Intended for
Eesti Energia

Document type
Materials action Pilot report including descriptions of applications
For Materials Action see another report

Date
January 2015

LIFE+ 09/ENV/227 OSAMAT
APPLICATIONS, PILOTING AND VERIFICATION ACTIONS NARVA-MUSTAJÕE PILOT REPORT

RAMBOLL

Revision **0.6**
Date **2015/01/14**

Ref **LIFE+ 09/ENV/227 OSAMAT**

Revisions

Revision	Date	Made by	Checked by	Description
0.1	01.03.2012	Olli Kiviniemi	Andres Brakmann	2011 construction works at Narva-Mustajõe
0.2	09.11.2012 21.02.2013	Olli Kiviniemi and Marjo Ronkainen	Pentti Lahtinen	Updated with 2012 construction works and follow-up studies at Narva-Mustajõe
0.3	25.09.2013	Marjo Ronkainen	Ain Kendra	Updated after comments in meeting 23.8.2013
0.4	09.01.2014	Ain Kendra	Liis Tikerpuu	Updated based on comments of experts from Tallinn University of Technology
0.5	17.11.2014	Andres Brakmann	Hendrik Puhkim	Updated based on follow-up study in autumn 2014
0.6	14.01.2015	Marjo Ronkainen, Andres Brakmann, Ain Kendra, Hendrik Puhkim	Hendrik Puhkim	Updated based on Client's comments received on 11.12.2014

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CONTENTS

ABBREVIATIONS

AB – asphalt beton; AC – asphalt concrete
BOTT – Bottom ash, retrieved from the bottom of burning chamber
CC – Composite Cement (also: KS - Komposiittsement)
CYCL – oil shale ash from the cyclone filters, pulverized firing
EE– Eesti Energia
EF – electrostatic precipitator oil shale ash from pulverised firing
EF CFB- electrostatic precipitator ash from circulating fluidised bed combustion

FWD – Falling Weight Deflectometer (to measure pavement bearing capacity)
MAC – Milled Asphalt Concrete
MWA – Mining Waste Aggregate
Niton- Portable x-ray Fluorescence Analyzer
OSA – Oil Shale Ash
RAKLI – Finnish Association of Building Owners and Construction Clients
RAKLI-SKOL-ATL – now renamed: RALA-certification – quality management system for construction and consultancy business
saGr- Sandy gravel
SKOL – Finnish Association of Consulting Firms (Suunnittelu- ja Konsultitoimistojen Liitto)
SRC- Sulphate Resistant Cement
Troxler - Portable x-ray based compaction and moisture measuring device
UCS – Unconfined Compressive Strength

ABSTRACT

The Project contains testing of two different technologies of using OSA (Oil Shale Ash) in road construction – layer-stabilisation with coarse materials on road base and mass-stabilisation of peat on weak surfaces before road construction. Layer-stabilisation process is being tested on Narva-Mustajõe road (issues of current report) and mass-stabilisation on Simuna-Vaiatu road (covered in separate reports).

Current report contains overview of layer-stabilisation tests within the OSAMAT project. Description of mass-stabilisation technology and process can be found in separate report within the same project. As the properties of old ashes, stored in landfill, are varying too much, project partners have decided to test technology only with different fresh ashes coming from energy production process.

Aggregate materials and binders were selected and tested in laboratory (Ramboll Finland). Three different binder combinations were selected for further road tests.

Layer-stabilisation was provided partially in 2011, continued and finished in 2012. There were difficulties in distribution of ashes (ash mixes). Some pavement defects were identified in one location, but in general, pavement had excellent bearing capacity.

Map 1. Location of Narva-Mustajõe test site

1. INTRODUCTION

OSAMAT project has come into life on the firm belief of the beneficiaries that the eventual environmental problems connected to the recycling of OSA (Oil Shale Ash) for civil engineering purposes can be overcome by exploiting the technical properties of the OSA to transform it into strong and impermeable materials in favourable conditions. OSA can be used in high volumes in the local construction markets, and for the wider European market OSA will become an interesting and cost-effective replacement of cement and other expensive commercial additives. OSA is a by-product or waste; the generation of OSA is not allocated to consume energy and generate airborne releases of greenhouse gases. This is a very significant aspect when promoting the utilisation of OSA in the European markets. In general the project will address the challenges of the European policies and legislation, concerning waste, and promote waste recovery, sustainable recycling with a focus on life-cycle thinking, and the development of recycling markets. Conversion of OSA into valuable civil engineering material for different purposes will be an encouraging model for European societies, to recycle corresponding types of industrial waste materials in civil-engineering applications.

According to the original project plan, one of the main parts in OSAMAT project was the demonstrations of practical implementation of three types of civil-engineering applications with materials based on OSA: stabilisation of existing road base course, mass stabilisation of peat e.g. for road and housing foundations, and construction of new base course based on mixtures of oil-shale mining waste and OSA. This report covers the design and construction of Narva-Mustajõe demonstration site, both of which were performed within actions Applications and Piloting. Objective of this pilot construction was to demonstrate utilisation of oil shale ash and aggregate produced from oil shale mining waste, thus to carry out two of the three piloting actions on one site.

While original plans included testing both fresh and stored ashes, during the initial phase of project decision was made to exclude further testing of externally stored ashes as the quality of the product is unstable and therefore not suitable for stabilisation as binding material.

We have in Ramboll the quality management system (by Rakli SKOL in Finland and ISO 9001 in Estonia) in use and all the equipment is calibrated in accordance with it. When we have used a subcontractor (for example drilling, bearing capacity etc.) we expect that they have a quality management system too.

The project flow-chart is presented in Figure 3-1 (page 10).

The site design was carried out in co-operation with Ramboll Estonia AS and Ramboll Finland Oy. Contractor’s tasks were handled by Nordecon AS. Designing of the pilot construction was performed during spring and summer 2011 and the construction was carried out partly in September 2011 and partly in October 2012.

2. SITE INFORMATION

Road nr 13109 Narva-Mustajõe is located in Ida-Viru County in Eastern Estonia. It is the main connecting road between Eesti Energia AS two facilities, Eesti and Balti Oil Shale Combustion Power Plants. The section that was improved in the pilot is located close to the junction of roads 13109 and 13145, approximately 8 km south of Estonian national road number 1 (Figure 2-1). The section chosen for the pilot construction is marked in the map with red colour.

Figure 2-1. The location of the pilot section 14.0-16.1 km of road 13109 (red on map)

The whole section between the junction of roads 13109 and 13145 and Balti power plant had an asphalt concrete surface before the improvement works. The asphalt concrete coating was determined to be in satisfactory condition, while considerably uneven in terms of longitudinal roughness. The reason for this roughness was found out when the old paving was milled out and an old cracked stabilised ash/concrete layer was found below (Figure 2-2). The stabilised ash layer had quite constant horizontal cracks, and the adjacent blocks seemed to have moved against each other in time so that the surface of the concrete layer was not even.

Figure 2-2. The concrete layer found below the pavement

Traffic volumes were measured in 2009 (Table 2-1). Traffic volumes have been considerably low, but a large percentage of the traffic is formed by mid-heavyweight and heavyweight vehicles, which apply the heaviest load to the road structure. It is forecasted that traffic volume will rise to 1 332 vehicles in year 2032.

Table 2-1. Traffic volume on road no 13109 Narva-Mustajõe, measured in 2009

Direction	Lightweight*	Mid-heavyweight*	Heavyweight*
Narva-Mustajõe	354	33	86
Mustajõe-Narva	352	37	99
Total, one day	706	70	185
Percentage	73.5	7.3	19.2

**Lightweight: bikes, cars and vans, mid-heavyweight: buses and lorries, heavyweight: articulated lorries*

The design value for the required load-bearing capacity was determined from the traffic volumes according to the Estonian regulations. The design traffic load was calculated to be 844 10-ton standard axles per day, which resulted in 259.1 MPa design load-bearing capacity.

The load-bearing capacities were measured within the whole stretch in 2002 with FWD measurements. The average of the measured load-bearing capacity value was 420 MPa and all measurements except the section 14.4–14.5 km yielded load-bearing capacity values above 300 MPa. The lowest reading in the section 14.4–14.5 km was 253 MPa, which is slightly below the minimum design value 259.1 MPa. According to the measurements, the load-bearing capacity was at adequate level for the road's normal usage. Based on the FWD measurements, there were no problems with the structural layer of the road. However, there were some problems with the subsoil in 1 km length, which was mostly located in section 14.2–14.8 km. The section 14.0–16.1 was thus chosen as the pilot site as it would benefit the most from the used methods. The problems in the road qualitative parameters were related to the surface smoothness and defects, not to bearing capacity.

Estimate of the longitudinal structure based on the preliminary investigations is presented in Figure 2-3. As this figure shows, the structure below the asphalt concrete (AB) and stabilised ash/concrete (Stab. ash) layers varies a lot. The sandy gravel (saGr) layer below them was degraded over time and contained a considerably high portion of fine particles and hence was slightly frost-susceptible. It is, however, possible that degradation of gravel layer is result of leaching from stabilized ash layer.

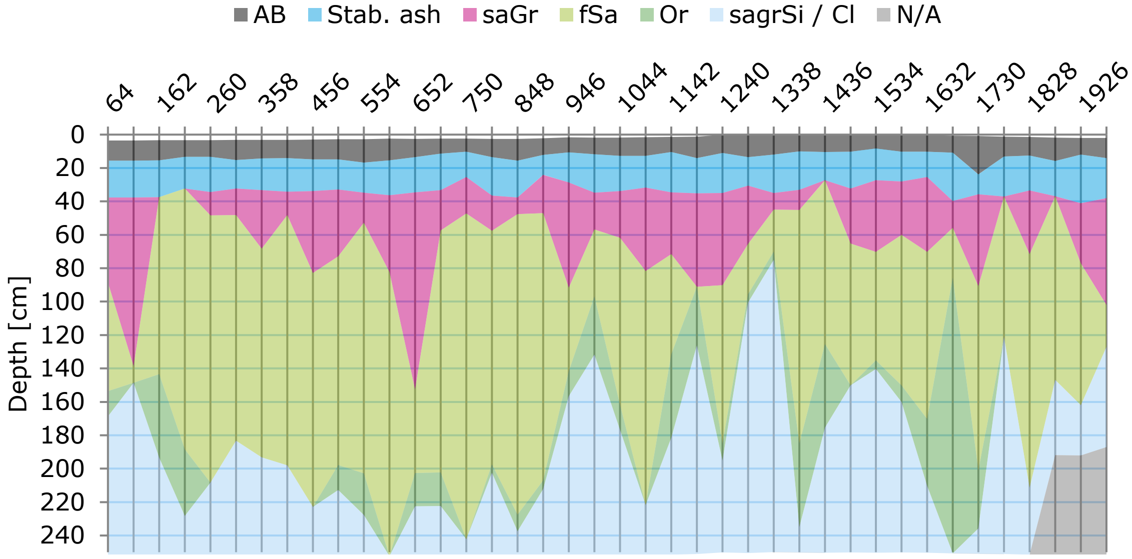

Figure 2-3. Longitudinal structure of the section before repairs. Horizontal: distance from STA 0 (located in the 16.1 km mark of road 13109)

3. DESIGN PRINCIPLES

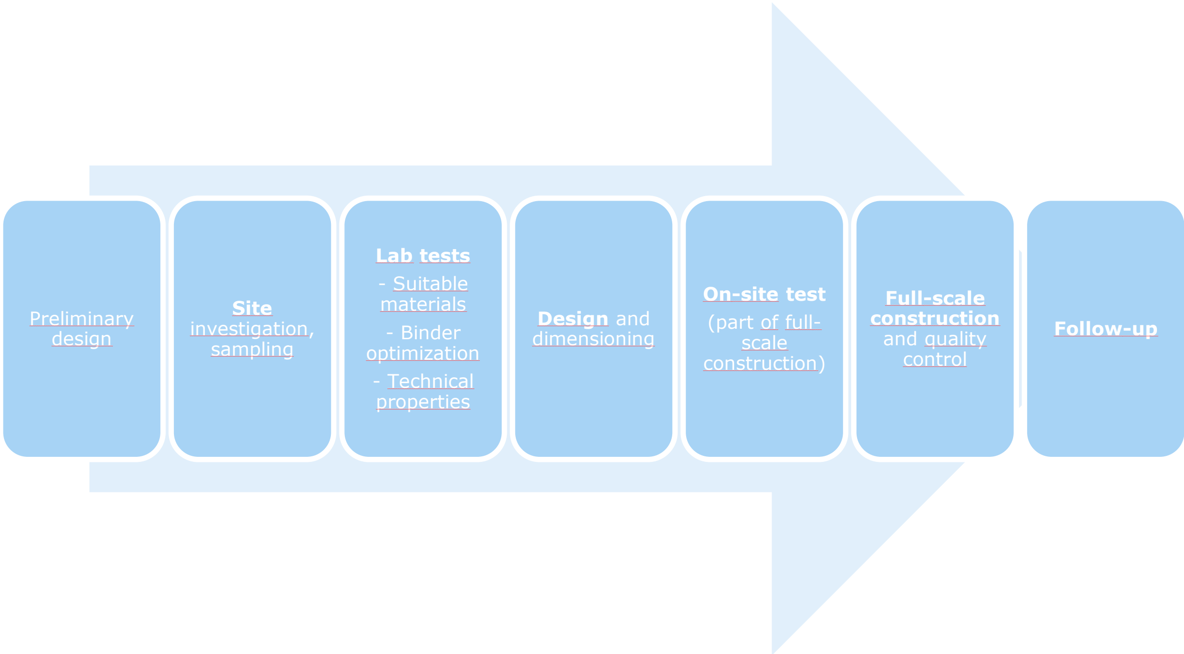

Figure 3-1. Flow-chart of test process

Flow chart (Figure 3-1) describes all steps of the Project from designing principles (preliminary design) to follow-up studies. It describes how the stabilisation project in a road construction takes place. As the properties of ashes and aggregates may differ, in every case the mix recipes have to be prepared based on laboratory tests with exact materials, available for particular project.

