upper sediment layer was contaminated by heavy metals, PCB and oils. Due to the contamination it was not allowed to relocate the top layer of the mud to Taulukari area. Instead, it was relocated to a basin between the new edge embankment and the previous shore line. Mass stabilisation method was then used both for the soil improvement and the soil remediation.

2. PRINCIPLES OF MASS STABILISATION

Mass stabilisation is a relatively new soil improvement method where dry binder is mixed into peat, mud or soft clay. The procedure is carried out with help of a mixing tool, which has been installed on an excavator machine (Figure 2). Mixing is done both in the horizontal

and the vertical direction so that a hardened soil slab is formed due to the effect of the binder. The mixing tool has been innovated by YIT Ltd in Finland at the beginning of 1990's.

The technology was initially developed for the stabilisation of soft peat layers. As the mass stabilisation technology has developed new fields of application have been innovated, for example the treatment of dredged mud and contaminated soils. Mass stabilisation can be implemented also by mixing binder agents to soils that have been for example excavated or dredged and lifted to the ground or basins constructed by edge embankments. Mass stabilisation improves the properties of the excavated or dredged low-quality masses so that instead of transporting this material to a deposit or relocation area it can be efficiently used for construction purposes: such as road structures, filling, noise barriers, coast banks etc. Mass stabilisation can in many cases replace the conventional method in which the soft soil layers are replaced with frictional materials like gravel or blasted rock.

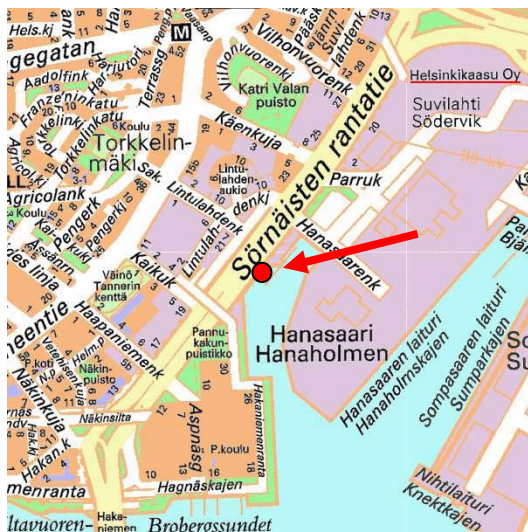


Figure 1. Location of the Sörnäinen mass stabilisation site in Helsinki

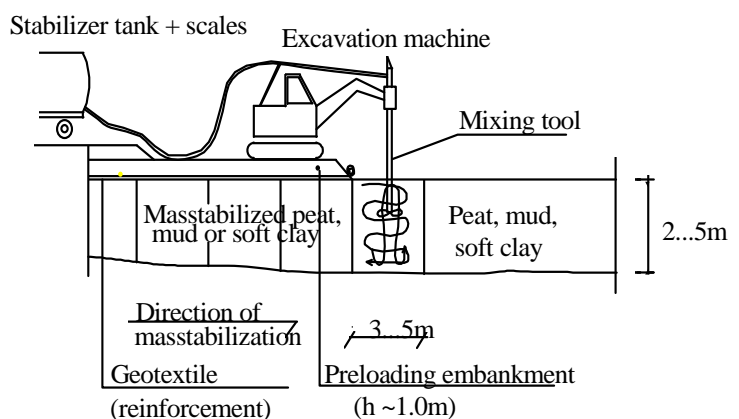


Figure 2. Principle of the mass stabilisation method (EN14679:2005; EuroSoilStab 2002)

3. LABORATORY TESTING BEFORE CONSTRUCTION

The total content of contaminants was determined from the samples taken from dozens of sampling points at Sörnäinen site during 1996...99. A summary of environmental tests is presented in Table 3. It was found out that about 1.5 metres of the upper sediment layer was contaminated by heavy metals, PCB and oils. After finding this the stabilisation test were carried out.

The safety and quality criteria set for the Sörnäinen project required a number of pre-stabilisation tests. Tests were carried out in order to find out the most suitable binders, to optimise the binder amount and to ascertain proper strength-deformation properties for the stabilised sediment. Also, the permeability of the stabilised sediment was determined in order to check the solidification of the contaminated soil. Three different binders were tested using three different binder amounts. The tested binders were Rapid cement, Norkalk FTC and Ekomix 2 (Lohja Rudus Oy) in amounts of 50, 80 and 110 kg/m³. Results from unconfined compression strength (UCS) tests after 14 days strength development are presented in Table 1 (for cement). The permeability test was made with two samples after two months strength development. On the basis of the results rapid cement was chosen as the binder, using an amount of 110 kg/m³.

Table 1. Unconfined compression strength q , strain at failure ε and water permeability k of stabilisation tests of contaminated top layer sediment of coast of Sörnäinen, Helsinki.

