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Monitoring and Evaluation of Fly Ash Stabilization Stabilized Subgrade Constructed by the Wisconsin Department of Transportation

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MONITORING AND EVALUATION OF FLY ASH STABILIZATION STABILIZED SUBGRADE CONSTRUCTED BY THE WISDOT

by

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FINAL REPORT

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16. Abstract

This report describes the monitoring and evaluation of a field site where Class C fly ash was used to stabilize the subgrade during construction of a rigid pavement in a portion of USH 12 near Fort Atkinson, Wisconsin. Additionally, information from a second similar project (STH 32 near Port Washington, WI) that was monitored only during construction and a third project (STH 60 near Lodi, WI) that was monitored for 8 years is reported. The following observations are made based on this investigation:

1. All of the tests consistently indicated that the stiffness and strength of the subgrade were improved significantly by fly ash stabilization at all sites. Observation during construction, however, clearly demonstrated the benefit of fly ash stabilized subgrade (FASS) because once the fly ash is mixed and compacted in a window of dry weather the FASS remained stiff in subsequent rain events.
2. It is noted several significantly different soil types were encountered in all sites and the fly ash contents and moisture contents were variable (10% and 12% and 7-14%, respectively). Resulting California bearing ratio and moduli also varied. The gain in stiffness and strength are typically 2-3 times due to fly ash stabilization. *In situ* stiffness measured with the soil stiffness gage and dynamic cone penetration index also illustrated that the addition of the fly ash and compaction increased the strength and stiffness appreciably. These findings suggest that fly ash stabilization of subgrade should be beneficial in terms of increasing pavement capacity and service life.
3. The data also indicates a complex relationship between base soil type, amount of fly ash, and water content. For instance, in some soils (e.g., clay) the effectiveness of stabilization decreases when the water content of the soil increases whereas in some other soils (e.g., sandy) it increases. Therefore, a careful mix design is needed for fly ash stabilization involving all potential subgrade soils. The results from all sites also confirm that fly ash stabilization results in a relatively stiffer layer irrespective of the type of soil.
4. Resilient Modulus of the field-mixed FASS is close to that of undisturbed samples of FASS obtained by thin-wall tube sampler. Thus, the field-mixed FASS can be considered to be an effective method of assessing the *in situ* soil stiffness.
5. Moduli back-calculated from the falling weight deflectometer (FWD) test data indicated that, modulus does not display significant reduction over the years.
6. Pavement distress surveys indicate that fly ash stabilized sections perform comparable to control sections stabilized with breaker run.
7. Percolation from the pavement varies seasonally. Concentration of some elements from the leachate exceeded the Wisconsin preventive action and enforcement limits. However, these concentrations are expected to fall below the limits during transport to the groundwater table.

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EXECUTIVE SUMMARY

This report describes a field site where Class C fly ash was used to stabilize soft subgrade soils during reconstruction of a section of US Highway 12 near Fort Atkinson, WI. Additionally, data from two other fly ash stabilization projects, i.e., construction data from STH 32 and performance monitoring data from STH 60 over 8 years are incorporated. These projects consisted of mixing existing subgrade soils with fly ash (10 - 12% by dry weight), compaction, and placement of a new base course and HMA surface. California bearing ratio (CBR), resilient modulus (M_r), and unconfined compression (q_u) tests were conducted on the soils alone and the fly-ash stabilized soils (FASS) prepared in the field and laboratory to evaluate how addition of fly ash improved the strength and stiffness. *In situ* testing was also conducted on the stabilized and unstabilized soils with a soil stiffness gauge (SSG), dynamic cone penetrometer (DCP), and falling weight deflectometer (FWD). Pan lysimeters were installed beneath the roadway, two beneath fly ash stabilized soils and one control beneath unstabilized subgrade soils in US 12 and two in the fly ash section and 2 in the control section in STH 60, to monitor the quantity of water percolating from the overlying layers and the concentration of trace elements in the leachate. A column leaching test was conducted in the laboratory for comparison.

