

Fly Ash in Forest Road Rehabilitation

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Abstract

Finnish forestry and bioenergy production is seeking novel uses for the fly ash deriving from biomass conversion. There are various possibilities for using fly ash in civil engineering including road construction. The increase in bioenergy production has created more interest for using ash in forest roads. However, no established methods for the rehabilitation of forest roads exist yet. Hence, this research aims to find a suitable construction method that involves using ash that provides adequate bearing capacity. It involved building ten test road sections: two of them were reference sections without fly ash. The study examined the effect of four different rehabilitation methods on the bearing capacity of roads. Measurements were made once before and four times after the rehabilitation. The measuring devices included a light falling weight deflectometer (LWD), a dynamic cone penetrometer (DCP) and a conventional falling weight deflectometer (FWD). Two of the rehabilitation structures were 50 and 25 cm thick fly ash layers. The other two were 15 and 20 cm thick layers made of fly ash and aggregate in different mixing ratios. In all cases, the constructed layers were paved with aggregate. Statistical comparison showed that the bearing capacity of the rehabilitated road sections had improved compared to the reference sections. The recorded bearing capacities after rehabilitation (during spring thaw in 2012, 2013 and 2014) were about the same as before rehabilitation in summer 2011. Based on this study, fly ash can be recommended as an option for forest road rehabilitation.

Keywords: forest road, rehabilitation, fly ash, bearing capacity

1. Introduction

Increasing utilisation of bioenergy is increasing the quantity of produced ash. The wood and peat burning processes of the forest industry are producing a significant amount of ash. About 600,000 tonnes of the ash produced in Finland annually is wood- and peat-based fly ash (Emilsson 2006). The total annual production of ash in the country is about 1.5 million tonnes. There are 52 power plants around Finland, which produce at least 1000 tonnes of fly and bed ash per year (Tuhkarakentamisen käsikirja 2012).

Ash utilisation is divided roughly evenly between earthworks, landfills and fertilizers or other use (Ojala 2010). The growing ash production necessitates finding sensible ways of utilising ash. Increased utilisation can bring several benefits and there are many potential ways of using fly ash. As a fertilizer, it increases timber growth in forests (Moilanen et al. 2005) and agricultural field crops (Patterson et al. 2004), it can be used as a construction material in road building reducing

the need of natural stone resources (Edil and Benson 2007) or as a filler in concrete (Wang et al. 2008). Ash is no longer considered waste from the energy and forest industries point of view. On the contrary, ash is nowadays a by-product that can be used in various ways. Yet, there are obstacles to expanding ash utilisation. In spite of the above mentioned possibilities of use, a remarkable amount of ash ends up in landfills. From the economic point of view, it is important to minimise the amount of ash going to landfills, because the dumping charge has lately been raised to €55 per tonne.

Ash is not as stable a material as mineral soil. Ash can be separated into fly ash and bottom ash based on particle size. The properties of fly ash from wood and peat vary widely depending on the fuel used, the burning process and the combustion gas filtering technique (Korpijärvi et al. 2009). Two features need to be taken into consideration when planning the use of ash: its content of environmentally harmful heavy metals and its technical properties. Ash has been used in the

structural layers of public roads because of the flexible licensing policy (Tuhkarakentamisen käsikirja 2012). However, forest roads do not belong to the same class as public roads. Instead, the use of ash in forest roads requires a more specific and precise license granting procedure. Proper use of ash can improve the bearing capacity of forest roads. The forest industry supply chain needs more constant year-round flow of raw materials, which requires adequate trafficability of forest roads. The bearing capacity of forest roads is at its lowest during the spring thaw when their moisture content is high (Salour and Erlingsson 2013). The thawing period either restricts timber haulage or breaks the road structure if haulage continues during the low bearing capacity period. On the other hand, high bearing capacity promotes wood supply management and reduces the need to repair road damage due to rut formation. There are about 135,000 km of forest roads in Finland, of which about 3000–4000 km are rehabilitated annually (Metsätalastollinen vuosikirja 2013). Forestry would benefit greatly if it could combine ash utilisation and forest road rehabilitation.

The use of ash in road structures is receiving increasing attention. Related studies have focused mainly on coal ash due to the common use of coal in energy production. There have also been studies on low-volume roads where both coal ash (Edil and Benson 2007) and bio ash (Lahtinen 2001) have been used. The studies indicated that ash is well-suited for road construction. Bio ash has become the focus of studies in recent years. The utilisation of wood-based fluidised and bottom ash in forest roads, where the ash was mixed into the existing road structure (Bohrn and Stampfer 2014), has also been studied. The measurements carried out by a light falling weight deflectometer (*LFWD*) showed improvement in bearing capacity during the half-year observation period. In another study, wood fly ash and bottom ash were found to have a positive impact on bearing capacity in both a laboratory test and a field survey using the *LFWD* over a four-week monitoring period (Supancic and Obernberger 2012). In a third study, wood fly ash was mixed into the gravel of an existing road. There, the falling weight deflectometer (*FWD*) results indicated improvement in bearing capacity during the monitoring period of a year and a half (Vestin et al. 2012). In the three above mentioned studies, ash was mixed into an existing road structure by a road grader. The grader breaks up the old road structure and the ash functions as a binding component. These three positive results were produced by short-term bearing capacity surveys.