Binder	amount (kg/m ³)	q ($2 \times \tau$) (kPa)	ε (%)	k (m/s)
Rapid cement	50	<10	-	-
Rapid cement	80	39	3	22×10^{-9}
Rapid cement	110	93	1,8	-

4. CONSTRUCTION OF THE CITY PARK OF SÖRNÄINEN

After it was found out that the top layer of the sediment was so badly contaminated that it was not allowed to relocate the dredged mud into Taulukari area, the construction plans were changed so that only an edge embankment was built with the conventional replacement method. The contaminated dredged mud was then transported into the basin formed by this embankment and the previous shoreline. The width of the basin was about 17 m, the length about 80 m and the depth 4-5 m. A two meters thick layer of crushed rock was designed to be placed on the top of the dredged mud layer.

Because of a designed short construction time (only about 7 months) there was no time to wait for the natural hardening and consolidation of the mud. Also, there was a need to find out a solution to prevent spreading of the contaminants in the environment. For these reasons the whole dredged mud layer was chosen to be mass stabilised. The principle of the cross section of the intended structure with the preload embankment is presented in Fig. 3.

Dredging was carried out in two phases. At first, the contaminated top layer of the mud, situating in a place of the edge embankment, was dredged and stored into two barges. In the second phase, the not-contaminated clay layer deeper in the sediment was dredged. The edge embankment was constructed simultaneously. The not-contaminated lower clay material was transported to the relocation area of Taulukari. The boundary surface of these two layers was monitored with help of several laboratory tests in beforehand. Laboratory tests were carried out from the clay being transported to Taulukari during the construction phase, as well. Later, when the construction of the edge embankment had been advancing for a while, the contaminated dredged mud was moved into the basin to the top of the underlying contaminated sediments (Figure 4a). A floating mobile silt fence was used during the whole dredging phase to avoid the spreading of contaminants into the nearby sea area.

The stabilisation work proceeded in sections from the shore line side edge of the basin. Directly after the stabilisation work, a one meter thick compaction layer was constructed on the top of the stabilised area (Figure 4b). After one month, when the stabilised mud had been hardened and the controlling soundings had been carried out, the embankment was raised by placing an about one meter thick preload embankment on the top of the compaction layer.

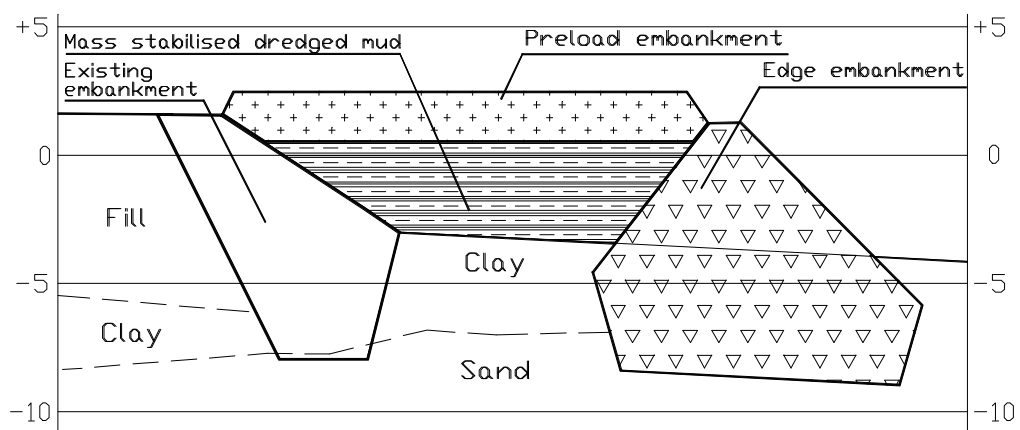


Figure 3. Cross section of the structure in principle, Sörnäinen, Helsinki.

a)



b)



c)



Figure 4. a) Dredging and building of the edge embankment. b) Mass stabilisation work and construction of the compaction layer. c) Preload embankment. Sörnäinen, Helsinki.

After the quality controlling soundings had been done (see chapter 5.1), the preload embankment was constructed for a four months settlement time (Figure 4c). The material of the embankment was chosen in such way that it could be utilized in the final structure. The plans and the timetable of Helsinki city changed during the construction, and the construction time with final structure was not 7 months as was the initial deadline in the design phase. The preload embankment is still after 9 years over the mass stabilisation area, and the final structures have not been constructed.