3.1. OSAMAT pilot construction plans

Site

Sites for implementation of three methods (new ashes; old ashes and peat stabilisation) were allocated in early 2011 and design of the sites commenced after that. The first two methods were suggested to be implemented in Narva-Mustajõe road and the third one in Simuna-Vaiatu road during the year 2011. However the latter site was postponed to year 2012.

Map 2. Map of Narva-Mustajõe test site

Methods

As the project commenced, three different methods of how to utilise oil shale ash and mining waste were planned to be used; namely

- in-situ stabilisation of existing aggregate road base with oil shale fly ash based binding agent,
- replacement of existing road base with a mixture of oil shale mining waste rock aggregate and oil shale ash and
- mass stabilisation of peat underneath a road structure with oil shale ash based binding agent.

Initial plans to do in-situ stabilisation of existing road base and replacement of existing road base with oil shale mining waste aggregate were altered after the preliminary site investigations. It was found that the road base within the stabilisation depth below thick layer of asphalt concrete consisted of concrete-like stabilised ash. The road base stabilisation was planned through milling of old asphalt concrete and mixing with aggregate road base below stabilizing the mix with ashes. As the actual rock aggregate layer was located deep below the surface, it was neither technically nor economically feasible to remove the old stabilized ash layer (Figure 2-3).

Therefore, in Narva-Mustajõe site, it was agreed that both initially foreseen methods would be applied together – a new road base constructed with oil shale mining waste based aggregates and stabilised with oil shale ash based binding agent.

Regulations

There are following regulations in Finland, which may be applied on stabilisation process:

- Stabilointiohje[1] (for testing; Tiehallinto 2002), design instruction concerning stabilisation with bitum, cement and mixes;
- Päällysrakenteen stabilointi[2] (regular; Tiehallinto 2007), design instruction concerning stabilisation with bitum, cement and mixes;
- Syvästabiloinnin suunnittelu[3] (Liikennevirasto 2010), design instruction for mass-stabilisation;
- Tuhkarakentamisen käsikirja[4] (Ramboll 2010), manual covering use of ashes from burning process of coal, communal waste, paper processing waste, water purifying process waste and biomasses;

Finnish Road Administration has released requirements on stabilisation of base layer in 2007[2]. According to instruction, in general, cement content should be at 2-5%. It must be mentioned, that Finnish experience is based only on igneous rocks, while Estonian experience is based on limestone aggregate. Finnish ashes typically originate from coal burning or forest industry (bio fuel – bark, wood, peat or paper industry waste). Compared to Finnish ashes, OSA is more reactive, containing more CaO and thus should be more suitable as binder.

Estonian Road Administration has released instruction on construction of stabilized base layers, on 2006. It is mostly based on Wirtgen manual, but within the concentration process essential aspects have been lost.

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[1] Stabilointiohje. Tiehallinto 2001: http://alk.tiehallinto.fi/thohje/pdf/2100009-02.pdf

[2] Päällysrakenteen stabilointi. Tiehallinto 2007:
http://alk.tiehallinto.fi/thohje/pdf/2100055-v-07paallysrakenteen_stabilointi.pdf

[3] Syvästabiloinnin suunnittelu. Liikennevirasto 2010:
http://www2.liikennevirasto.fi/julkaisut/pdf3/lo_2010-11_syvastabiloinnin_suunnittelu_web.pdf

[4] Tuhkarakentamisen käsikirja, Ramboll 2010: http://www.infrary.fi/files/3985_Tuhkarakentamisen_kasikirja.pdf

- Stabiliseeritud katendikihtide ehitamise juhis[5] (Maanteeamet 2005), construction regulation for stabilized pavement layers, covers stabilisation with cement, bitum and mixes (9 pages).
- Wirtgen külmstabiliseerimise käsiraamat[6] (Maanteeamet 2004), translation of german manual (267 pages).

According to the regulation, recommended content of road cement or Portland cement in cement stabilisation is at 2-3%. Despite currently there are no valid regulations for ash-stabilisation, former regulations specified ash content of 4-5% in stabilized material. In wider scale, it can be concluded that ash as binder is twice weaker than cement to achieve comparable results. Estonian Road Administration intends to replace the current instruction to updated release within 2014.

Earlier experience

Fly ash has been used in road construction since 1961 in Estonia. From 1971-1986 about 100 – 120 000 tons of oil shale fly ash was used to stabilize road pavements, altogether ca 1000 km of stabilized pavements were built in Estonia. TECER (Teede Tehnokeskus) has studied the experience in 2005[7]. Test samples (after 35 years of use) indicated UCS of 20-34 MPa with average of 25.3 MPa resulting with all negative aspects of concrete pavements (built without joints); also there were difficulties in adhesion of stabilized layers with bitum-based top layers.

Ramboll Estonia has studied possibilities of mass-stabilisation of soft grounds[8] together with Ramboll Finland and continued the study with test site in Kose-Mäo section of Tallinn-Tartu road in 2007-2011[9]. Experience of this small site (900 m^2) has laid basis for mass-stabilisation tests within OSAMAT project.

3.2. Laboratory tests and used binding agent compositions

An extensive set of laboratory tests was executed to assess the feasibility of different oil shale ash and mining waste fractions in the suggested methods. The whole laboratory study is explained in detail in separate report (OSAMAT Materials action report, November 2012[10]). The results and conclusions related to Narva-Mustajõe pilot are referred in this report. As agreed with EE, standard crushed limestone samples were included in the comparison to have the baseline on the quality materials (altogether 11 different samples, including 5 of them directly from shale mining sites.

Laboratory test plans were designed according to the initial project plans, and prior to allocation of the demonstration sites, to assess feasibility of different oil shale ash fractions as road base stabilisation binding agents. The feasibility in stabilisation was tested with unconfined compressive strength and freeze-thaw weathering tests on several stabilised mining waste based aggregates and gravel samples taken from Narva-Mustajõe road structure. Thus the tests also gave information on the feasibility of different mining waste aggregates in road construction. The samples from asphalt layer were not included because asphalt has not been milled during that time yet. All materials were also tested with geotechnical index tests (loss on ignition, initial water content, pH, particle size distribution and compaction parameters). All laboratory tests and their results are published in OSAMAT Materials action report.

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[5] Stabiliseeritud katendikihtide ehitamise juhis. Maanteeamet 2005 (2): http://www.mnt.ee/failid/juhised/stabiliseeritud_katendikihtide_ehitamise_juhis.pdf

[6] Wirtgen külmstabiliseerimise käsiraamat. Maanteeamet 2004: http://www.mnt.ee/failid/uuringud/wirtgen.pdf

[7] AS Teede Tehnokeskus. 2005 (56): http://www.mnt.ee/failid/uuringud/tuhkkatted.pdf

[8] Pinnase mass-stabiliseerimisvõimaluste uuring. Ramboll Eesti AS 2007 (4): http://www.mnt.ee/failid/uuringud/2007-4_Mass-stabiliseerimise_voimalused_aruanne.pdf

[9] Pinnaste mass-stabiliseerimisvõimaluste katselõigu uurimistöö. Ramboll Eesti AS 2007 (4): http://www.mnt.ee/public/uuringud/2008_Pinnaste_mass-stabiliseerimise_katseloik_110427.pdf

[10] Material report. LIFE09 ENV/EE/000227 OSAMAT intermediate report of materials action. Ramboll, August 2012

Aggregate selection

All aggregates used in the stabilisation tests are listed in **Error! Reference source not found.**. Narva-Mustajõe sample was taken from the first aggregate layer from the top. All the other samples were provided from oil shale mining sites located around Estonia. Tondi-Väo and Koigi samples were pre-crushed to the grain sizes listed in **Error! Reference source not found.**, whereas Aidu sample had to be crushed in laboratory and hence used only in comparative tests. Tondi-Väo material was considerably harder and more rock-aggregate-like than the softer material from Koigi. Aidu material was considered to be somewhere between those two.

Table 3-1. Properties of tested aggregates

Sample	w* [%]	Soil type	Notification
Narva-Mustajõe		grsiSa	Sample from the road
Tondi-Väo 0-4 mm Tondi-Väo 4-16 mm	7 2.3	grSa saGr	1:1 mixture in stabilisation tests
Koigi 0-8 mm Koigi 8-16 mm	7.5 5.1	grsiSa Gr	1:1 mixture in stabilisation tests
Koigi 0-32 mm		saGr	
Aidu 0-20 mm		saGr	Originally 0-70 mm grain size

**w = water content*

For stabilisation, four different aggregates were selected:

- Tondi-Väo samples were mixed together (1:1);
- Koigi samples 0-8 mm and 8-16 mm were mixed together (1:1);
- Narva-Mustajõe samples from road;
- Aidu 0-20 mm (crushed from original of 0-70 mm).

Binder selection

Binding agents used in the stabilisation tests are listed in Table 3-. The table includes two fractions of ashes collected from electric filters, one fraction from cyclone and one bottom ash. The table also includes the cements used in stabilisation tests.

Table 3-2. Properties of binding agents*

Sample	Explanation	w [%]	LOI [%]	Soil type	pH
EF PF	Oil shale ash, electric filter, Block 3, Pulverized firing (Silo 1)	0.4	3.4	Si	13.0
EF CFB	Oil shale ash, electric filter, Block 8, Circulated Fluidized Bed combustion (field 1+2+3+4)	0.2	3.4	Si	13.0
OSA BOTT EF CFB	Oil shale ash, bottom ash, Block 8, Circulated Fluidized Bed combustion	0	11.8	grSa	12.9
OSA CYCL.	Oil shale ash, cyclone ash, pulverized firing		1.0	saSi	13.0
CC, Komp. CEM	Composite Cement (CEM II /B-M(T-L) 42,5 R)	-	-	-	-
SRC	Sulphate resistant cement (CEM I 42.5 N)	-	-	-	-

**During construction in addition Block 11 ash quality was used, but its characteristics are equal with EF CFB, Block 8.*

In approach to find an adequate binding agent composition, while the overall yielded unconfined compressive strength (UCS) is regarded important, the main emphasis is put on the difference between the 28 day UCS result and the corresponding 28 day UCS test result after the freeze-thaw weathering test (FT). Below 30 % strength drop after the 12 freeze-thaw cycles of the FT is considered to be the safe limit in stabilisations intended for road base in Finland2. If the difference is much larger than that, the probability that the stabilised structure loses its strength completely before its service life is up rises considerably.

The 28 day UCS and freeze-thaw weathering test results of the stabilisation tests are presented