The aim of this research was to study various rehabilitation techniques and the development of bearing

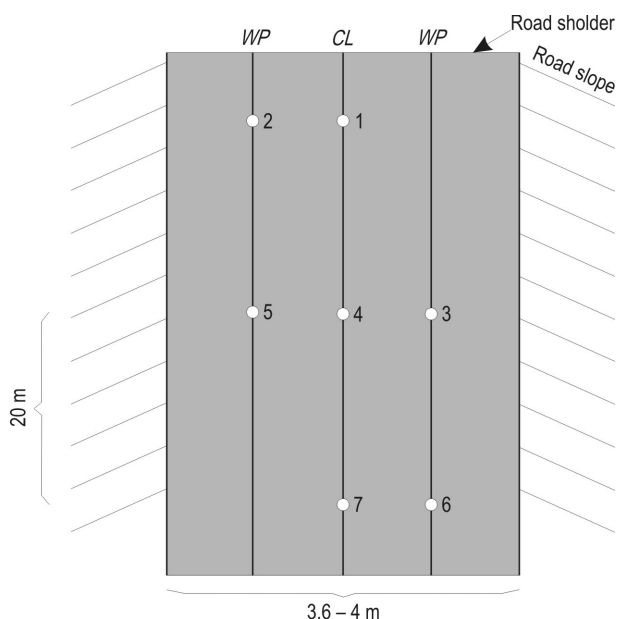


Fig. 1 Locations of measuring points along the test section. Numbers 1–7 denote measurement points, *WP* stands for wheel path and *CL* for centre line

capacity when forest roads are renovated with bio fly ash. Four different test structures were made of fly ash. Two of them included only fly ash, while other two contained fly ash mixed with an aggregate. The development of the bearing capacity of the test structures and a reference structure was compared. The goal was to achieve higher bearing capacity especially during spring thaw, when insufficient bearing capacity restricts haulage on forest roads.

2. Material and methods

2.1 Test sections

Ten test road sections were established to monitor the effect of ash on bearing capacity. The test sections were located along two forest roads where through-traffic was possible in Central Finland near Jämsä Municipality (62°1'31" N, 24°54'5" E). The test sections were prepared before the actual rehabilitations were done. The test sections had seven measuring points (Fig. 1). Four of them were on the wheel path (*WP*) and three on the centre line (*CL*). The test sections and measuring points were marked by poles at the road side in summer 2011. That allowed finding the test sections in the following years. A total of ten test sections were established. There were four different test structures and a reference structure on both forest roads.



Fig. 2 Measuring devices: dynamic cone penetrometer, light falling weight deflectometer and falling weight deflectometer

2.2 Bearing capacity measurements

Instead of a single measuring device, this study used three different bearing capacity measuring devices: the conventional falling weight deflectometer (*FWD*), the dynamic cone penetrometer (*DCP*) and the light falling weight deflectometer (*LFWD*). The bearing capacity results can be considered a crucial indicator of successful rehabilitation. The most suitable measure of bearing capacity is elastic modulus (later *E*-modulus) measured in MPa. More information about bearing capacity and road structure can be acquired by using several measuring devices at the same time. The initial bearing capacity measurements were carried out with *LFWD* and *DCP* devices. Post-rehabilitation measurements were carried out with *LFWD*, *DCP* and *FWD* devices. Initial measurements took place in August 2011 and the rehabilitations were done in September 2011. Bearing capacities of the constructed roads were measured four times in 2012–2014. The monitoring period was three years, longer than with Bohrn and Stampfer (2014), Supancic and Obernberg-

er (2012) and Vestin (et al. 2012). A specific timetable of data collection is presented in Table 1.

DCP measurement is based on making a cone tip penetrate into the ground by the impact force of a falling weight. The mass of the falling weight was 8 kg and dropping height about 575 mm. The diameter of the cone was 20 mm and the angle of the tip 60 degrees. The equipment also included a measuring rod, which allowed reading the vertical penetration (mm) after each drop or a certain number of drops. The *DCP* Penetration Index (*DPI*; mm per blow) was calculated on that basis. The California Bearing Ratio (*CBR*) can be estimated from the *DPI* value with empirical Eq. 1 (Webster et al. 1992). The *CBR* value can then be used for estimation of elastic modulus (*E*-modulus) with another empirical Eq. 2 (Powell et al. 1984) that is one of the most well established alternatives among the available equations for making this conversation. More detailed justification and reason for using these equations was presented in a study by Kaakkurivaara et al. (2015). Measurement was continued until the depth of 400 mm was reached. The *E*-modulus values, derived based on *DCP* measurements, are hereafter referred to as E_{DCP} .