All of the tests indicated that the stiffness and strength of the subgrade were improved significantly by fly ash stabilization at all sites. However, this was not readily apparent at US 12 as the subgrade was substantially drier than the FASS because of several rain storms that took place during construction. Observations during construction, however, clearly demonstrated the benefit of fly ash stabilization because, once the fly ash is mixed and compacted in a window of dry weather, the FASS remained stiff in subsequent rain events. The gain in stiffness and strength were typically 2-3 times due to fly ash stabilization. *In situ* stiffness measured with the

SSG and dynamic penetration index (DPI) measured with the DCP also illustrated that the addition of the fly ash and compaction increased the strength and stiffness appreciably. These findings suggest that fly ash stabilization of subgrade should be beneficial during construction and, if durable, in terms of increasing pavement capacity and service life in the long-term.

The data also indicates a complex relationship between base soil type, amount of fly ash, and water content. It is noted that several significantly different soil types were encountered at these sites and the fly ash contents (10% - 12%) and the water contents (7-14%) varied. For instance, in some soils (e.g., clay) stabilized with fly ash M_r decreases when the water content of the soil increases whereas in some other soils (e.g., sandy) it increases. Therefore, a careful mix design is needed for fly ash stabilization involving all potential subgrade soils.

Analysis of the FWD data collected after several years of freeze-thaw cycles showed some initial degradation in the modulus but the modulus otherwise remained essentially unchanged or slightly increased at the US 12 site. However, long-term FWD data collected at STH 60 indicated that the fly ash section continued to gain stiffness and had a higher stiffness than the control section. There was no long-term monitoring at STH 32.

A review of the pavement distress surveys indicate that fly ash stabilized sections perform comparable to control sections stabilized with breaker run in USH 12 4 years after construction and also over 8 years of observation in STH 60.

Percolation from the pavement varies seasonally. The concentration of 5 elements at US 12 exceeded the Wisconsin enforcement limits and 4 elements exceeded the preventive action limit. All other elements were below these limits. A similar observation is also observed at STH 60. There was also exceedence of the limits at the control section lysimeters. These concentrations are the effluent concentrations, studies indicate attenuation occurs during transport through subsurface to the groundwater.

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1. INTRODUCTION

The Wisconsin Department of Transportation (WisDOT) has recognized the need and value to improve the quality of subgrades constructed on its improvement projects. One of the initiatives developed to achieve this improvement is the inclusion of select materials in the upper portions of subgrades constructed from silty or clayey soils. Eight alternate subgrade improvement methods have been approved for this application. Seven of the approved methods use conventional materials such as sand, gravel, and crushed stone to achieve the improvement. The eighth method is chemical stabilization of the subgrade, which includes using fly ash.

Fly ash stabilization of subgrades constructed from fine-grained soils is a developing and promising technology (Edil et al., 2002; Bin-Shafique et al., 2004; Trzebiatowski et al., 2004). For example, Edil et al. (2002) use fly ash to stabilized fine-grained subgrades in two short experimental sections in STH 60 at Wisconsin. Minnesota, Kansas and several other states have reported considerable success using fly ash for this purpose. Fly ash producers have also been active in the demonstration and promotion of this application. Many states have active programs promoting the beneficial reuse of fly ash and other high volume industrial by products. However, the effectiveness of stabilizing subgrades with coal fly ash in full-scale applications on an improvement project in Wisconsin is unavailable.

The study conducted both short-term and long-term monitoring and evaluation of a fly ash stabilized subgrade (FASS) constructed on the US12 improvement project. The purpose of this effort is to provide a document for the performance of a full-scale fly-ash stabilized subgrade located on US12 between Cambridge and Fort Atkinson, Wisconsin.