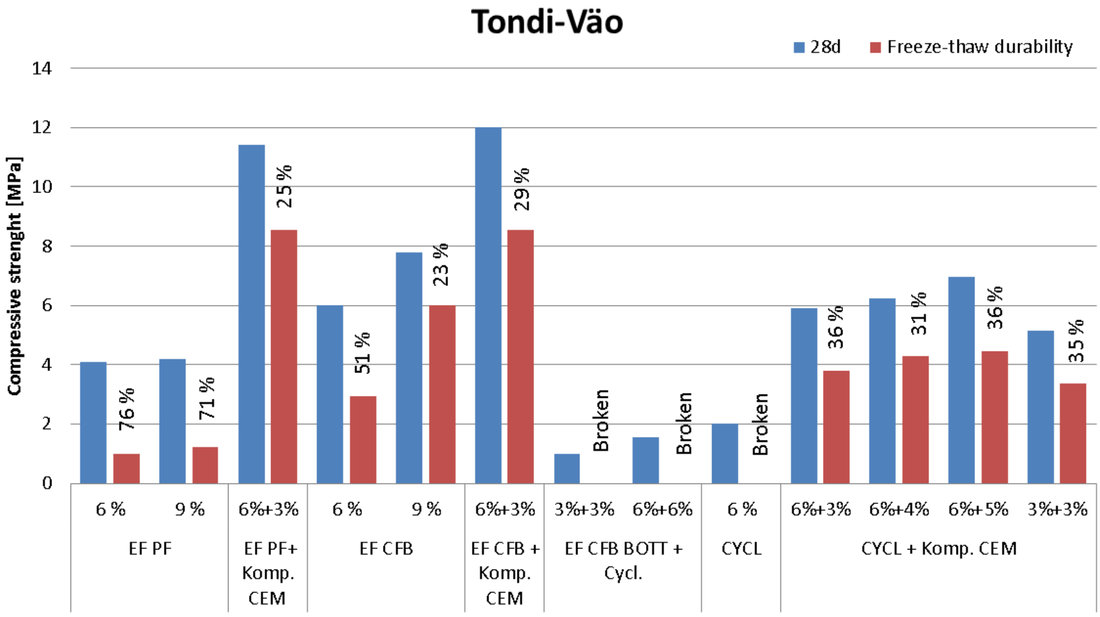

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Figure 3-2,

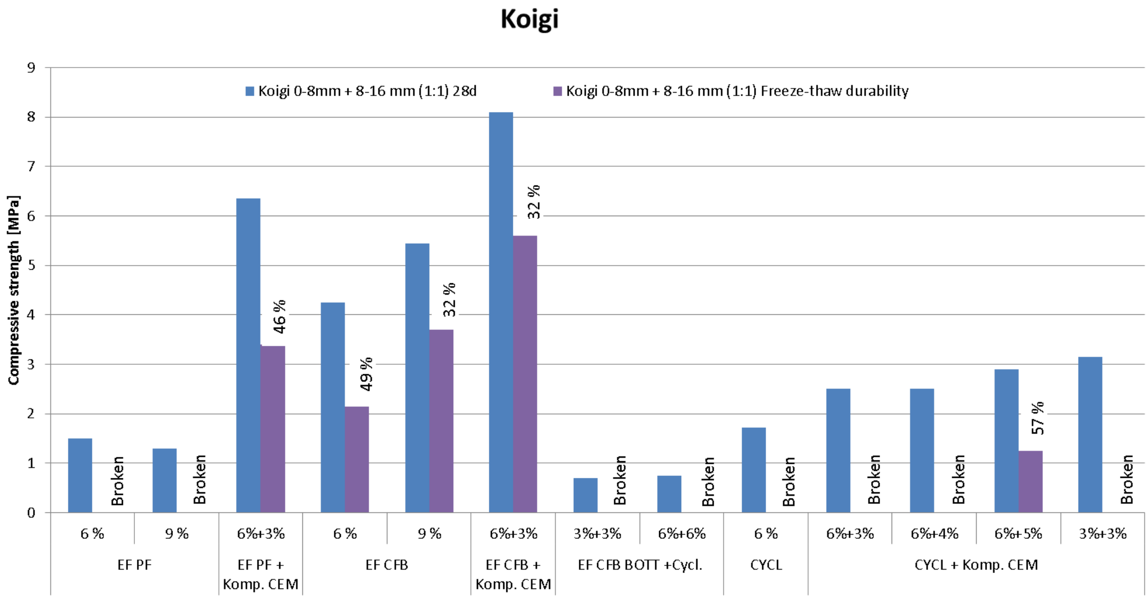

Figure 3-3,

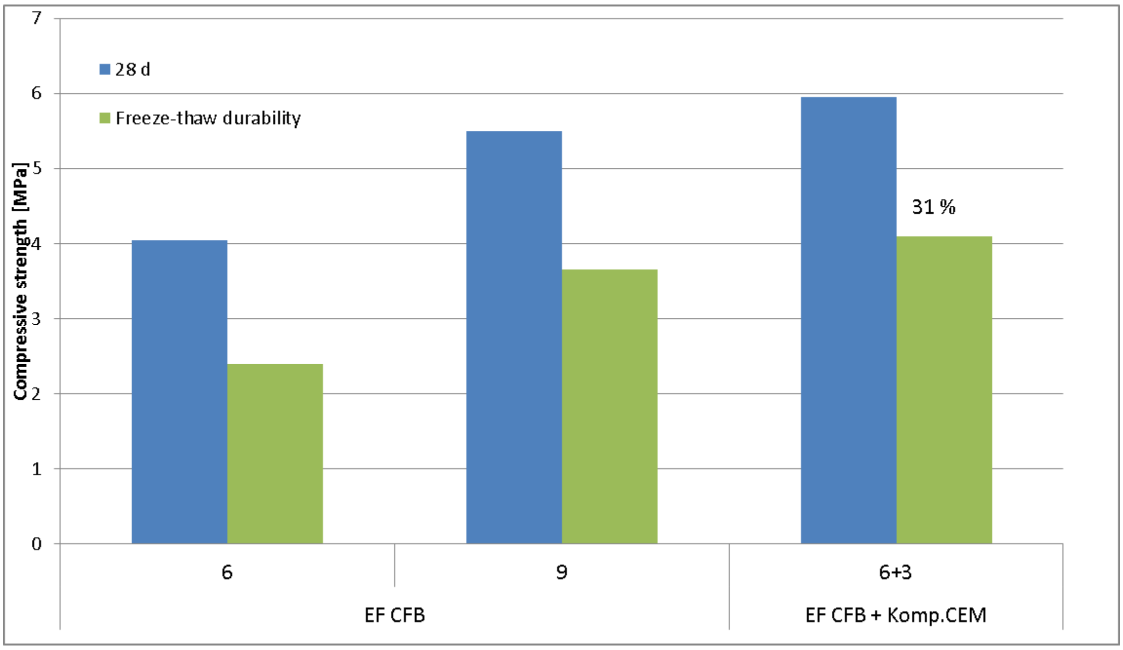

Figure 3-4,

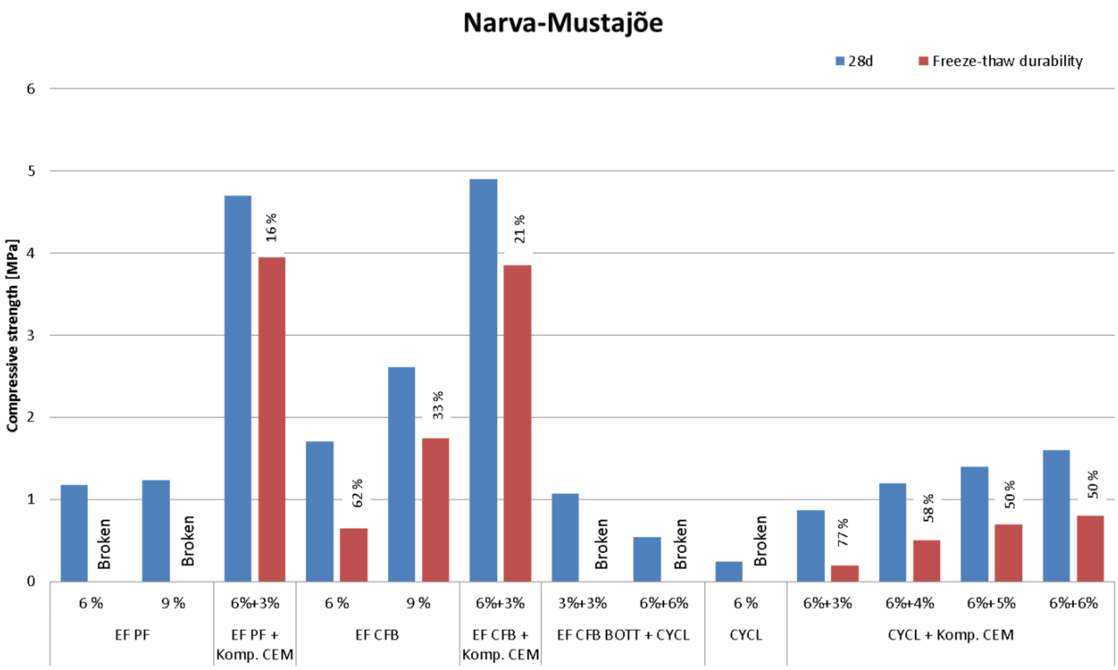

Figure 3-5.

OSA EF CFB ash worked better than the other ashes. Using 9% of it without any cement addition yielded good results. However, the strength drop after FT was below 30% only with Tondi-Väo aggregate. Still, using ash-only binding agent would be more favourable, a slight cement addition would give reasonably safer results.

OSA CYCL yielded very mixed results. With the Tondi-Väo aggregate, the results were quite usable, but with the Koigi aggregate, only the samples with 6% ash and 5% cement withstood the FT, and still with considerably high strength loss. The results with Narva-Mustajõe aggregate were also weak.

OSA BOTT EF CFB was compared only to the OSA CYCL, and it yielded comparably less strength.

In the end three different compositions were chosen for the demonstration purposes:

- 9% of OSA EF CFB. While the composition was not completely satisfactory in terms of freeze-thaw durability, it was decided to try one composition without cement as it will very likely yield excellent results with proper aggregate.
- 6% of OSA EF PF & 3% of CC. OSA EF PF ash didn't succeed very on its own, but worked reasonably well with cement addition. Lower cement amount would have been sufficient, but as it wasn't tested, the tested amount of 3% was used.
- 5% OSA CYCL & 5% of CC. Due to very mixed results, a considerably higher amount of cement was decided to be used.

Following graphs indicate the UCS before and after freezing test; percentage indicates strength loss during freezing test:

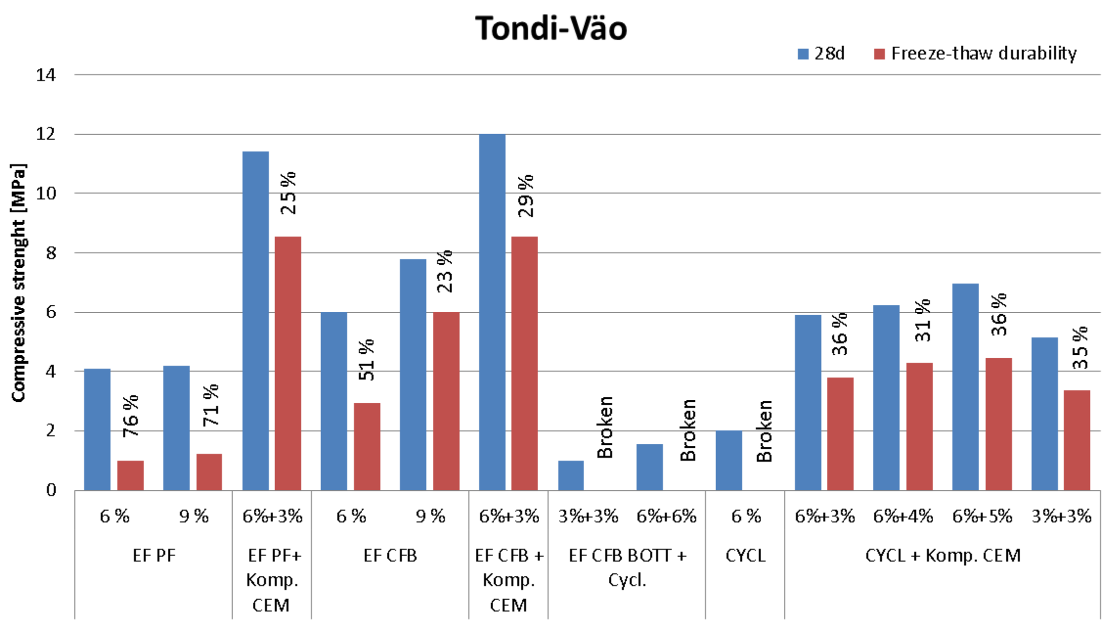

Figure 3-2. Stabilisation test results for Tondi-Väo aggregate

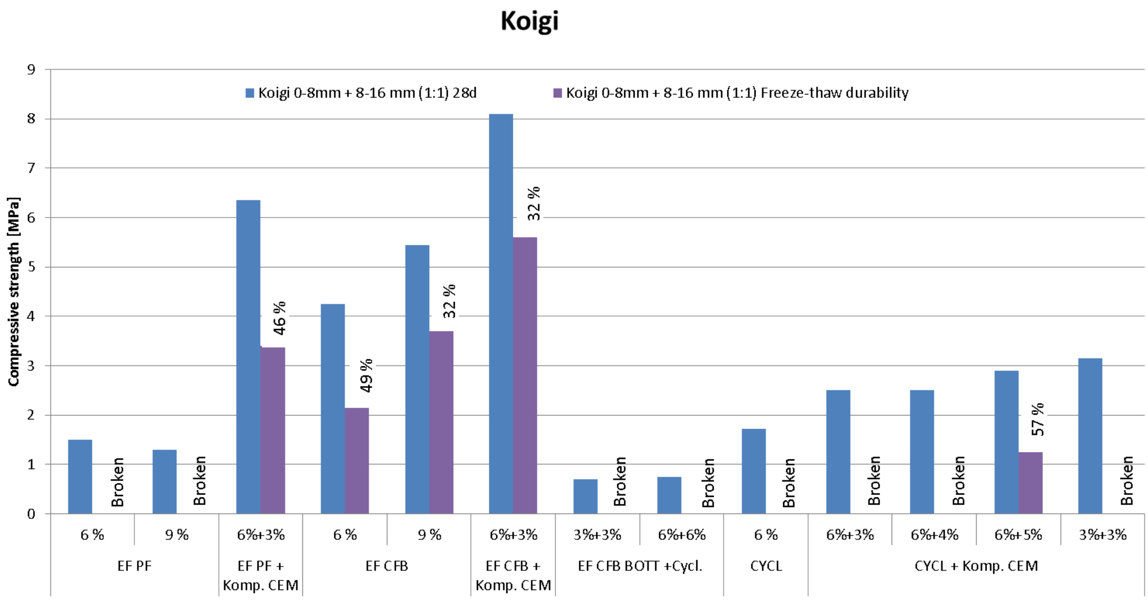

Figure 3-3. Stabilisation test results for Koigi aggregate

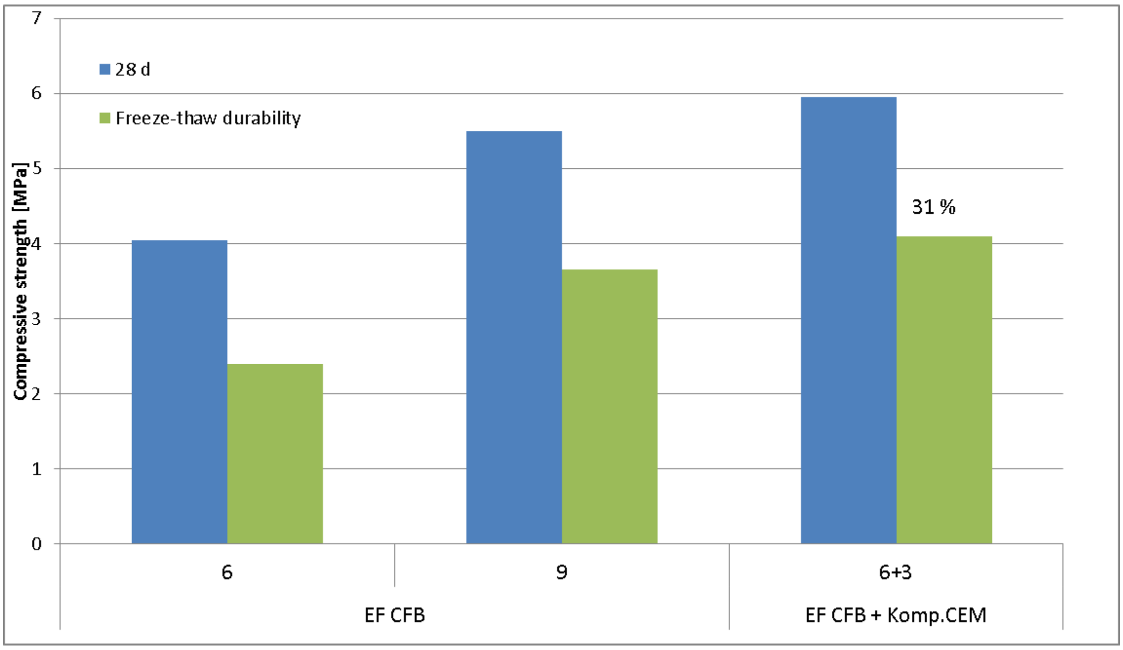

Figure 3-4. Stabilisation test results for Aidu mining waste (aggregate)

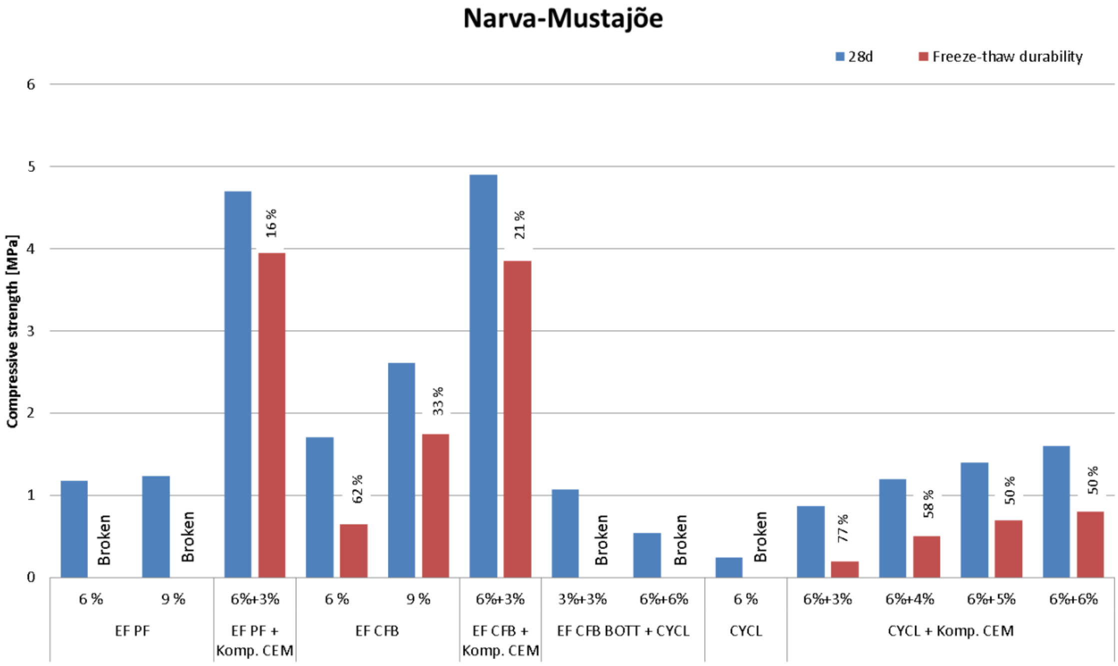

Figure 3-5. Stabilisation test results of Narva-Mustajõe road material

3.3. Detailed construction design

The detailed construction designs for both test site locations including layer and mass-stabilisation were made in co-operation with Ramboll Finland and Ramboll Estonia. The contractor was provided a separate specifications document written in Estonian. The essential parts of the original document are referred in this chapter.

Road base stabilisation

Three sections with stabilisation were designed to be constructed, each of them with different oil shale fly ash fraction as a binding agent (Table 3-). The three different sections were designed as the provider of the fly ash, Eesti Energia, requested experimentation with at least three different fractions.

The binding agent compositions listed in Table 3- were chosen according to the laboratory test results (Chapter 3.2). The OSA EF CFB ash provided the notably better results than the other two ashes; it was sufficient on its own. The other two ashes needed addition of cement to withstand the freeze-thaw cycles.

Table 3-3. Stabilisation sections and related binding agents

Section	Length	Procedure
0+50 –5+00	450 m	Layer stabilisation, 25 cm New base aggregate: MWA + MAC EF PF 6 % + CC 3 % Old structure (old ash stabilised layer not removed)
5+00 -9+50	450 m	Layer stabilisation, 25 cm New base aggregate: MWA + MAC CYCL 5 % + CC 5 % Old structure (old ash stabilised layer not removed)
9+50 -10+50	100 m	Layer stabilisation, 35 cm Base aggregate: MWA + MAC

Section	Length	Procedure
		EF PF 6% + CC 3% Old structure (old ash stabilised layer was removed)
10+50 -11+50	100 m	Layer stabilisation, 35 cm EF CFB 9 % Old structure (old ash stabilised layer was removed)
11+50 -12+50	100 m	Layer stabilisation, 35 cm CYCL 5% + CC 5% Old structure (old ash stabilised layer was removed)
12+50 -15+80	330 m	Layer stabilisation, 25 cm EF PF 6% + CC 3% Old structure (old ash stabilised layer was not removed)
15+80 -16+80	100 m	Layer stabilisation, 25 cm (EF CFB 9%) Old structure (old ash stabilised layer was not removed)

Aggregate layer for stabilisation

The aggregate layer for stabilisation was designed to be constructed with 16–32 mm oil shale mining waste aggregate (MWA) and milled asphalt concrete (MAC). Original idea was to construct the whole layer with MWA, but it wasn't possible to get aggregate with proper grain size distribution (See guidelines in Figure 3-6). The most feasible alternative was to combine coarse grained MWA (Figure 3-7) with fine grained MAC (Figure 3-8) to create an aggregate with sufficient grain size distribution for stabilisation purposes. The estimated grain size distribution (grading curve) of the combined aggregate layer is compared to the Estonian[5] and Finnish[2] guidelines for cement based stabilisations in Figure 3-6. Grading curve indicates the estimates for MAC and MWA compared with regulations. The estimated grain size distribution is a little bit coarser than the ones proposed in the guidelines. However, according to the past experiences in Finland, it is possible to use aggregates with grain size distribution out of the guideline boundaries and still have good results. It means that after stabilisation mixing the binder and aggregate are together the bound structure. Used aggregate (MWA) is softer than Finnish gravel and also the binder content should be higher. On the other hand, the soft aggregate will be milled to finer grain size, the binder can influence more effectively. In addition, it has to be noted that the amount of binding agents used is higher than in typical cement based stabilisations, which will move the overall grain size distribution a little bit more to the finer particle side. So the grain size curve of aggregate is not easily possible to determine after stabilisation mixing.

The aggregate layer was designed to be constructed so that the MWA was spread first onto the milled surface of the stabilised ash layer and MAC above the MWA layer. Layer thicknesses are specified in Table 3-.

Table 3-4. Aggregate layers for stabilisation

Section	MWA [mm]	MAC [mm]	GSD curve in Fig. 3-5
0+50–9+50 & 15+80–16+80	160	100	MWA+MAC combined 1
11+50–15+80	260	100	MWA+MAC combined 2

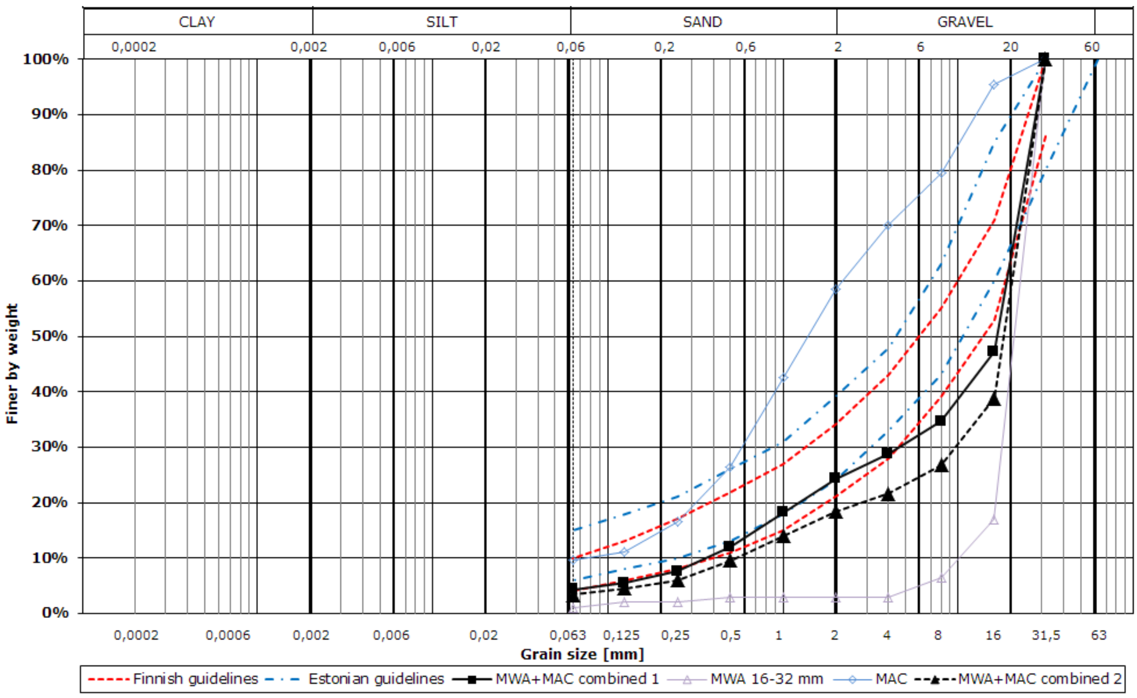

Figure 3-6. Grading curve (Est and Fin guidelines and estimates for MWA and MAC).

Figure 3-7. Aidu 16-32 mm aggregate

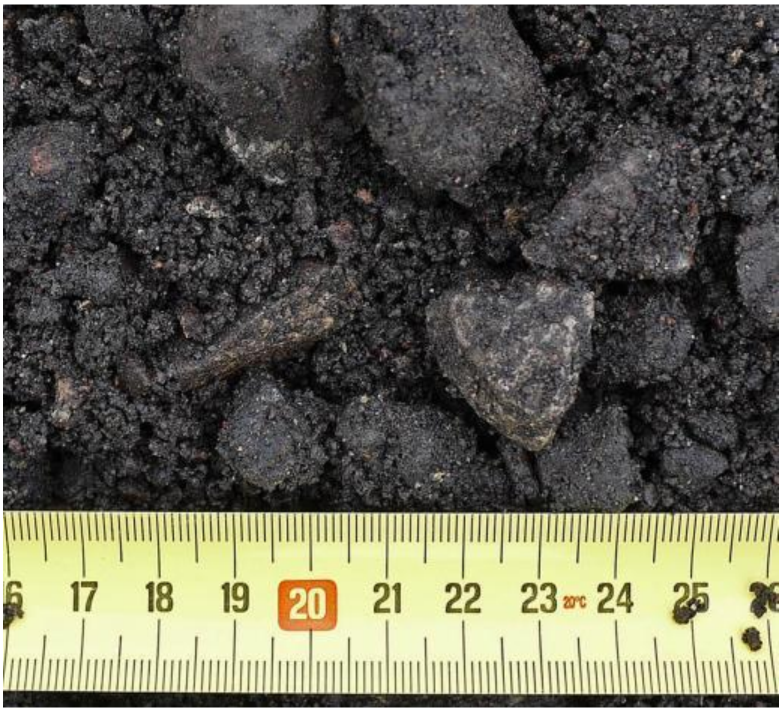

Figure 3-8. Milled and crushed asphalt concrete (MAC)

Cross section of the road structure

Cross section of the road structure was designed according to the Estonian guidelines[11]. The asphalt concrete surface was narrowed a bit compared to the old one. The old surface had been constructed past the stabilised ash layer, which had caused differing load bearing capacities to the centre and edge sections of the structure. Outcome of that difference can be seen in Figure 3-10 – the side section has settled in lower level than the other part of the surface. The new cross section (Figure 3-9) was designed so that the whole asphalt concrete surface will have consistent layer below it. The side slopes were also designed to be shaped more slanted to allow easier water movement out of the road structure.