Table 1 Times of measuring rounds and rehabilitation

	2011	2011	2012	2012	2013	2014
	Summer	Autumn	Spring	Summer	Spring	Spring
<i>LFWD</i>	x	Rehabilitation	x	x	x	x
<i>DCP</i>	x		x	x	x	x
<i>FWD</i>			x	x	x	x

$$\log CBR = 2.46 - 1.12 \times \log DPI \quad (1)$$

$$E = 17.6 \times CBR^{0.64} \quad (2)$$

Where:

- CBR* California Bearing Ratio, %;
- DPI* *DCP* Penetration Index, mm/blow;
- E* soil's elastic modulus, MPa.

The *LFWD* device used in the study was a Loadman, manufactured by AL-Engineering Ltd of Finland. The operating principle of the *LFWD* is based on dropping a weight that causes a momentary deflection of ground surface. The mass of the falling weight was 10 kg and dropping height 800 mm. The diameter of the loading plate was 132 mm. The deflection was captured by integrating the acceleration signal twice. The internal electronics of the device calculated and showed maximum deflection (mm) and *E*-modulus (MPa) immediately after measurement. The Loadman uses Eq. 3 (Pidwerbesky 1997) to convert deflection measurements into *E*-moduli. The measurement results can later be transferred to computer via a USB connection.

$$E = 1.5 \left(p \times \frac{a}{\Delta} \right) \quad (3)$$

Where:

- Δ deflection under the Loadman loading plate;
- p vertical pressure on the base plate;
- a radius of the base plate.

The results of the third drop were used to determine the *E*-modulus in this study. It was done to reduce the influence of the loose surface layer. More detailed information about the use of the *LWFD* is presented in a study by Kaakkurivaara et al. (2015). The *E*-modulus yielded by the *LFWD* was denoted by E_{LFWD} in this study.

A conventional falling weight deflectometer was made using the Kuab *FWD* device. The operating principle of the *FWD* is also based on a falling weight, which causes a deflection impulse on the road surface. The impulse corresponded to the load of a 50 kN tyre. The *E*-modulus for the *FWD* was calculated with the same Eq. 3 as for the *LFWD*. Later in the article, the *E*-modulus of the *FWD* has been denoted as E_{FWD} . At this point it is important to notice that the efficient depth of influence in *FWD* measurements is far greater than that of *LFWD* measurements. Thus, the values of *E*-modulus derived based on *LFWD* represent basically only the top 200 mm of the road structure, while those based on *FWD* also include the influence of underlying subgrade up to the depth of 500 mm. More specific information about *FWD* measurements is available in an article by Kaakkurivaara et al. (2015).

2.3 Rehabilitation operations

In this study, the existing road structure was not altered, but a new structural layer was built on top of the old one. Five different test structure types were selected. Four of them contained different amounts of fly ash. Aggregate was also added to the reference



Fig. 3 A road grader was used to shape the road surface and mix the ash and aggregate

structure in deviation from the others which are normal in rehabilitation and common with forest roads. Two test sections of each type were built. The aggregate used in all test sections was similar. The particle size of the aggregate, crushed from oversize rock material, was between 0–32 mm. The total length of the rehabilitated forest road was 2400 m and a total of 1936 tonnes of fly ash was used. The fly ash was less than one year old. It had been stored outdoors exposed to the elements.

Two test sections were chosen as references. They were surfaced with an about 100 mm thick layer of aggregate. Technical implementation of the fly ash test structures involved the following. Firstly, side barriers were formed on both sides of the road to prevent fly ash from escaping into the ditches. The first test structure received a 50 mm layer of fly ash and a 100 mm layer of aggregate. The layers were mixed by a road grader (Fig. 3). The road grader was also used in all other test sections. Hereafter, the first test structure is referred to as #1. The content of fly ash was increased in the second test structure: fly ash and aggregate layers 100 mm thick were mixed. The second test structure is referred to as #2 later. Thus, fly ash was mixed with aggregate in both of the above mentioned test structures on top of the road. Another technique would have been to mix the materials at the storage site before transportation to the construction site. The plan was to have a uniform layer of fly ash on the third and fourth test structures. The thickness of the fly ash layer was 250 mm on the third test structure (#3) and 500 mm on the fourth one (#4). All test road structures were compacted immediately after construction by driving over them with an excavator several times. An environmental permit was required for the operation,