This report describes a project where self-cementing Class C fly ash from a coal-fired electric power plant was used to stabilize subgrade during reconstruction of a 1.6-km section of

US12 highway between Cambridge and Fort Atkinson in Wisconsin (≈ 40 km east of Madison). The subgrade soil was prepared by grading and compaction using motor grader and tamping foot compactor. Class C fly ash (12% by dry weight) was spread uniformly on the surface using truck-mounted lay-down equipment similar to that described in Edil et al. (2002). This equipment drops a fixed thicknes of fly ash on the surface without generating dust. The fly ash was mixed with the subgrade to a depth of 305 mm using the Wirtgen WR 2500S road reclaimer, with water being added during mixing using a water truck (see photographs in Appendix A). This mixture was compacted within 1-2 hours of mixing by a tamping foot compactor followed by a vibratory steel drum compactor. The fly-ash stabilized subgrade was cured for 7 d (days) and then overlain with 254 mm of base layer with recycle asphalt and gravel material and 203 mm concrete pavement.

Prior to stabilization, samples of the subgrade were collected and tested to determine their index properties and how addition of fly ash would affect the California bearing ratio (CBR), resilient modulus (Mr), and unconfined compressive strength (q_u) of the soil. Tests were also conducted on samples of the *in situ* stabilized soils to determine if similar improvements in properties were obtained during construction. Dynamic cone penetrometer (DCP) and soil stiffness gauge (SSG) was used to measure the strength and stiffness near the surface of the stabilized subgrade. Falling weight deflectometer tests (FWD) were conducted to evaluate the overall improvement in stiffness achieved through stabilization. Three lysimeters were installed in the test section to collect leachate percolating through the subgrade and overlying materials, and concentrations of trace elemental contaminants in the leachate are regularly analyzed. Instrumentations were used to monitor the soil water content, temperature, and air temperature.

2. TEST SECTIONS AND MATERIALS

US12 is a major east-west arterial route that serves as an alternate route to the Interstate system for carrying traffic from the Twin Cities to the Chicago area. US12 is also one of the primary truck routes in Dane and Jefferson counties. The previous roadway between the village of Cambridge and city of Fort Atkinson had narrow lanes and shoulders and a number of steep hills and sharp curves. Deterioration of both the asphalt surface of the roadway and the concrete base is widespread. To address these issues, WisDOT reconstructed 15.1 km of US12 between the US12/18 intersection north of the village of Cambridge to the WIS 26 interchange west of Fort Atkinson from April 2004 to January 2005.

During the construction (prior to October 2004), a selected portion of US12 between Cambridge and Fort Atkinson was used as a test section for fly ash stabilization to improve the subgrade. The length of this portion is 610 m. Fourteen stations in a 30.5-m segment of US12 were evaluated in this study. All measurements were made and all samples were collected along the centerline between the two lanes in the study area. In the control section, the thickness of base layer with recycled asphalt and gravel material is 305 mm.

2.1. Subgrade Properties Prior to Mixing Fly Ash

Disturbed samples of subgrade soil (≈ 20 kg each) were collected from a depth between 0-0.5 m at the fourteen stations during construction (Fig. 1). Tests were conducted on these samples to determine index properties, soil classification, water content, dry unit weight, compaction characteristics, and CBR.

A summary of the properties of the subgrade is shown in Table 1. Particle size distribution curves for the subgrade are shown in Fig. 2. All fourteen soils are broadly graded.

The subgrade consists of lean clay (CL), clayey sand (SC), and poorly graded sand with silt (SP-SM) according to the Unified Soil Classification System (USCS). However, highly plastic organic clay (CH by USCS) is present in one region (Station 594+00). According to the AASHTO Soil Classification System, most of subgrade soils at this site are A-7-6 with a group index (GI) larger than 10 and A-6 with GI larger than 2. Four of the coarser-grained subgrade soils are classified as A-2-6 (Stations 582+00, 590+00, 610+00, and 614+00) and have GI < 1. CBR of the subgrade soils ranges from 2 to 48 (mean = 26), indicating that the subgrade ranges from soft to very stiff. The *in situ* water content of the subgrade soils was approximately 2% dry of optimum water content based on standard compaction effort (ASTM D 698).