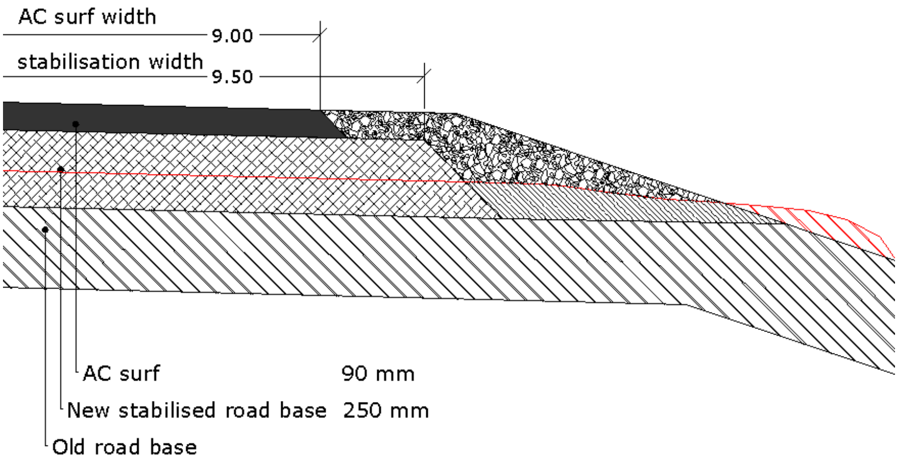

Figure 3-9. Extract of Cross section (STA 0+50-9+50). Red line - old road structure

[11] Elastsete teekatendite projekteerimise juhend. Maanteeamet 2001 (52): http://www.mnt.ee/failid/juhised/elastsete_teekatendite_projekteerimise_juhend.pdf

Figure 3-10. AC surface set lower on road side (Oct 2010)

4. CONSTRUCTION

4.1. Description of construction methods

The different steps of the whole construction work on the road section 0+50–STA 9+50 are shortly explained below (Table 4-1).

Table 4-1. Construction process timing

Work description	Date (year 2011)	Weather conditions
1. Milling of the old asphalt concrete surface	16th Aug	
2. Construction of the aggregate layer	17th-24th Aug	
3. Construction of stabilisation layer	14th-17th Sep	11-14°C, cloudy, showers
4. Construction of asphalt concrete layer AC 20 base 5 cm	21st-22nd Sep	15°, partly cloudy
5. Construction of asphalt concrete layer AC 12 surf 4 cm	22nd-23rd Sep	15°C, partly cloudy

Milling of the old asphalt concrete surface

The old asphalt concrete surface was milled off and stored for later usage.

Construction of the aggregate layer

The mining waste-milled asphalt concrete aggregate layer was constructed onto the stabilised ash layer or onto the sandy gravel layer. The mining waste layer was spread first and then the milled asphalt concrete was spread above the mining waste layer. The whole layer was compacted for traffic usage before the stabilisation works.

Spreading of the binding agents

The initial plan was to spread the binding agents with so-called wet method. In the wet method, the binding agents are mixed and wetted with off-site mixing facilities and transported to the construction site in ready-to-use form. The wet binding agent mixture is spread with an asphalt spreading machine.

Spreading method was however changed to dry method, which uses a tractor-pulled cement spreading car. Reason: factory-mix product was not transferable from silo to truck and from truck to spreader. The binding agents were spread separately – cement first and oil shale ash in second round. Cement spreading car worked quite well although especially the fly ash spreading was considerably slow with it and it clearly worked as a bottleneck in the whole construction method.

Stabilisation mixing

Stabilisation mixing was performed using a rotary mixer with water adding feature. The mixing was done in two phases – first phase after spreading of cement and second phase after spreading of oil shale ash. Water was added during the second phase to prevent the cement reacting too quickly.

Surface grading

The surface of the mixed layer was graded quickly after the second phase of mixing with a road grader.

Compaction

Immediately after grading, the layer was compacted with a vibrating roller with at least six compaction rounds. The first round of compaction was always done without vibration and the rounds after that with vibration. Compaction and load-bearing capacity was followed with Inspector during the construction works. The results of the measurements were recorded.

Asphalt concrete surface

Asphalt concrete layers, AC 20 base 5 cm and AC surf 4 cm, were constructed after three days the stabilisation works were finished. Before laying the asphalt concrete the surface of the stabilisation layer was cleaned and binder BE50 R was spread 0,3 kg/m2. The receipts of asphalt concrete mixes were verified before the asphalt works. Laid asphalt concrete mixes were tested in the laboratory. The compaction and thicknesses of the layers were followed during the works and the results were recorded.

4.2. Site journal – stabilisation works 2011

Site journal with measurements sheets was recorded by Ramboll Finland's and Ramboll Estonia's on-site quality controllers.

Tuesday 13th of September 2011

Figure 4-1. A view at STA 0+00

Stabilisation works were supposed commence on Tuesday 13th of September, but the concrete factory, which was in charge of the binding agent mixing had to change their mixing schedule and hence the start of the works was postponed to Wednesday morning.

The water content of the body material (mining waste aggregate and crushed asphalt concrete, Figure 4-2) was measured in a few different points and it was between 5.5 and 5.7 %. Compact of the body material was also analysed as the materials, which were tested in the laboratory varied compared to the body material on the road.

It was raining lightly the whole day, but the rain didn't seemingly wet the surface.

Figure 4-2. Dried body material weighed for water content analyses

Wednesday 14th of September 2011

Stabilisation works were supposed to commence at 7.00, but the concrete factory, which was in charge of the binding agent mixing had problems with the mixing process. At 10.00, one full 15 tonne lorry load of binding agent mixture (6% of OSA EF PF and 3% CC) was ready for stabilisation mixing. However, there were problems connecting the lorry platform to the asphalt paver (Figure 4-3 and Figure 4-4) and also with getting the binding agent mixture out of the platform. Eventually, the whole load of binding agent mixture was emptied onto the road and loaded to the asphalt paver with a Bobcat (Figure 4-5).

Figure 4-3. Asphalt paver used for wet binding agent mixture spreading

Figure 4-4. Trying to connect the truck and paver

Figure 4-5. Bobcat filling the asphalt paver punker

The first batch of stabilisation was tested with the first 15-tonne truckload of wet binding agent mixture. Width of the mixing drum of the rotary mixer used for stabilisation work was approximately half of the width of one lane, thus both of the lanes were divided into two sides.

Before mixing, approximately 2 l/m^2 of water was added onto the road surface. 5 l/m^2 of water was added through the mixer. This day the surface of the road was seemingly wetter than day before, because of the rain during the day and night before.

The first batch of stabilised material on the ditch side of right lane (STA 0+50–0+75) ended up being too wet and it was not possible to compact it well enough. In the centre side of the right lane within same section material was drier and it was possible to compact it well enough. Water content of the ditch side was measured at 10.1% and respectively 7.1% in the centre.

Assumed that the water content in the aggregate layer was around 6% (measured 5.5% and 5.7% a day before) 7 l/m^2 of added water should have risen the water content approximately to 7.5% mark (1 l/m^2 of added water roughly rises the water content 0.2 percentage unit), which should have been adequate amount. However, as the water content was as high as 10.1% at the ditch side, either the rain had over-wetted the aggregate layer or the rotary mixer had spread the water unevenly during the mixing.

The whole section was mixed and graded once more with road grader to equalise the water content, after which the ditch side ended up being dry enough for compaction while it was still a little bit elastic (bouncy). After the second round of mixing, water content of 8.4% was measured. 10% optimum water content determined in the laboratory was clearly too high for on-site conditions as even the 8.4% water content made the material too elastic for even light-weight traffic.

After the second round of mixing, grading and compaction, the test section was left untouched over the next night (Figure 4-8).

Figure 4-6. The Rotary mixer used in stabilisation work

Figure 4-7. Road grader shaping freshly mixed road base

Figure 4-8. The first batch of stabilisation mixing after grading and compaction

Because of the mixing problems at the concrete factory, it was decided to spread the binder in dry form with a cement spreading car (Figure 4-9). The dry binding agent mixing was used for the rest of the day. Dry spreading was started at STA 5+00 with the cyclone ash and Komposiittsement binding agent composition (5% OSA CYCL. and 5% of CC). In the end of the day, stabilisation on the right lane was finished until STA 7+25 without any bigger problems.

The dry binding agents were spread and mixed in two phases. Cement was spread first and mixed into the aggregate layer without any added water. The surface of the road was quickly graded after the first round of mixing. After that, fly ash was spread and mixed into the road with 5 l/m2 added water. The spreading car operator calibrated the spreading amount by weighing the binding agent spread on a 0.25 m^2 container (Figure 4-10).

Figure 4-9. Tractor-pulled cement spreading car

Figure 4-10. Binding agent spread measurement during spreading

Thursday 15th of September 2011

First thing in the morning, the wet binding agent mixed section (STA 0+50–0+75) was checked and it was seemingly hardened enough for traffic use. The surface was very smooth and had no detached material on it. Figure 4-11 shows comparison of the surfaces of "too wet section" and "drier section" one day after stabilisation.

Figure 4-11. Left: surface of the „too wet section", right: surface of the „drier section"

Work started at STA 7+23 with the dry mixing method using similar methods tried a day before. The first 75m (until STA 8+00) of the right side of the right lane was already mixed with cement a day before, but without added water. It was decided to mix it again with both of the binding agent components, thus the cement amount was doubled. 12 l/m^2 of water was used for that section. The water content at that section was measured to be 10.0%, but it didn't show any "elasticity" problems as the material with 8.4% water content showed yesterday (note: different binding agents).

After STA 8+00 7 l/m^2 of water was added at the beginning, but it was quickly raised to 9 l/m^2 as the material seemed to be too dry (w = 6.1%). After changing the added water amount to 9 l/m^2, the measured water content was between 6.7% and 8.2%. For some reason, the centre sector of the right lane was wetter (8.2%) than the side contrary to experiences acquired a day before even though the added water amount was the same for the both sides. The "too wet" material could be easily seen from the surface (Figure 4-12).

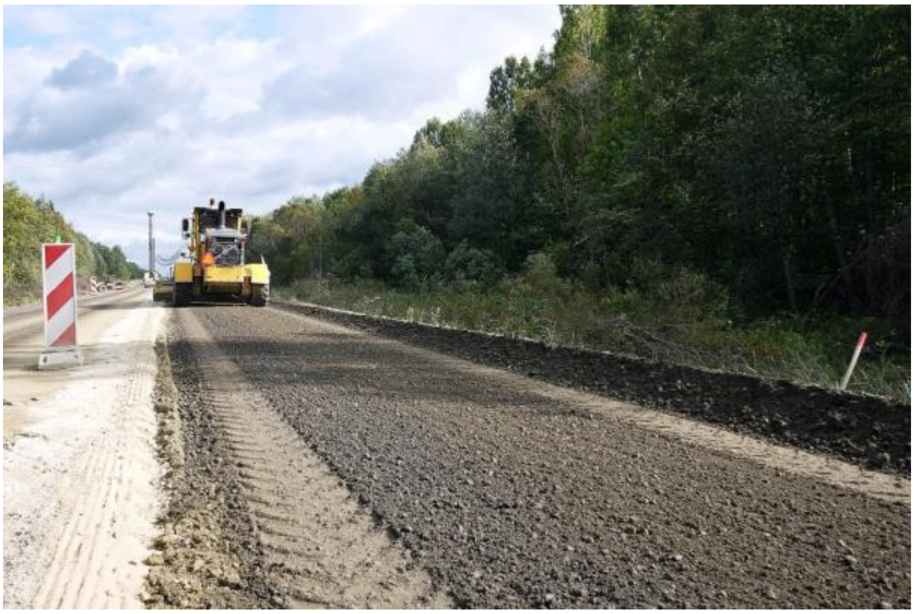

Figure 4-12. Too wet material in the centre sector of freshly stabilised section at STA 8+50

After reaching the end of right lane (STA 9+50), traffic was moved to the stabilised right lane and stabilisation work was started from the first part of the left lane (STA 50) around 14:00. According to the water content measurement, the material was similar to the material in the other end of the section. Water addition of 5 l/m^2 was instructed to prevent material getting too wet. Weather was dry and considerably windy for the whole day, which resulted in a lot of dust formation during the binding agent spreading.

Figure 4-13. Grain size sorting on the surface of freshly stabilised section. The lane was stabilised in two rotary mixer runs and the coarser looking tracks are located in the extreme left side of the Rotary mixer track

Friday 16th of September

Works continued under supervision of Ramboll Estonia. The contractor was instructed to add 5–7 l/m^2 of water through the rotary mixer and visually follow the state of the mixed material.