5. QUALITY CONTROL TESTS, GEOTECHNICAL PROPERTIES

5.1 Column penetrometer and vane penetrometer tests

It was necessary to make the quality control tests after the mass stabilisation work was finished, in order to find out the obtained shear strength and to control that the mixing work was done in due form. In Sörnäinen the quality control tests were made one month after stabilisation. Tests included column penetrometer soundings at 10 points and vane penetrometer soundings at 5 points. In the column penetrometer method a mechanical penetrometer equipped with two vanes ($A = 100 \text{ cm}^2$) is pressed down (without rotation), and the compression strength employed is measured at the upper end of the penetrometer rod (Halkola 1999).

The target value of the shear strength was 30 kPa. However, the soundings showed that the average strength of the stabilised layer was above 120 kPa (minimum value ever measured 40 kPa) with vane test and above 200 kPa with column penetrometer test. This means that the designed (and targeted) shear strength value of 30 kPa was achieved by a wide margin.

5.2 Sampling and testing 2 years after stabilisation

About two years after construction (2001) the undisturbed samples were taken from 3 different points of the mass stabilisation area as 3-4 metres long continuous tube samples. The samples were taken with a simple tube sampler ($d = 65 \text{ mm}$). The meaning was to take undisturbed samples, but there were some disturbances in the samples taken in the hardest parts of the mass stabilisation.

Two piezometers were installed into the sampling holes in the level about -1 to determine the water permeability of the mass stabilised dredged mud in-situ (Table 2). Plate load tests were carried out with a loading plate ($d=300 \text{ mm}$) and a maximal load of 60 kN (Table 2) beside all three sampling points.

The continuous samples were split into 90-100 mm long test samples for the following laboratory tests:

- unconfined compression tests (1-axial)
- frost durability tests (cyclic freezing – thawing tests)
- water permeability tests

The shear strengths in 1-axial compression tests varied between 25...143 kPa, apart from one weaker sample (sampling point 2). The average shear strength was 85 kPa (Table 2). The shear strength measured with the unconfined compression strength test is naturally lower than the shear strength measured with the column penetrometer test or shear vane test in-situ, because no lateral stress affects in the unconfined compression strength test. Also there were some disturbances in the samples taken in the mass stabilisation area. Therefore the column penetrometer and vane shear test represents better the undisturbed shear strength of the mass stabilisation.

The freezing – thawing tests contains 12 freezing and thawing cycles ($8 \text{ h } -18 \text{ }^\circ\text{C} \Rightarrow 8 \text{ h } +20 \text{ }^\circ\text{C} \times 12 \text{ cycles}$), and 1-axial compression strength test after that. In the samples taken about 2 years after the stabilisation ($z = +0.3...0.7 \text{ m}$) the shear strength was in average 9 kPa after the freezing-thawing test (12 cycles). Without freezing-thawing treatment the

shear strength of the parallel samples at the same depth was about 67 kPa. On the basis of these results the strength of mass stabilised mud would decrease by about 86 per cent after freezing-thawing cycles (Table 2).

Water permeability tests were made with a flexible wall permeameter for 2 samples from a level of -2....-2.8 metres. Determined water permeabilities of these samples were $k = 0.088...4.1 \times 10^{-9}$ m/s. These values are clearly lower than the values determined in-situ $k = 26...690 \times 10^{-9}$ m/s (Table 2). The reason for the difference is not known, but it is assumed that the sampling has caused some cracks to the walls of the sampling hole and these cracks were conducting water. The water permeability values obtained in the laboratory are considered more reliable.

Table 2. Quality controlling results, geotechnical properties. Tests 1) before stabilisation, 2) quality controlling soundings 1 month after stabilisation and 3) sampling, laboratory and in-situ tests ≈ 2 years after stabilisation. Sörnäinen, Helsinki.

Parameter	Age of mass stabilisation	In-situ / laboratory sample	Variation of test results	Average of test results	Number of tests	Test method
Shear strength, kPa	14 d	laboratory	-	47	2	1-axial compression test in laboratory
Water permeability, m/s	2 m	laboratory	-	22×10^{-9}	1*	Water permeability in laboratory
Shear strength, kPa	≈ 1 m	in-situ	40...600	> 200	10	Column penetration test in-situ
Shear strength, kPa	≈ 1 m	in-situ	115...207	> 120	5	Vane penetrometer for columns, in-situ
Shear strength, kPa	≈ 2 a	in-situ	25...143	85	7	1-axial compression test in laboratory
E ₅₀ -modulus, MPa	≈ 2 a	in-situ	2...41	16	7	- - -
Water permeability, m/s	≈ 2 a	in-situ	$0.09...4 \times 10^{-9}$	2.1×10^{-9}	2	Flexible wall permeameter in laboratory
Water permeability, m/s	≈ 2 a	in-situ	$26...690 \times 10^{-9}$	358×10^{-9}	2	Piezometer BAT in-situ
Resistant against freezing - thawing, %	≈ 2 a	in-situ	10...20	14**	3	Freezing - thawing test in laboratory
E ₂ -bearing capacity, MPa	≈ 2 a	in-situ	52...123	85	3	Plate load test in-situ