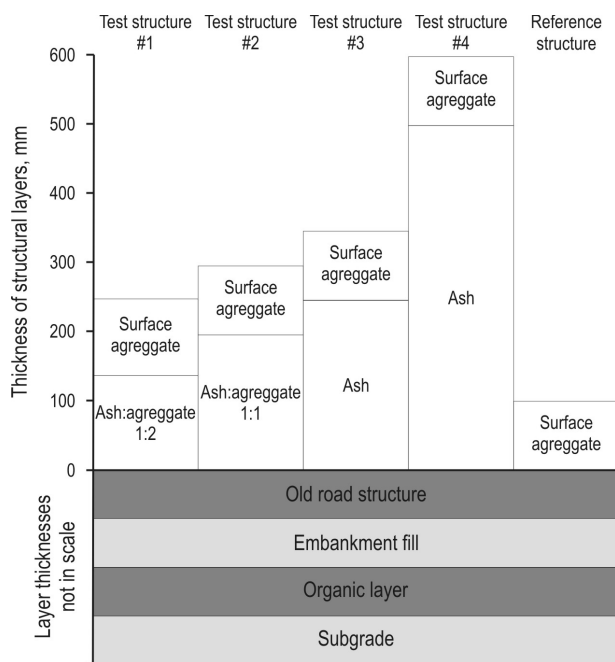


Fig. 4 Test structure profiles. Four different test structures included an ash layer

and its provisions required ensuring that the fly ash cannot escape from the road surface after construction. Due to the provisions, all test sections were surfaced with a 100 mm thick layer of aggregate. The thickness of the surface aggregate was hence the same as that of the reference sections. The excavator shaped all road slopes of the test sections. It was especially important in the case of the high profile test structures #3 and #4 to give them horizontal support. Extra soil materials from ditches were added to the road slopes of #4 test structure types. All test structure profiles are presented in Fig. 4.

2.4 Properties of ash

The fly ash was obtained from a forest industry power plant fuelled by peat and wood. When the ash used in this study was produced, the burned material was mainly wood-based. The share of peat was higher in winter and lower in summer. The fly ash was collected over one year. The exact mix ratio is unknown, because it varied continuously during the year. Variation in the mix ratio had an impact on fly ash properties. The fluidised bed combustion technique was used where the temperature was about 850°C. The contents of various elements were determined three times from fresh fly ash during the collection period. The content of calcium oxide was most important for the self-hardening process. Three analyses showed that the content of calcium varied between 15 and 19%. Higher calcium

oxide content also improves the bearing capacity of the structure due to the chemical bond between calcium oxide and water (Pecqueur et al. 2001). A modified Proctor test (standard SFS-EN 13286-2) was done on the fly ash, which showed that the optimum content of water is between 35 and 40% for compaction. The weather was rainy during the rehabilitation operation, which helped the compaction process.

2.5 Soil sample analyses of existing road

Soil samples were collected from the test sections before the rehabilitations to determine particle size distribution and organic content. Soil samples were taken from the existing road surface structure and subgrade at each test section. The samples describe the properties of the aggregate transported to site from other locations. The subgrade samples describe the properties of the subgrade and the embankment fill layer. Soil material from ditches had been used in the embankment fill layer. The grain size distribution curve was plotted using the wet sieving and pipette methods. Used sieve sizes were 0.63, 2, 6.3 and 20 mm. The organic content of the samples was determined by the burning method for grain sizes below 2 mm. Each plotted distribution curve was compared to the design grading curves of the Finnish Transport Agency (Finra 2005) to estimate the E -modulus according to the Odemark bearing capacity design method (Odemark 1949). Frost susceptibility of the samples was assessed according to the design grading curves of Finnish guidelines (Finra 1993). Specific details of these assessments are presented in a study by Kaakkurivaara et al. (2015). Hereafter, aggregate E_{GSD} refers to the surface layer of the existing road structure and subgrade E_{GSD} to the subgrade and embankment fill layers for E -modulus estimation based on grain size distributions.

3. Results

3.1 Structure of the road before rehabilitation

The test road sections were established on an existing forest road. The E -modulus values estimated based on soil samples taken from the existing road structure before rehabilitation are presented in Table 2. The E -modulus for the surface aggregate material of the old road was generally about 150 MPa. The E -modulus for the subgrade was poor or at best moderate, which is typical for fine sediment soils. The surface aggregate layer was equally very thin and the subgrade material poor at every test section. Thus, the bearing capacity of the whole test section area was poor.