4.3. Stabilisation Works 2012

A part of stabilisation works, which couldn’t be constructed in year 2011. The needed binder was not available. At the beginning of October 2012 the works have been finalised using same construction methods as 1 year before. There are shown two machine pictures in Figure 4-14 and Figure 4-15.

Figure 4-14. Binder transporting to the construction site

Figure 4-15. Rotary mixer and water transporting

4.4. Quality control during construction

Geometry

- Stabilisation depth (Ramboll EE / Nordecon)
- Width (Ramboll EE / Nordecon)

The depth, width and cross fall of the stabilisation layer were measured and recorded.

Niton test results

Niton[12] – field test equipment (Portable XRF Analyzer)

Water content

Water content values were measured during the construction of the right lane. Measurements were done with (microwave) oven drying method, which, while simple and usually very accurate, wasn't completely adequate for pilot construction as each measurement took 10–20 minutes and the water addition through the rotary mixer required constant control. Now the body aggregate also contained considerable amount of bitumen since the milled asphalt concrete in it, which may have caused some of the inconsistencies in the results.

All measured water content values are listed in

Table 4-2. The results obtained after all phases of mixing were finished are marked with darker colour. The other results are from the measurements done on plain body aggregate or after adding only cement with no added water. As the measurement work had to be focused more on controlling the amount of added water through the rotary mixer than actually following the quality of the work, the combined results are very diverse.

[12] http://www.niton.com/en/

Table 4-2. Measured water content values during the construction (light: measured before completion of mixing**)**

	Chainage	Lane*	w [%]	Notes
6% OSA EF PF OBT+ 3% Komp. CEM	0+65	R d	8.4	After two round of mixing, seemingly too wet
	0+75	R d	10.4	1st batch of stabilisation mixing, + UCS sample
	0+75	R c	7.1	1st batch of stabilisation mixing
	1+50	R d	5.5	Only 3% PCC added
	1+65	R d	6.0	UCS sample
	1+65	R d	7.0	Seemingly too dry material
	2+00	R m	5.5	Body materials only
5% OSA CYCL + 5 % Komp. CEM.	5+10	R m	5.8	Only PCC added, sample 1
	5+10	R m	5.2	Only PCC added, sample 1
	5+20	R m	6.8	UCS sample
	7+00	R m	5.7	Body materials only
	7+35	R d	10.0	Double PCC amount, 12 l/m^2, still Works
	7+60	R m	5.0	Only 5% PCC added
	8+00	R d	6.1	UCS sample
	8+50	R c	8.2	9 l/m^2 water added, slightly too wet
	9+00	R c	7.2	UCS sample
	9+00	R d	6.7	

**R = right lane, s = ditch lane, c = centre side, m = mixed*

Compaction parameters

Maximum dry density and optimal water content was assessed in the field using a amended version of modified Proctor compaction test. Instead of 25 hammer blows per layer, only 8 blows were used. It has been noticed in the past occasions assessing the rate of compaction that lower blow count represents the actual compaction done by a roller compactor better than its standardised counterpart. This is a practical value what is usable for the purposes of this kind material. If aggregate material differs, it has to be estimated again. Obviously it is clear that the value is about 6 to 11 and it is far from standard 25 hammer blows.

Figure 4-16 shows the modified Proctor compaction test results at STA 5+20 on the right lane (Binding agent 5 % OSA CYCL. + 5 % CC). The water content - dry density combinations were not completely consistent, but according to the results the maximum dry density was around 2080 kg/m3 and the water content around 8 %. It has been experienced that typically, the actual roller compaction is easier in fly ash based stabilisations when the water content is at least or a little bit above the optimum content, which is also recommendable since the hardening reactions start rapidly consuming water. Now, it was experienced that the mixture of aggregate, binding agents and water transformed its state to more elastic when water content was just slightly above the optimum, and while it compacted seemingly well, it was not possible move on top of it with anything with smaller tire print than the roller without making any ruts.

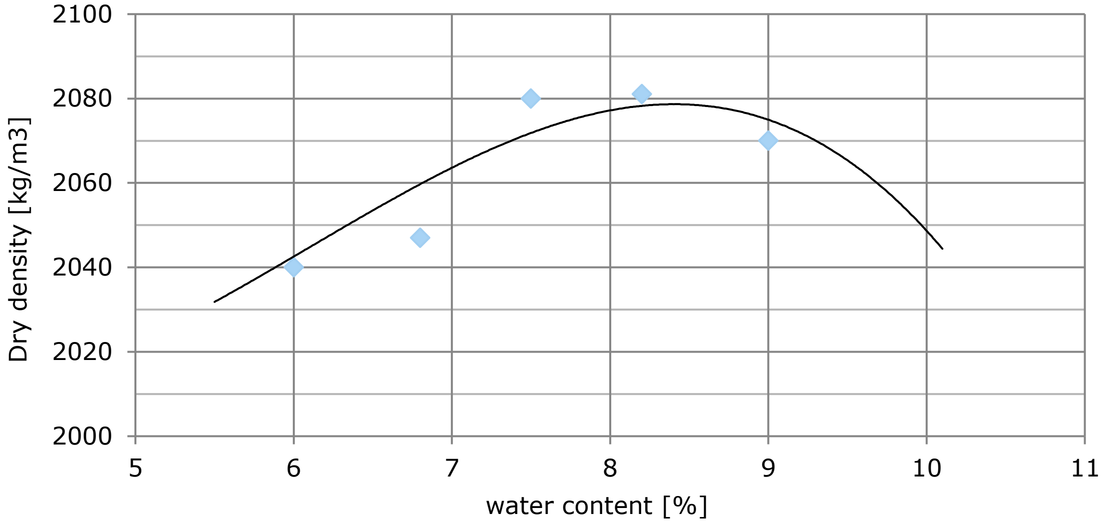

Figure 4-16. Mod. Proctor compaction results (STA 5+20). Binder: 5% OSA CYCL. + 5% KC

Compaction

Bearing capacity of the stabilisation layer was measured later. The "light weight Loadman" (Inspector-device) was tested during pilot construction by contractor. But it doesn't work well with coarse materials.

Troxler[13] has not been tested for measurements of compaction degree or moisture content because in Finland it has been tested many times for fly ash stabilisation. Troxler device doesn't work with fly ash and milled asphalt concrete. The heavy metals of ash or the hydrogen of asphalt may disturb the measurements.

Laboratory tests

The samples of stabilisation layer were tested in the laboratory. There were tests for unconfined compression, density and freeze-thaw durability (see Table 4-4 with explanations in Chapter 4.5). The *Measured water content values during the construction* test results are included in the

Table 4-2 (see above).

4.5. Construction facts

- Used material amounts
- Construction times

The quantities of the works and used materials on road section STA 0+50…STA 9+80:

Table 4-3. Use of materials by sections

[13] http://www.troxlerlabs.com/home/

	Location (LS/RS)	Area [m²]	Quantity [t]	Consumption [kg/m²]
Mining waste aggregate (MWA) 16-32 mm	0+50-9+50	8526.25	2319.14	272.0
Milled asphalt concrete (MAC)	0+50-9+50	8526.25	1534.74	180.0
Cement CEM 42,5R	RS 0+50-5+00	2113.75	30.86	14.6
	RS 5+00-9+50	2137.5	52.16	24.4
	LS 0+50-5+00	2137.5	31.21	14.6
	LS 5+00-9+50	2137.5	52.15	24.4
Total:	0+50-9+50	8526.25	166.38	19.5
Ash EF PF	RS 0+50-0+88	156.75	4.59	29.3
	RS 0+88-5+00	1957.0	51.19	26.16
	LS 0+50-5+00	2137.5	62.63	29.3
Total:	0+50-5+00	4251.25	118.41	27.85
Ash CYCL	RS 5+00-9+50	2137.5	52.16	24.4
	LS 5+00-9+50	2137.5	52.15	24.4
Total:	5+00-9+50	4275.0	104.31	24.4
Water	RS 0+50-0+88	156.75	0.94	6.0
	RS 0+88-5+00	1957.0	5.87	3.0
	RS 5+00-7+25	1068.75	5.34	5.0
	RS 7+25-9+50	1068.75	9.62	9.0
	LS 0+50-5+00	2137.5	6.41	3.0
	LS 5+00-7+46	1168.5	5.84	5.0
	LS 7+46-9+50	969.0	4.85	5.0
	0+50-9+50	8526.25	38.87	4.56
Asphalt concrete AC 20 base	LS 0+49-9+70	4184.34	462.3	111.0
Total:	RS 0+45-9+78	4235.82	484.62	115.0
	0+45-9+78	8417.16	946.92	112.5
Asphalt concrete AC 12 surf	RS 0+38-5+14	2142.0	238.38	111.0
	RS 5+14-9+80	2097.0	241.66	115.0
Total:	LS 0+38-9+80	4239.0	488.64	115.0
	0+38-9+80	8478.0	968.68	114.3

Compressive strength and freeze-thaw durability

In total, nine unconfined compressive test specimen were made; four in the section with EF PF ash (STA 0+50–5+00) and five at the section with CYCL ash (STA 5+00–9+50). The test results are presented in Table 4-4.

The samples from the EF PF ash section is compared to earlier laboratory results and new result values are expected. The comparable samples from STA 0+75 were tested for time-strengthening comparison between 7 and 28 days. According the results, the structure shows adequate strength after one month, and the strengthening hasn't been too fast. The comparable samples from STA 1+65 were tested for freeze-thaw weathering comparison. The strength loss between normal UCS result and FT weathered result was measured to be only around 15 %, which points to successful stabilisation. Both of the sample pairs had almost the same results with 28 d UCS.

Water content (w) is measured on site using microwave oven. It would be also possible use normal drying oven, but that microwave works faster. The measurement tolerance has to be tested before in laboratory. The differences in water contents and dry densities didn't seem to have any clear effect on the results in these site tests. Despite it is known from earlier experience that they will have an effect on the strength results.

The samples with CYCL-ash, however, yielded very mixed results. 28 d UCS values were on the low side for the STA 5+20 and 9+00 samples and very low for the STA 8+00 sample. The comparable STA 8+00 samples were used for assessing the strength difference between 7 d and 28 d UCS and according to the results the UCS had dropped to even lower level, which doesn't indicate too promising strength increase over time. The STA 8+00 28 d sample had also expanded its volume, which was probably the cause for low strength. Unconfined strength test and freeze-thaw results of the field specimens are presented in (Table 4-2). There is no official target value for UCS or UCS (freeze-thaw) tests for road construction. Based on previous experience r it can be said that UCS-values over 1 MPa are good and over 3 MPa very good. UCS values should not exceed 12 MPa, to reduce risk of transverse cracking from shrinking of layer.

Table 4-4. Compressive strengths of the samples taken during the construction

STA	Lane	w [%]	ρ_d [kg/m³]	7 d UCS [MPa]	28 d UCS [MPa]	28 d UCS FT [MPa]	90 d UCS [MPa]	Notes	Struc-ture type*
0+75	R d	7.1	2091	1.4	3.8				A
1+65	R d	6.0	2053		3.9	3.3			A
5+20	R d	6.8	2047		1.5			28 d ex-panded	B
8+00	R d	6.1	2048	0.5	0.3				B
9+00	R c	7.2	2113		1.3	1.2			B
9+75	L d	7.3	2041		5.6				C
10+75	R d	8.7	1969		3.0	2.1			D
11+00	R d	9.0	1900		3.4	2.4			D
11+00	R c	9.7	1938		3.5		4.2		D
11+75	L c	7.4	1978		2.1				E
13+00	R d	7.7	2058		4.0		5.2		F
14+50	R c	8.0	2024		4.6				F
15+80 - 16+80	No sampling because material in structure type D is same as G								G

UCS = unconfined compressive strength, FT = freeze-thaw durability test, R = right lane, d = ditch side, c = centre side

**A = layer stabilisation 25 cm, old structure, EF PF 6% + KS 3%*
B = layer stabilisation 25 cm, old structure, CYCL 5% + KS 5%
C = layer stabilisation 35 cm, old structure removed, EF PF 6% + KS 3%
D = layer stabilisation 35 cm, old structure removed, EF CFB 9%
E = layer stabilisation 35 cm, old structure removed, OSA CYCL 5% + KS 5%
F = layer stabilisation 25 cm, old structure, EF PF 6% + KS 3%
G = layer stabilisation 25 cm, old structure, EF CFB 9%

5. ENVIRONMENTAL STUDIES AND FOLLOW-UP PROGRAM

5.1. Properties of oil shale ashes and mining wastes

- Leaching properties
- Overall concentration of harmful substances

Detailed information is given in the Environmental on-going survey report annexes.

5.2. Summary of pre-construction environmental studies

Initial monitoring (environmental quality control) was carried out before construction in piloting sites began, to determine the regional background values. Follow-up Monitoring results are compared with these values.

The first pilot test section (No. 13109 Narva-Mustajõe road km 14.5 to 16) is located near the Estonian power plant and is heavily influenced by power plants' waste facilities (ash storage sites). Pilot test site is surrounded by woods and farmlands, the nearest residential building is farther than 1 km.