* Amount of Rabid cement 80 kg/m³

** Shear strength of freezing -thawing -handled samples compared to original shear strength of parallel samples

6. ENVIRONMENTAL CONTROL TESTS

Two years after construction (2001) the environmental laboratory tests were carried out for nine drilled samples – tube samples from 3 different points and at 3 different depths. For the environmental tests, a subsample of 0.5 kg was randomly taken from each tube sample. Part of the tube samples were used for the geotechnical tests. Each of the 3 samples from one point's different depths were mixed into one homogenised sample per point. Thus there was 1.5 kg of random sample material from each of the 3 sampling points.

The tests were:

- total concentration of certain metals and oils (measured as hydrocarbons),
- total leaching of certain metals and hydrocarbon with EN 12457-3 test and
- long-term leaching of certain metals with column test (NEN 7343).

The inorganics (As, Cd, Cr, Cu, Ni, Pb, Zn) were analysed by using ICP-MS/AES, and the total hydrocarbons by using GC/MS. Total concentrations were analyzed and CEN-tests were performed on the 3 mixed samples of each sampling point. Column test was made on one homogenised mix of all sample materials.

Results of the environmental laboratory tests are given in Table 3. The results indicate that the hydrocarbon content of the site after mass stabilisation is smaller than in 1998 (before mass stabilisation). The metal concentrations are at same level than the threshold values on clean soil and clearly lower than the lower guideline values for contaminated soil (Decree 214/2007). Oil concentrations are below detection limit. Also leachable fractions of the metals seem to be very small. In this case we can say that the stabilisation and solidification has been quite effective.

Table 3. Environmental follow-up data from samples two years after the stabilisation 2001 (a) and the limit values for total content and leaching and the values before stabilisation 1998 (b). Sörnäinen, Helsinki.

a)

	RESULTS (AVERAGE FROM THE SAMPLES)			
	Concentration	Leaching	Leaching	CEN/Total cont
	mg/kg	CENprEN 12457-3 L/S 10, mg/kg	NEN 7343 L/S 10, mg/kg	
As	6.1	0.02	0.015	0.00328
Cd	0.6	0.0003	0,000	0.0005
Cr	44.9	0.074	0.037	0.00165
Cu	25.5	1.389	0.863	0.05447
Ni	24.7	0.562	0.422	0.02275
Pb	20	0.053	0.001	0.00265
Zn	97.6	0.018	0.072	0.00018
Arom *	< 0.1	<0.1	-	-
Alif **	< 0.1	<0.1	-	-

b)

	Total content; Finnish guidelines (Decree 214/2007)		Leaching; Finnish guidelines (Finnra 2007)		Total content in 1998 mg/kg TS
	Threshold value mg/kg	Lower/higher limit mg/kg	Group 1 mg/kg	Group 2 mg/kg	
As	5	50/100	0.141	0.85	1.4 ... 8.9
Cd	1	10/20	0.011	0.015	<1 ... 31
Cr	100	200/300	2	5.1	32 ...120
Cu	100	150/200	1.1	2	48 ...220
Ni	50	100/150	1.2	2.1	23 ...70
Pb	60	200/750	1	1.8	13 ...280
Zn	200	250/400	1.5	2.7	140 ...1200
Arom *	< 0.1 yl.	< 1 yl	Group 1: no pavement required		<50 ...5600
Alif **	< 0.1 yl.	< 1 yl.	Group 2: pavement required		

* Aromatic hydrocarbons

** Alifatic or cyclic hydrocarbons

7. CONCLUSIONS

The measured shear strength was about four times higher than the targeted shear strength. Therefore, it might have been possible to reduce the binder amount. On the other hand, the stabilisation tests did not encourage this. One reason for the obtained exceptionally good bearing capacities is probably that the dredged mud had to be mixed with some clay from the deeper soil layers in order to have the surface of the stabilised area above the sea water level. The stabilised materials was not similar to the material used for the laboratory tests.

Rapid cement was used as a binder on the basis of the stabilisation test results of the laboratory. New binder agents like admixtures of cement with industrial by-products are often more cost effective than the traditionally used pure cement and lime-cement admixtures. The new binders have also made it possible to stabilise organic soils. Active R&D to develop even more effective binders as well as mixing tools creates new applications and improves the competitiveness of this environmentally friendly mass stabilisation technology.

The treatment of dredged mud is a good example about the application of mass stabilisation in an efficient way as the method can be used for the improving of the properties of low-quality soil masses and for the binding of contaminants in these masses. Previously, the contaminated masses had to be replaced. Because of the huge lack of the deposit and relocation areas the future of mass stabilisation method seems to be very encouraging.

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