Table 2 Mean elastic modulus (MPa) based on grain size distributions (E_{GSD}) for existing road structure materials before rehabilitation. N stands for numbers of soil samples

	#1	#2	#3	#4	Reference
Aggregate E_{GSD}	150	150	200	150	125
Subgrade E_{GSD}	23	35	20	18	38
Samples, n	2	2	2	2	2

3.2 Bearing capacities of the rehabilitated road sections

Fig. 5 shows the bearing capacities of the test structures in terms of mean MPa. Each value is based on measurements from two test sections. The mean values of E_{DCP} and $E_{\text{LFW D}}$ are based on eight measuring points on the wheel path and six on the centre line. The mean values of E_{FWD} are based on four measuring points on the wheel path and two on the centre line. Each measuring round included these measurements on every test structure. The initial situation is represented by the values of summer '11, which were measured in summer before rehabilitation in autumn. It is important to keep in mind that after the rehabilitation, one measuring round was undertaken in summer and three rounds in spring.

It can be said that, in general, the bearing capacity started to decline after initial positive development. The highest E -modulus values were measured with the DCP for every test structure type. The highest values were measured for test structure #2 (222 MPa) and the second highest for test structure #1 (198 MPa). In the case of test structure #4, the bearing capacity (154 MPa) increased slightly higher than with #3 (135 MPa), but it remained clearly lower than for test structure #1. It is noteworthy that the bearing capacity of #4 test structures was 10–20 MPa lower than for other test sections initially. The positive development of the bearing capacity ended two years after rehabilitation and the results of the fourth measurement round were generally weaker than those of earlier measurement rounds. The bearing capacity of test structure #2 was about 160 MPa and those of test structures #1, #3 and #4 about 120 MPa. The adding of surface aggregate did not increase the bearing capacity of the reference structure. It even decreased from 120 MPa to 100 MPa, as shown by the last measurement round. This indicates that in spring '14 measurement conditions may have been different from the other measurement rounds. Nevertheless, the purpose was to do the measurements at the same phase of the frost thaw period.

The E_{DCP} results for the centre line were in line with the E_{DCP} results for the wheel path, but the E -modulus values were lower across the board.

The $E_{\text{LFW D}}$ measurements do not suggest similar clear improvement as the E_{DCP} measurements. The $E_{\text{LFW D}}$ results improved only by a few MPa, while the E_{DCP} values increased almost double compared to the initial situation for the #1 and #2 test structures. Initially, the $E_{\text{LFW D}}$ measurements showed clear improvement for #1 and #2, when bearing capacity values were over 70 MPa at their highest. In the last measurement round, the bearing capacities were about the same (53–56 MPa) for the #1, #2 and #3 test structures. The most positive observation was made concerning the #4 test structure, whose initial bearing capacity was the lowest (44 MPa), but the highest (59 MPa) in the last measurement round. The lowest bearing capacity was measured for the reference structure, whose bearing capacity values varied widely around 50 MPa during the entire survey. The above mentioned changes in the bearing capacity were measured on the wheel path. The $E_{\text{LFW D}}$ results for the centre line were similar to the $E_{\text{LFW D}}$ results on the wheel path, but the E -modulus values were lower across the board.

FWD measurements were not carried out before rehabilitation. After rehabilitation, the first bearing capacity values (E_{FWD}) were lowest for the reference structures and the #1 test structures (30 MPa). The highest bearing capacity was measured for the #4 test structure and the next highest for the #2 and #3 test structures (47 MPa). Bearing capacity values rose naturally in the summer '12 measurement round for all test structure types. The next year (spring '13), only #2 and #3 had maintained the achieved bearing capacity (54–55 MPa), while the bearing capacities of the other test structure types had declined. The capacities of all test structure types had deteriorated by the last measurement round, where test structure #3 (41 MPa) fared the best and #4 (38 MPa) next best. On the whole, bearing capacity values were slightly lower in the last measurement round than in the first. On the centre line, E_{FWD} results fell under the above described values varying between 40 MPa (#4, summer '12) and 16 MPa (reference, spring '13).

3.3 Statistical results of paired sample t -test

Table 3 presents a paired sample t -test, where the initial bearing capacity result and the result of each measurement round after rehabilitation for every test structure type were compared separately. The initial values were measured in summer before rehabilitation which took place in autumn. Mean values were