To identify the chemical composition of water, the water samples were taken by certified water sampler in 27.07.2011 as follows:

- Road no 13109 Narva-Mustajõe monitoring point was located in the ditch upstream of the OSAMAT's works near road km 15.7 (L-Est X 6583566.1 and Y 724515.9[14]).

Following tests were carried out in the water samples: pH, electrical conductivity, anions Cl^-, NO_3^-, SO_4^{2-}), cations (NH_4^+, K^+, Na^+, Ca_2^+, Mg_2^+) and metals that can get into the water from OSAMAT's works: As, Pb, V, Mo and Cr. Tests were carried out in accordance with the Water Act, laboratory test methods and testing requirements.

Hazardous substances in soil were analysed according to the monitoring program and data was obtained from the following substances: Sb, As, Ba, Hg, Cd, Cr, Cu, Pb, Mo, Ni, Se, Zn, V, Cl, F, SO_4 and pH. No contamination was found in both road sections' soil and ground; and the limit values set out by the Estonian Ministry of the Environment Regulation No. 38 'Limit values for hazardous substances in the soil' were not exceeded.

Narva-Mustajõe road section is surrounded by meadowsweet (*Filipendula ulmaria*) and common sedge, whose usual habitat type is swamped forest; and goutweed (*Aegopodium podagraria*) whose usual habitat type is coniferous forest. The forests are mostly middle-aged or young, dominant tree species are birch and white alder. There are no protected plant species and valuable habitats in the neighbourhood of the road section.

The solubility of the harmful substances was studied according to the survey programme with 1 step batch tests. Sb, As, Ba, Hg, Cd, Cr, Cu, Pb, Mo, Ni, Se, Zn, V, Chloride, Fluoride, Sulphate, pH-start and pH-final total content and solubility (L/S 10) were analysed in raw materials and in road mixtures.

More detail information can be found in Environmental on-going survey results report of OSAMAT project.

[14] Estonian Land Board: http://www.maaamet.ee

5.3. Environmental follow-up program

The environmental procedures are the same before and after the piloting: appropriate soil and groundwater samples are taken close to the piloting site, i.e. at spots and distances that could be affected by the OSA (the sampling spots are to be determined on the basis of soil and hydrological conditions of the site). Samples are also taken from the pilot construction area at a depth that will be directly below the structural course that will contain OSA material. The samples are analysed for those contaminants which OSA contains in noteworthy amounts and which might be a risk to the environment. Long-term follow up will repeat the environmental tests once a year, always at the same time period and at least for the whole project period.

Water samples near the road have been taken before the piloting works and it showed that the water content is not natural as some of the elements as As, Pb, V, Mo and Cr are not found in Estonian natural groundwater. So it can be concluded that the area has been affected by nearby ash fields. Still all investigated elements were below accepted contamination levels and so are not dangerous. Because this area is still influenced by nearby ash fields it is not possible to find out how much stabilisation will affect natures chemical composition. So it was decided, that water sample will be taken 12 and 24 months after background water tests (27.07.2011), as this will only show if the work area is still below accepted contamination levels, not how much stabilisation changes water content. Water sample tests are the same as before construction works.

Vegetation conditions were investigated on 20.06.2011 and the same investigation were done in June 2012 and June 2013. Investigation purpose is to find out if some vegetation changes can be found after stabilisation works.

Total content and leaching tests for the common detrimental/harmful substances:

- The leaching tests will be done with diffusion (NEN 7347) or two stage batch tests (DIN-EN-12457-3) and the total content tests will be done by leaching the substances from the material (DIN-EN 13656 or DIN-EN 13657 and by analyzing the contents of harmful substances by standard methods (ICP- MS, ICP- AES or AAS).
    - Tests for samples that represent the structure to be constructed
    - Tests to un-contaminated samples
- Analysing (at least) the same components as from the water samples. These are tested according to the Finnish legislation about the utilisation of recycled materials (VNa 403/2009), Sb, As, Ba, Cd, Cr, Cu, Hg, Pb, Mo, Ni, Se, Zn, V, SO_4^{2-}, F^-, Cl^-.

5.4. Environmental follow-up results summary

Soil

The soil samples were taken from Narva-Mustajõe section in 30-31.10.2012. The plots where more oil shale ash was used in a binder were selected as sampling places of the first and second construction phases.

The vast majority of sample results had all lower results than background data samples. However, one exception was with a sample where sulphate level was significantly higher than in all the rest of the sample results. As the rest of the samples, this rise of sulphate content was not observed this one sample increase is likely to be an anomaly. Such an anomaly can also be caused if naturally (in Estonian sediment) occurring pyrites were in the sample. Because of just such reason samples were taken from several locations, so that the results of just such sample anomaly could not affect the final conclusions.

The only clearly observed rise in all samples was with copper content. The copper content of all the samples was still 5-10 times lower than the target value.

Water

A control water sample was taken from the Narva-Mustajõe road section a year after the analysis of background data sample (26/07/2012), which should in time provide comparable data on the chemical composition of the water. The results showed a significant content reduction of hazardous substances (As, Pb, V, Mo and Cr); the only analysed substances that showed an increase in the content were Na, Cl and SO_4.

These results show that the work completed on the site does not substantially increase the content of hazardous substances in the water. However, the reason for the reduction of the substances is not due to implementation of testing method.

About a month after the end of the second construction phase of Narva-Mustajõe road new water samples were taken (17.10.2012). Again the results had not been changed significantly. Last control sample from Narva-Mustajõe road section, taken on 30.07.2013, can be compared with the baseline samples as well as the sample taken in 2012. Therefore, it can be said that no significant change in the sample levels was detected. In conclusion, the test section road works do not have an effect on the water quality. Additionally, the analysis shows rather a sign of improvement due to the ditches that has been reconstructed during the road building and the reparation of the water system.

Flora

During the visual survey conducted on site in June 2012 water in the ditch was clear and natural-looking, oil shale ash or alkaline pollution effects were not detected. If alkaline compounds or other hazardous substances leach out for a longer period of time, vegetation and other biota surrounding the ditches can be affected. However, side effects may occur to the aquatic biota that can be found in watercourses connected to the ditches (Kulgu stream). Given the relatively large dilution effect and leaching test results, significant impacts on the quality of water and aquatic biota are unlikely.

During the reconstruction of the road, embankment was renewed and cleaned, roadside ditches were deepened; also, the land around the ditches, around 5-7 meters wide, was cleared and levelled. Bushes were cut down from the cleared area. The topsoil layer of the soil was stripped and later used in replanting the roadsides. Beyond the roadside forest areas, road reconstruction and activities related to it manifested no immediate or significant indirect effects. The work did not harm any natural plant communities or valuable habitats; also there were no known protected species on the affected area or in the neighbourhood.

By the end of June 2012, the land was cleared and levelled; also the bottoms and sides of the ditches were spontaneously re-vegetated. Effects on roadside vegetation were similar to normal, standard technology work. Oil shale ash used in mass-stabilisation had no detectable impact on the vegetation.

Due to the fact that no active construction works were conducted in 2013, no supplementary disturbance in addition to traffic was associated. Compared to the results of 2012, it can be said that in 2013 there was remarkably more vegetation. The vegetation cover on road edges was rather common for this type of habitat and no clear signs of using oil shale ash as the alkaline substrate could be detected.

6. TECHNICAL FOLLOW-UP

The technical survey of the layer stabilisation and mining waste structures includes:

- Load bearing rate measurements before construction (also measurements of the base structure after grinding the surfacing of subgrade after excavating the old road) and 28 days, 90 days and one year after construction.

Technical testing of drilled samples:

- Comparison of 28 days and 90 days drilled samples to tests made in materials action

Making of follow up plans:

- Load bearing rate measurements three years after construction
- Paving quality measurements
- Defect counting and IRI measurements before construction and in 2014

Compression strength test results done for drilled samples on 28 days hardening time are presented in the below. In total 5 samples were chosen for testing from total of 37 drill samples. Results of unconfined compression strength are good and values are higher than unconfined compression strength of 28 days after construction (Table 4-4.). This is very positive and it tells that strengthening of the structure is continuing. Result of the sample taken from 9+70 in 2012 autumn is an extra measurement for age 1 month. Other results (in year 2012) the drill samples are one year age.

Strength of samples tells that EF-binder gives much more strength that cyclone–ash binder despite of lower cement content. EF-binder (structure type A) gives very high strength results and cyclone-ash binder gives good strength results.

Table 6-1. Follow-up testing of drill samples

Construction period	Age of structure	Location	Depth	Sample diameter ø	Height [mm]	28 d UCS [MPa]	Structure type*
Autumn-2011	1 year	0+72	0.09-0.19	93	104	9.4	A
Autumn-2011	1 year	3+83	0.13-0.23	93	102	8.5	A
Autumn-2011	1 year	5+50	0.08-0.18	93	103	2.1	B
Autumn-2011	1 year	8+23	0.1-0.2	93	105	2.0	B
Autumn-2012	1 month	9+70	0.08-0.18	93	98	4.5	C

**A = layer stabilisation 25 cm, new base aggregate: MWA + MAC, EF PF 6% + KS 3%*
B = layer stabilisation 25 cm, new base aggregate: MWA + MAC, CYCL 5% + KS 5%
C = layer stabilisation 25 cm, new base aggregate: *MWA + MAC, EF PF 6% + KS 3%*

In total of 37 drill samples were taken out of the constructed test road. The following list of all the drill samples includes their location, sample depth and description of drill sample quality is presented in the table below (Table 6-2). There are quite many loose samples. It is possible that drilling is so hard action that it breaks the samples during sampling. Some of samples were taken from locations of transverse cracks.

Table 6-2. Description of drill samples (autumn 2012)

	Location	Depth	Remarks/quality of sample	Structure type*
1	00+72	0.09-0.25	piece, ~140 mm OK	A
2	00+72	0.095-0.28	piece, side not intact	A
3	00+72	0.09-0.23	piece, side not intact	A
2	00+72	0.28-0.40	piece, side not completely intact	A
6	3+83	0.0-0.27	OK	A
4	3+85	0.09-0.20	piece, side not completely intact	A
7	5+50	0.10-0.30	piece, ~160 mm OK	B
8	5+50	0.085-0.315	piece, ~220 mm OK	B
9	5+50	0.0-0.275	piece, OK	B
11	8+23	0.085-0.31	piece, OK	B
12	8+23	0.09-0.23	piece, not intact	B
10	8+23	0.09-0.23	piece, ~120 mm OK	B
10	8+23	0.39-0.50	piece, ~110 mm OK	B
15	9+70	0.085-0.35	loose, max ~50 mm OK	C
13	9+70	0.075-0.27	piece cut	C
26	11+80	0.075-0.44	loose, max ~50 mm OK	E
25	11+80	0.09-0.45	loose, max ~50 mm OK	E
30	12+40	0.085-0.42	loose, max ~50 mm OK	E
28	12+40	0.07-0.43	loose, max ~90 mm OK	E
32	13+74	0.08-0.44	loose, max ~80 mm OK	F
33	13+74	0.08-0.40	loose, max ~70 mm OK	F
35	15+00	0.38-0.58	~180 mm OK	F
35	15+00	0.04-0.38	loose, max ~70 mm OK	F
34	15+00	0.09-0.42	loose, max ~50 mm OK	F
38	16+25	0.09-0.43	loose, max ~50 mm OK	G
39	16+25	0.12-0.42	loose, max ~50 mm OK	G
37	16+25	0.08-0.44	loose, max ~50 mm OK	G
41	16+70	0.08-0.39	loose, max ~50 mm OK	G
43	16+70	0.10-0.45	loose, max ~50 mm OK	G
42	16+70	0.12-0.42	loose, max ~50 mm OK	G

**A = layer stabilisation 25 cm, new base aggregate: MWA + MAC, EF PF 6% + KS 3%*
B = layer stabilisation 25 cm, new base aggregate: MWA + MAC, CYCL 5% + KS 5%
C = layer stabilisation 35 cm, new base aggregate: MWA + MAC, EF PF 6% + KS 3%
D = layer stabilisation 35 cm, old structure, EF CFB 9%
E = layer stabilisation 35 cm, old structure, OSA CYCL 5% + KS 5%
F = layer stabilisation 25 cm, old structure, EF PF 6% + KS 3%
G = layer stabilisation 25 cm, old structure, EF CFB 9%

The following pictures represent the remarks on the quality of the sample. The word 'piece' means that the sample is intact and the word 'loose' means that sample material is destroyed during the drilling. The loose material doesn´t hold its form for proper material testing as can be seen from the picture below (Figure 6-1).

Figure 6-1. Example of piece: sample 6, and loose: sample 26

Following load bearing measurements have been made in three consecutive autumns (2012-2014). Target value for the road according to the design is 260 MPa. Measured lowest values were about 260 MPa and highest values above 600 MPa. Bearing capacity of the road is high. It is typical for structures of stabilised layer and base course stabilisation. The structures in STA 0+00-9+50 were constructed in 2011 and the rest of the structures in 2012. From the Figure 6-3 it can be seen that the FWD measurements show high increase in bearing capacity when the age of the structure has reached 1 year. From the figure it can be seen that the structure before renovated section (STA -1+00-0+50) and section after (STA 16+80-18+00) have lower bearing capacity than the renovated sections.