2.2. Fly Ash

Fly ash from Columbia Power Station in Portage, Wisconsin was used for stabilization. Chemical composition and physical properties of the fly ash are summarized in Table 2 along with the composition of typical Class C and F fly ashes. The calcium oxide (CaO) content is 23%, the content of SiO_2 + Al_2O_3 + Fe_2O_3 is 55.5%, and the loss on ignition is 0.7%. According to ASTM C 618, Columbia fly ash is a Class C fly ash.

Results of Water Leach Tests (ASTM D 3987) on Columbia fly ash (Sauer et al 2005) suggest that it meets WI NR 538.22 requirements for use in confined geotechnical fill, such as the stabilized road materials in this project. Chemical analysis of Boron was not conducted on the Water Leach Test leachate, which would be needed to confirm NR 538.22 compliance for these uses.

2.3. Fly-Ash Stabilized Subgrade

Profile of pavement structure in the fly-ash stabilization test section is shown in Fig. 3a. Water content and unit weight of the compacted FASS were measured at each station using a nuclear density gage (ASTM D 2922) immediately after compaction was completed. Grab samples (≈ 20 kg) of FASS were also collected at these locations and were compacted into a CBR mold (114 mm inside diameter x 152 mm height) and a resilient modulus mold (102 mm inside diameter x 203 mm height) to the unit weight measured with the nuclear density gage. Three lifts were used for the CBR specimens and six lifts were used for the M_r specimens. After compaction, the specimens were sealed in plastic and stored at 100% humidity for curing (7 d for CBR specimens, 14 d for M_r and q_u specimens). These test specimens are referred to henceforth as 'field-mix' specimens. Because of the cementing effects of the fly ash, index testing was not conducted on the fly-ash stabilized subgrade.

Undisturbed samples of fly ash stabilized subgrade were also collected after compaction using thin-wall sampling tubes. These samples were cured at 25 ^oC and 100% relative humidity for 14 d. However, some samples broke after extrusion rendered them useless. Similar problems with samples collected with thin-wall tubes have been reported for fly-ash stabilized soils (Edil et al. 2002), cement-stabilized wastes (Benson et al. 2002), and fly-ash stabilized recycled pavement materials (Li et al. 2006). The remained useful undisturbed samples were extruded from the thin-wall sampling tubes and sealed in plastic and stored at 100% humidity curing 14 d for M_r and q_u tests.

Specimens of fly-ash stabilized subgrade were also prepared in the laboratory using samples of the subgrade soils and fly ash collected during construction. Subgrade soils were collected from Station 580+00 representing CL/CH soil (A-7-6), from Station 582+00 representing SC soil (A-6), and from Station 614+00 representing SP-SM soil (A-2-6). These specimens, referred to henceforth as ‘laboratory-mix’ specimens, were prepared with 12%, 15%, and 18% fly ash (dry weight) at the water content of 2% dry, 0%, 3% wet, and 7% wet of optimum water content (Table 4). The laboratory-mix specimens were compacted and cured using the procedures employed for the field-mix specimens. A similar set of specimens was prepared with subgrade soils only (no fly ash) using the same procedure, except for the curing phase.

3. LABORATORY TEST METHODS

3.1. California Bearing Ratio

The CBR tests were conducted in accordance with ASTM D 1883 after 7 d of curing (field-mix or laboratory-mix fly-ash stabilized subgrade) or immediately after compaction (subgrade soils). The specimens were not soaked and were tested at a strain rate of 1.3 mm/min. The 7-d curing period and the absence of soaking are intended to represent the competency of the subgrade when the concrete pavement is placed (Bin-Shafique et al., 2004). Data from the unsoaked CBR tests were not intend as a measure of stiffness of the fly-ash stabilized subgrade and are not for use in pavement design with fly-ash stabilized subgrade.