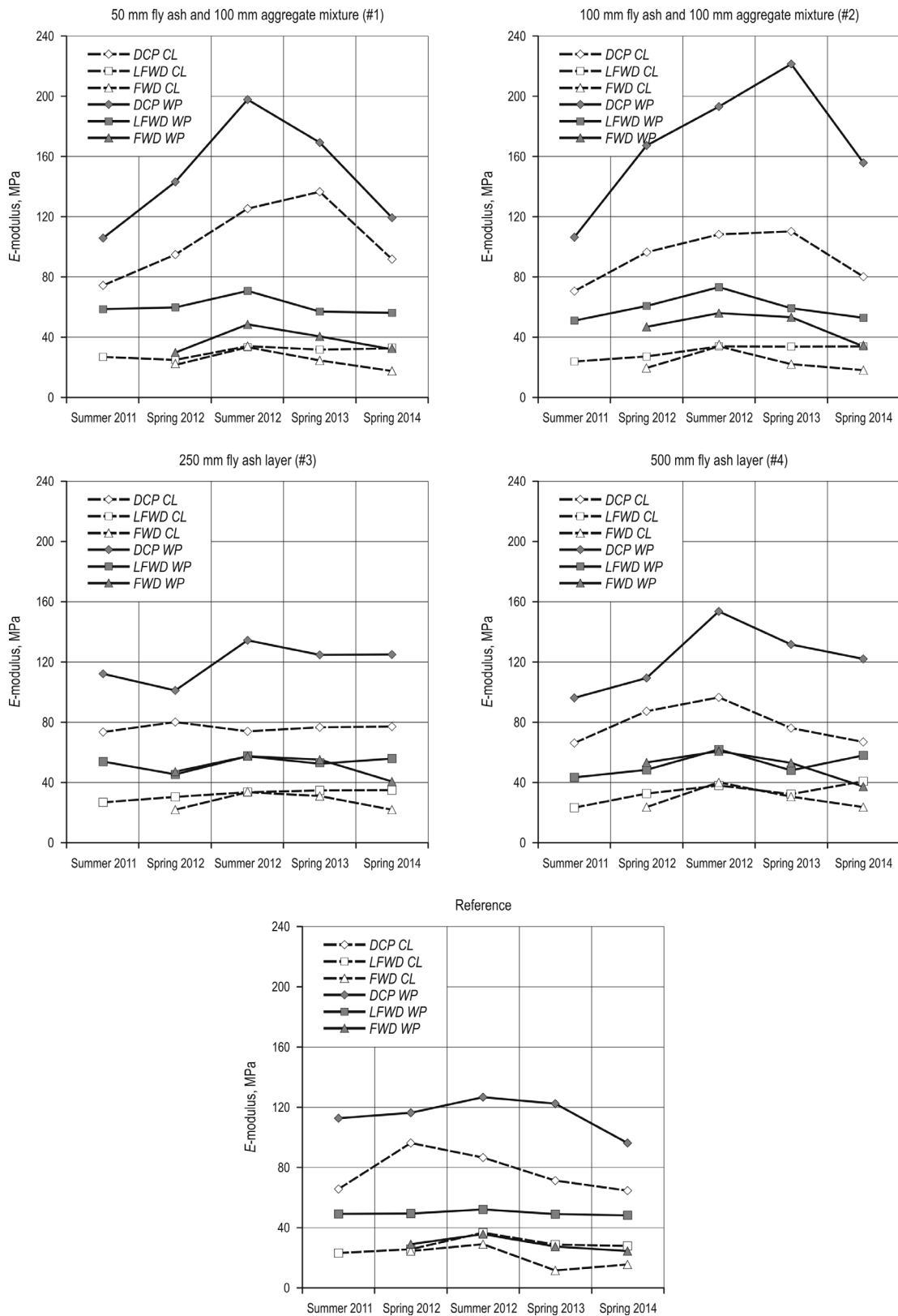


Fig. 5 Results of bearing capacity measurements for sections are presented in the following order: #1, #2, #3, #4 and reference test structures. The measurements were done with a dynamic cone penetrometer (DCP), a light falling weight deflectometer (LFWD) and a falling weight deflectometer (FWD). CL stands for centre line and WP for wheel path

Table 3 Statistical significance of paired sample *t*-test results between measurement rounds, when $p < 0.05$. Measurement unit: MPa. Minus indicates declined bearing capacity and plus improved bearing capacity. A blank box indicates insignificant change between values

	Device	Class	Test structure	Summer'11 MPa	Spring'12 MPa	Summer'12 MPa	Spring'13 MPa	Spring'14 MPa
Mean	DCP	WP	#1	106	+37	+92	+64	–
St. dev.					27	78	41	–
Mean	DCP	WP	#2	107	+61	+87	+115	+50
St. dev.					37	17	56	30
Mean	DCP	WP	#3	112	–11	–	+13	–
St. dev.					12	–	15	–
Mean	DCP	WP	#4	97	–	+58	+36	–
St. dev.					–	64	28	–
Mean	Loadman	WP	#1	59	–	+12	–	–
St. dev.					–	7	–	–
Mean	Loadman	WP	#2	52	–	+22	–	–
St. dev.					–	15	–	–
Mean	Loadman	WP	#3	54	–9	–	–	–
St. dev.					8	–	–	–
Mean	Loadman	WP	#4	44	–	+19	–	+15
St. dev.					–	14	–	9
Mean	DCP	CL	#1	74	+21	–	+62	–
St. dev.					18	–	42	–
Mean	DCP	CL	#2	71	–	+38	+40	–
St. dev.					–	28	12	–
Mean	DCP	CL	Ref.	66	+31	–	–	–
St. dev.					24	–	–	–
Mean	Loadman	CL	#1	27	–	+7	–	+6
St. dev.					–	5	–	4
Mean	Loadman	CL	#2	24	–	+10	+10	+10
St. dev.					–	6	3	5
Mean	Loadman	CL	Ref.	23	+9	+15	+9	+18
St. dev.					6	7	6	8