The defect count analysis have also been made in three consecutive autumns (2012-2014), the results are given in Table 6-3. On the first 450 m there are more transverse cracks than on the following 450 m. It is typical for stabilized structures that there are narrow cracks. They are caused by shrinking of the stabilized layer, very typical for stabilisation of coarse materials with hydraulic binders. Typically they are transverse-type cracks (see Figure 6-2). Such cracks are often formed immediately within hardening process of material and generally not growing further. There are also some longitudinal cracks, usually caused by frost. The joint crack is in the asphalt joint. Places of fatigue cracking indicate problems in the pavement structure. In present case those are probably results of works quality.

Table 6-3. Defect counting 2012/2013/2014

STA	Length of the section [m]	Transverse crack [pcs]	Narrow longitudinal crack [m]	Narrow joint crack [m]	Fatigue cracking (m2)	Defects sum [%]
0+00-0+50	50	1/1/1	0/0/0	0/3/0		0.7/0,8/0,7
0+50-1+00	50	1/1/1	0/0/0	0/0/0		0.7/0,7/0,7
1+00-2+00	100	3/3/3	0/0/0	0/10/0		1.1/1,2/1,1
2+00-3+00	100	2/2/2	0/0/3	0/0/		0.7/0,7/0,9
3+00-4+00	100	2/2/2	0/11/9	0/2/0		0.7/1,5/1,4
4+00-5+00	100	2/2/2	2/3/3	0/3/0		0.9/1,0/0,9
5+00-6+00	100	1/4/4	0/0/0	0/5/0		0.4/1,5/1,4
6+00-7+00	100	1/2/2	0/2/3	10/11/0		0.5/1,0/0,9
7+00-8.00	100	0/1/1	0/0/0	0/0/0		0.0/0,4/0,4
8+00-9+00	100	1/2/2	1/0/0	0/0/0		0.4/0,7/0,7

STA	Length of the section [m]	Transverse crack [pcs]	Narrow longi-tudinal crack [m]	Narrow joint crack [m]	Fatigue cracking (m2)	Defects sum [%]
9+00-9+50	50	0/0/0	0/0/0	0/0/0	-/-/1,5	0.0/0,0/0,4
9+50-10+00	50	-/0/0	-/0/0	-/0/0		-/0,0/0,0
10+00-11+00	100	-/0/0	-/0/0	-/0/0		-/0,0/0,0
11+00-12+00	100	-/0/0	-/0/0	-/0/0		-/0,0/0,0
12+00-13+00	100	-/0/1	-/0/1	-/0/0	-/7/7	-/1,0/1,4
13+00-14+00	100	-/0/0	-/0/0	-/0/0	--	-/1,0/0,0
14+00-15+00	100	-/0/1	-/0/0	-/10/10	-/0/0	-/0,0/0,5
15+00-16+00	100	-/0/2	-/0/0	-/0/0	-/0/0	-/0,1/0,7
16+00-17+00	100	-/2/1	-/0/0	-/3/3	-/0/0	-/0,4/0,4
17+00-17+50	50	-/0/-	-/0/0	-/0/0	-/0/0	-/0,1/0,0

Figure 6-2. Transverse crack in 2013

FWD measurement show that the target bearing capacity as average for all sections has been reached. However there is one short section where the bearing capacity is somewhat lower than was designed to be. This weak spot is located at STA 9+00 and at this location also fatigue cracking was recorded during defects counting.

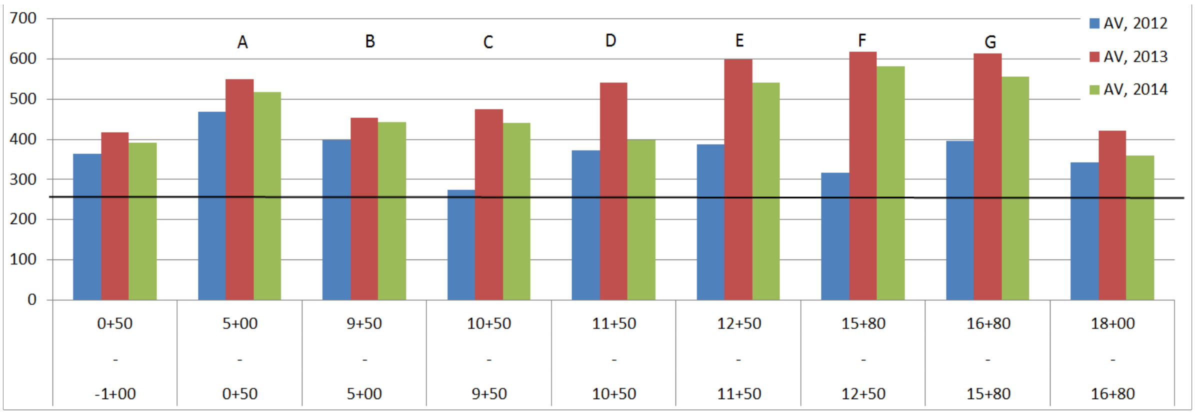

Figure 6-3. Progress of bearing capacity (comparison of FWD results 2012-2014)

Comparing the results of measurements, bearing capacity has raised ca 40% (2012-2013) as well as in average, as in the lowest values achieved. Lower strengthening was identified in sections without renovation. This indicates the hardening of stabilized layer within first year of use. Follow-up figures on bearing capacity recorded in 2014 indicate everywhere lower levels than 2013 (but considerably higher than 2012), probably reflecting that bearing capacity depends on humidity levels in the construction layers. Humidity conditions during the measurements can have considerable effect.

During the follow-up investigations also the rut depth has been recorded. The average rut depth in different test sections is between 2.3-3.6 mm. The highest rut depth values are in the locations of fatigue cracking e.g. STA 9+00 and STA 12+75, the highest rut depth is 14-17 mm. However no general conclusions can be made based on the differences in rut depth as the differences are very small. Rutting, in general, develops fast during first year (after-densification of bitumen-bound layers), continuing with much lower speed during pavement maturing and increasing in last phase of pavement lifecycle together with other defects.

Swelling and shrinking effect

During laboratory tests swelling effect of OSA was experienced. Pure OSA material broke the test equipment (glass) in laboratory and it was caused by OSA expansion. On the road, mixture of OSA and cement was used as binder material. After construction cracks occurred on the road pavement as described above. Estonian and Finnish expert's position is that the main and most important reason of cracks is shrinking effect of the monolith body, which formed after stabilisation. Big amount of binder material was used in stabilized layer and this created very hard monolith. Reasons for shrinking were impact of cold weather and water/moisture leaving the material. Our understanding is that expanding material does not induce reflecting cracks but only uneven surface (higher IRI/unevenness rate). Decades ago in Estonia were used cement-stabilization in road construction and mostly these sites had similar transverse cracking.

7. CONCLUSION

Laboratory testing and feasibility of the materials

Laboratory tests proved that it is possible to utilise oil shale ashes and mining waste in creative way to construct road base courses.

The EF PF ash seems to be a very suitable material for stabilisation purposes. It might be even possible to use it without any cement addition when mixed with high quality aggregates, or at least combined with lower cement amount than 3 % of aggregates dry weight used in the construction. According to EE, they already have adequate facilities to separate and store that fraction.

Used aggregate has also a strong effect for constructed layer. In this case there was old structure under asphalt layer and mining waste aggregate in the mixture. Multi-component mixture complicates quality control and pre-testing in laboratory. The variation of aggregate mixture cannot be measured on site exactly. So it is important to test optimal water content on site too, as it is done in this pilot case (optimal in laboratory conditions was not suitable for processing on site).

In the future the strength and freeze-thaw durability testing could be provided with longer curing time, i.e. 90 d, because similarly working fly ash based constructions typically continue hardening even for years to come. In theory, if the strength development is not disturbed by external factors, the freeze-thaw durability should be higher after a longer curing time. If the stabilisation work is performed before August, the stabilised structure should have at least 90 days ahead to develop its strength before the first freeze-thaw cycles start to weather the structure.

Construction and quality control

Combining the milled asphalt concrete (MAC) and mining waste aggregate (MWA) to create an aggregate with adequate grain size distribution is a clever way to utilise both of the materials as they are. No external mixing is needed as both materials are mixed during the stabilisation mixing. The mixing result should however be assessed with grain size distribution analysis from different horizontal points of the rotary mixer track as it seemed, at least on the surface, that the mixture of milled asphalt concrete and mining waste aggregate was slightly sorted in places. If the body aggregate for stabilisation is created by combining two (or more) different materials with unsuited grain size distributions, the combined grain size distribution for different compositions should be assessed to ensure that the end product has an adequate grain size distribution.

In this project, the dry binding agent mixing method worked a lot better than its wet counterpart. Preparing the wet binding agent mixture has usually been straightforward with coal or biomass based fly ash and cement combination, but this does not work with OSA because of high content of CaO. Dry method always induces some dust formations, which should be taken into account if the work is performed close to housing or settlements that might be affected by the high pH dust. One solution is that mixing equipment feeds binder during mixing the layers.

In the quality control process, however, is important that the ashes tested in laboratory and ashes, used in stabilisation process on site, should have same age and storage conditions, because the CaO content is changing in time through contact with water vapour (reaction, $CaO+H_2=Ca(OH)_2$).

Controlling the water content and interrelated compactness is very important. In this process, aggregate material mixing and measuring water content is very demanding. For the technology development, one possibility is that mixer feeds also water, it will improve the control process. Compaction density or degree is also very demanding to measure as it is noticed in this case. Troxler or "light weight loadman" (Inspector) doesn't work. It needs more field research.

Estimated effects on environment

In conclusion, it can be said that the use of oil shale ash does not involve any additional immediate negative impacts on the environment. The impacts that came with the construction of the test section were according to the forecast and are temporary. There were no unplanned or significant negative effects on the natural environment, associated with the work.

Technical follow-up

There are very high strength values from using EF-ash in the binder mixtures. Also strength values from cyclone ash binder mixture are high. Bearing capacity is a little higher in EF-mixtures binder section (type A) than cyclone ash binder mixtures (type B). In the same structure (type A) which has better bearing capacity and higher strength values were more shrinking cracks. The cracks are narrow and it is known that they are mainly caused by shrinking effect of very hard structure. Shrinking effect is caused because of impact of cold weather and water/moisture leaving the material.

There are no bearing capacity measurements during stabilisation works layer by layer because strengthening of layer takes time and structures can't be open so long.

It is important to continue monitoring (measuring the bearing capacity, IRI, rutting and defect counting) in the future. Initially, narrow cracks were identified (also seen in the pictures). Next follow-up years will show impact for the structure. Drilling of the samples from stabilised layer is not very easy because drilling breaks hard and brittle materials easily (to get better cores, large diameter drill is recommended). It could be reasonable to do drilling not too often. All follow-up studies have to be done for a long time after construction.

Impact of water content deviation in laboratory conditions has been identified, it should be verified by drill samples as optimum levels appeared different (laboratory optimum is not the same as on-site optimum). Based on these findings the recommendations on water content can be improved.

8. RECOMMENDATIONS

Binder selection

It seems that higher strength and bearing capacity of construction will cause more narrow shrinking cracks. In the future it is important to test binder mixtures of OSA and cement and probably gypsum too. There have been good experiences with binder mixtures of ash, cement with gypsum and using them in a harbour field stabilisation – case in Finland some years ago. Experiences in Finland show that gypsum decreases shrinking effect.

Reducing binder content

It seems among the laboratory tests that pure OSA has also even better swelling effect as gypsum has. The specimens of pure OSA expanded visibly in laboratory. In actual road environment occurred shrinking effect. Probably the effect of OSA was insufficient. Another possibility is to optimise the binder amount of the stabilisation (reducing binder). Cement and OSA content must be lower, because measured strength and bearing capacity are very high in this case.

Aggregate quality

The quality of aggregates is also very important. As the laboratory tests show, there are big differences between different tested aggregates. Therefore aggregates should be tested beforehand in all cases. Bearing capacities in all the test constructions were clearly higher than target value 260 MPa. In the section A they were mainly between 400-500 MPa and in the section B mainly between 300-400 MPa. However, within the material testing process, only grain distribution of aggregate was studied, not the strength parameters (LA) – it should be added in future.

Water content and bearing capacity

Different equipment to test bearing capacity should be tested to find best suitable technology. Water content measurement is important so portable equipment should be found to measure humidity of material even without sampling.

Mixing

OSA binder mixtures should be mixed beforehand and not separately on the road. This means that investments should be made on development of mobile mixer equipment. Layer stabilisation equipment should be improved so that it will be possible to feed dry binders and water. All these test results show that OSA binders are working very well in these types of road constructions. Therefore OSA binders may have a good market situation for such applications in many European countries, when the quality controls on the binder treatment site and on the constructing site are successfully solved.

Cutting of joints

Recommendation for future is to consider cutting of joints to the road to reduce the risk of cracks However, current Estonian regulations do not foresee such necessity. Russian regulation on low-strength hydraulic-bonded mixes specifies need for use of joints at 12…15 meters (Spanish sources declare optimal ca 4 m, Germany even 3 m for narrow induced cracks), cutting the stabilized base (at ¼ thickness) and asphalt layers (joints in asphalt to be filled with polymer mastics). EUPAVE recommends cutting transverse joints in stabilized base or sub-base at 2-3 meters in the fresh material (pre-cracking).

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