3.2. Resilient Modulus and Unconfined Compression Tests

Resilient modulus tests on the fly-ash stabilized subgrade and subgrade soils were conducted following the methods described in AASHTO T292 after 14 d of curing (fly-ash stabilized subgrade) immediately after compaction (subgrade soils). The 14-d curing period is based on recommendations in Turner (1997), and is intended to reflect the condition when most of the hydration is complete (Edil et al., 2006). The loading sequence for cohesive soils was used for the fly-ash stabilized subgrade as recommended by Bin-Shafique et al. (2004) and Trzebiatowski et al. (2004) for soil-fly ash mixtures. Subgrade soils were tested using the loading sequence for cohesive soils. Two specimens of field-mix fly-ash stabilized subgrade split horizontally after curing. These specimens were trimmed to an aspect ratio of 1 prior to testing. All other specimens had an aspect ratio of 2.

Unconfined compressive strength was measured on specimens of fly-ash stabilized subgrade after the resilient modulus tests were conducted. Only those specimens having an aspect ratio of 2 were tested. The strains imposed during the resilient modulus test may have reduced the peak undrained strength of the fly-ash stabilized subgrade. However, strains in a resilient modulus test are small. Thus, the effect on peak strength is believed to be negligible.

A strain rate of 0.21%/min was used for the unconfined compression tests following the recommendations in ASTM D 5102 for compacted soil-lime mixtures. No standard method currently exists for unconfined compression testing of materials stabilized with fly ash.

4. FIELD METHODS

4.1. Environmental Monitoring

The environmental monitoring program consists of monitoring the volume of water draining from the pavement, concentrations of trace elements in the leachate, temperatures and water contents within the pavement profile, and meteorological conditions (air temperature, humidity, and precipitation). Monitoring of the pavement began in October 2004 and is still being conducted.

Leachate draining from the pavement was monitored using three pan lysimeters installed in the fly ash test section (West one adjacent to Station 580+00 and East one adjacent to Station 615+00) and control section (adjacent to Station 578+00). A profile of the road layers and lysimeter is shown in Fig 3a, and layout and dimensions of the three pan lysimeters are shown in Fig.3b. Depth to the pan lysimeter at control section is 30 cm deep, and the depth of the two lysimeters at fly ash section is 60 cm deep. Each lysimeter consists of a 1.5-mm-thick linear low density polyethylene (LDPE) geomembrane overlain by a geocomposite drainage layer (geonet sandwiched between two non-woven geotextiles). Subgrade soil was directly placed over the lysimeter at the control section using the conventional procedure. For the two lysimeters at the fly ash test section, the mixing of fly ash and subgrade soil could not be conducted in the lysimeter to prevent damage of geomembrane and geocomposite. Instead, the two lysimeters were filled with mixture of fly ash and subgrade soil that had been mixed at an adjacent location on the site. The soil mixed with fly ash was placed over the lysimeter immediately after mixing, and then was compacted following the same procedures used for the remainder of the fly ash section. Photographs showing the lysimeter are in Appendix B.

Water collected in the drainage layer is directed to a sump plumbed to a 120-L polyethylene collection tank buried adjacent to the roadway at each lysimeter. The collection tank is insulated with extruded polystyrene to prevent freezing. Leachate that accumulates in the collection tank is removed periodically with a pump. The volume of leachate removed is recorded with a flow meter; a sample for chemical analysis is collected; and the pH, Eh, and EC of the leachate are measured. Volumes of leachate are normalized as pore volumes of flow (PVF), calculated as the volume of leachate divided by the total pore volume of the subgrade layer directly over the lysimeter. The elemental analysis samples are prepared by filtering with a 0.2 μm filter and preservation with nitric acid to pH < 2.

Leachate samples were analyzed by inductively coupled plasma-mass spectrometry (ICP-MS) or inductively coupled plasma- optical emission spectrometry (ICP-OES) following the procedure described in USEPA Method 200.8 and SW-846. Analysis was conducted for the following elements: Ag, Al, As, Ba, Be, Cd, Co, Cr, Cu, Fe, Mn, Ni, Pb, Sb, Se, Tl, and Zn.