calculated and compared to initial values between measurement rounds. A negative value indicated a decline in bearing capacity and a positive value strengthening of it. Negative values were measured only twice – in the first comparison. Here, the calcula-

tion of standard deviation was based on the groups of observations behind both mean values. The number of observations was highest in the column where comparisons were made between measurement rounds of the same season (summer '11 and summer

'12). Statistically significant observations were almost double in the case of #1 and #2 test structures compared to uniform fly ash structures. The number of statistically significant observations was the same for both measurement classes – *WP* and *CL*. If a statistically significant observation was made in the same measurement round concerning a test structure, indicated by both E_{LFWD} and E_{DCP} , the improvement in bearing capacity was higher on the wheel path than on the centre line, as expected. This observation describes well the functioning of the structure since the bearing capacity improved more specifically on the wheel path where compaction of the structure was responsible for the improvement. About the same numbers of statistically significant observations were made with *DCP* and *LFWD* devices. Improvements of bearing capacities were generally higher when comparison was based on E_{DCP} values. The E_{LFWD} values of the bearing capacity did not improve as much as E_{DCP} values. Comparison of E_{FWD} results could not be done because initial values were not measured. Development on the centre line measured by E_{LFWD} was positive compared to the reference structure, which showed no improvement on the wheel path measured either by E_{LFWD} or E_{DCP} . In this statistical study, the number and location of the blank boxes of Table 3 must be taken into consideration. It means that statistical improvement was not detected. Some rows, where the combination of the measuring device, measurement class and test structure were not statistically significant, are also missing. They indicate that rehabilitation did not improve the bearing capacity.

3.4 Statistical results of independent sample *t*-test

Table 4 presents a *t*-test comparison of independent samples, where the results of rehabilitated test structures are compared to results for reference structures of the same measurement round. In Table 4, E_{LFWD} and E_{DCP} values have been divided between wheel path and centre line measurement classes. The table presents the bearing capacity and its standard deviation for each measurement round, if the statistical difference between the reference structure and test structures was significant. E_{FWD} results were also analysed, but statistically significant results were not found. The differences in bearing capacities were quite often statistically significant between the #1 test structure and the reference structure, and between the #2 test structure and the reference structure. The difference between test structure #2 and the reference structure was significant in every measurement round. On the other hand, no statistically significant difference was ob-

Table 4 Statistically significant results of independent sample *t*-test based on mean bearing capacity values (MPa) between test structures and reference structure, when $p < 0.05$. The number of observations consists of eight measurements on wheel path (*WP*) and six measurements on centre line (*CL*). A blank box indicates an insignificant change between values

	Device	Class	Round	Reference MPa	#1 MPa	#2 MPa	#3 MPa	#4 MPa
Mean	<i>DCP</i>	<i>WP</i>	Spring	116	–	+52	–	–
St. dev.			2012	36	–	24	–	–
Mean	<i>DCP</i>	<i>WP</i>	Summer	127	–	+67	–	–
St. dev.			2012	32	–	24	–	–
Mean	<i>DCP</i>	<i>WP</i>	Spring	122	+47	+100	–	–
St. dev.			2013	26	40	42	–	–
Mean	<i>DCP</i>	<i>WP</i>	Spring	96	–	+60	–	–
St. dev.			2014	25	–	16	–	–
Mean	Loadman	<i>WP</i>	Summer	52	+18	+21	–	–
St.Dev.			2012	7	12	12	–	–
Mean	<i>DCP</i>	<i>CL</i>	Spring	71	+65	–	–	–
St. dev.			2013	30	55	–	–	–
Mean	Loadman	<i>CL</i>	Spring	28	–	–	–	+13
St. dev.			2014	8	–	–	–	5

served in the case of #3 and #4 on the wheel path. The number of *DCP* observations was more than double that with the *LFWD*. Only two statistically significant differences were observed on the centre line. It should be noted that no observations were made during the first measurement round before rehabilitation (summer '11).

4. Discussion and conclusions

The study presents changes in bearing capacity during a survey covering a period of a little less than three years. In that period, a marked improvement occurred as the initial summer bearing capacity was reached already the next spring after rehabilitation. It meant considerably better trafficability during the most crucial time for haulage. The means of bearing capacity were clearly higher on the wheel path than on the centre line. That indicated that compaction of test structures had taken place as a consequence of light traffic loading after rehabilitation. The weather

conditions of spring 2014 can be the reason for the lower bearing capacity since conditions vary from year to year. Weakening of bearing capacity was also observed in the reference structure, whose bearing capacity had not changed before, which supports the above conclusion. A clear difference between the test structure types was indicated by all measuring devices. Similar development of bearing capacity in each measurement round was shown by all measuring devices on all test structures as shown in Fig. 5. In other words, development of bearing capacity was parallel in every measurement round independent of the measuring device. Improvement in bearing capacity can be mainly detected with *DCP*, to some extent with *LFWD*, but not at all with *FWD*. The weather conditions before measurement may be more relevant than the rehabilitation itself in explaining the similar values of E_{FWD} between the test structure types and the reference type. This argument is based on the measuring principle of *FWD*, where measurement depth is substantially deeper than with the other devices. Therefore, poor existing road E_{GSD} values under the test structure have a negative impact on all measurement results with *FWD*. It seems that empirical equals of *DCP* overestimated *E*-modulus values for these construction materials when compared to measurements of other devices. Time of the year also needs to be taken into consideration; there was high moisture content on road structure during spring thawing season. It may have affected the measurement results.

The paired sample *t*-test concentrated on following the development of the bearing capacity of each test structure type over time. Statistical correlation was observed between the E_{DCP} results when comparisons were made between the initial measurement and three following measurement rounds. Similar correlation did not occur in the case of the fourth measurement round with *DCP*. The fact that the *E*-modulus values were calculated by two equations does not matter in the paired sample *t*-test, because the target of comparison was another *DCP* measurement, not a measurement made with another device. The aim of this study was not to measure exact *E*-modulus values, but to observe the changes in bearing capacity by the same measurement device over the time and between different test structures. The empirical equations for *DCP* are usually working well; if the structures are homogeneous and particle size is not too large. Disadvantage of the *DCP* is that the measurement operation can be very time consuming on well compacted good quality aggregate. Improvement of the bearing capacity occurred when

initial E_{LFWD} results were compared to summer measurement round results, whereas comparison to spring measurement rounds did not indicate a clear improvement. There are two reasons for that: the measurement rounds were conducted in different seasons, and spring conditions also varied between years. The ranking of the test structures was not very clear. Statistical significances were observed more often in the case of the #1 and #2 test structures than the #3 and #4 test structures. Moreover, improvement of the bearing capacity seems to be bigger with the #1 and #2 test structures. The lack of observation in the case of the reference structure was a very good finding, because addition of surface aggregate did not improve the bearing capacity in this study. The paired sample *t*-test confirmed the impact of traffic on improving the bearing capacity along the wheel path, because related observations were made more often on the wheel path than on the centre line. However, the numerous blank boxes (Table 3), i.e. missing observations, challenge the functionality of the test structures.

The independent sample *t*-test excluded seasonal variations and focused on comparing test structure types at a particular moment in time. The independent sample *t*-test proved that the #1 and #2 test structures had clearly better bearing capacity than the reference structure in each measurement round. Two thirds of the observations were made on the wheel path and one third on the centre line. The tendency of the observations was the same concerning both E_{DCP} and E_{LFWD} . The *DCP* measurements revealed variation of the bearing capacity on the wheel path between the structures over time. It can be deduced that the used mixture of aggregate and fly ash works better than other structures in this study. The missing statistical observations from the measurement round before actual rehabilitation testify that the test sections had been initially in similar condition. The independent sample *t*-test does not confirm successful improvement of uniform fly ash structures since only one statistically significant observation was made.

The bearing capacity measurements were carried out by three different devices, which meant differences in the results. The bearing capacity results were device-specific and no contradictions were observed. Similar development trends of bearing capacities between the test structures or between measurement rounds guarantee the technical feasibility and reliability of the measurement methods. Bearing capacities showed clear improvement immediately after addition of fly ash. Statistical analyses revealed that

test structures #1 and #2 had improved. The improvement of test structures #3 and #4 was less pronounced. It should also be noted that it is not possible to exclude external factors in field circumstances, as it is in laboratory circumstances. Discussion of results rests on field measurement and full confidence of functionality requires longer survey time.

A conclusion can be drawn, although clear improvements in bearing capacities could not be established. The bearing capacity did not improve by increasing fly ash. It seemed to be more a consequence of adding aggregate. The conclusion is not in line with the expectations about the functionality of using fly ash on forest roads. The reason could be inadequate compaction during construction work. Unequal storage times of fly ash and the lack of a better mixing technique may also have affected the results. The results showed, nevertheless, improvement in all test structure types in spring compared to initial measurements since initial summer bearing capacity was reached in spring after rehabilitation. None of the test structures rose above the others based on the received results, but the fly ash and aggregate mix seemed to outperform uniform fly ash layers.

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