Air temperature and relative humidity (RH) are measured with a HMP35C temperature/RH probe manufactured by Campbell Scientific Inc. (CSI). A tipping bucket rain gage (CSI TE 525) is used to measure precipitation. Subsurface temperatures and water contents are monitored at three depths in the control section: 355 mm below ground surface (bgs) (mid-depth of the recycled asphalt) and 810 and 1117 mm bgs (subgrade). Subsurface temperatures and water contents are monitored at three depths in the fly ash section (beneath the west side lysimeter): 330 mm below ground surface (bgs) (mid-depth of the recycled asphalt), 609 mm bgs (mid-depth of the fly-ash stabilized subgrade) and 914 mm bgs (subgrade). Type-T thermocouples are used to monitor temperature and CSI CS616 water content reflectometers (WCRs) are used to monitor volumetric water content. The WCRs were calibrated for the materials on site following the method in Kim and Benson (2002). Data from the meteorological and subsurface sensors are collected with a CSI CR10 datalogger powered by a 12-V deep-cycle battery and a solar panel. Data are downloaded from the datalogger via telephone modem. Photographs of the instrumentation are included in Appendix B.

4.2. Mechanical Evaluation of Pavement Materials

Strength and stiffness of the fly-ash stabilized subgrade were measured with a soil stiffness gauge (SSG), a dynamic cone penetrometer (DCP), a rolling weight deflectometer (RWD), and a falling weight deflectometer (FWD). Photographs of the testing are included in Appendix A. Testing with the SSG, DCP, and RWD was conducted directly on the FASS after 7 d of curing. FWD testing was conducted four times after the concrete pavement was placed on August 2004, May 2005, August 2006, and June 2007. A final testing was planned for 2008 but could not be undertaken because the WisDOT equipment was broken. The RWD testing was unsuccessful due to problems with the instrumentation and is not discussed further; otherwise, it could provide information about how uniform the constructed stabilized layer was.

The SSG tests were conducted in accordance with ASTM D 6758 using a Humboldt GeoGauge. Two measurements were made at each station within a 0.1-m radius. These measurements are deviated by less than 10%. Thus, the mean of the two stiffness measurements is reported herein. DCP testing was conducted at each station in accordance with ASTM D 6951 using a DCP manufactured by Kessler Soils Engineering Products Inc. The dynamic penetration index (DPI) obtained from the DCP was computed as the mean penetration (mm per blow) over a depth of 150 mm.

FWD tests were conducted at each station by WisDOT in August 2004 (2 months after construction) and annually in 2005, 2006 and 2007 using a KUAB 2 m-FWD following the method described in ASTM D 4694. Moduli were obtained from the FWD deflection data by inversion using MODULUS 5.0 from the Texas Transportation Institute. Because of the difficulties of back-analyzing the moduli of sublayers under a stiff concrete layer, an alternative analysis method, ANN (Artificial Neural Network) developed by Bayrak and Ceylan (2008) was also used.

4.3. Pavement Distress Surveys

A pavement distress survey was conducted by the Wisconsin Department of Transportation personnel in June 2009 over the section of USH 12 from St. 550+90 to St. 715+84 covering both the east and west-bound lanes. This segment of USH 12 is essentially the same segment where the FWD surveys was conducted and includes mostly the fly ash stabilized subgrade segments with controls at both ends. Additionally, the pavement distress surveys were conducted at STH 60 where a fly ash stabilized subgrade was employed under an asphaltic concrete flexible pavement for a period of 8 years (2001-2008). The procedure followed is given in the WisDOT document entiled “PDI Survey Manual.” The survey provides 3 measures of pavement condition (PDI, IRI, and rut depth). Pavement Distress Index, PDI is based on a multi-attribute multiplicative model. There are 11 independently rated distress factors (e.g., alligator/block cracking, transverse cracking, longitudinal cracking, surface raveling, rutting, etc.). The index is an algebraic result incorporating the scaled values of these factors. International Roughness Index, IRI is used to define the characteristic of the longitudinal profile of a traveled wheeltrack and constitutes a standardized roughness measurement. It is expressed in m/km and is the ratio of a standard vehicle’s accumulated suspension motion divided by the distance traveled. Rut depth (reported in inch) measures the longitudinal depression in the wheel path relative to the surrounding surface and indicate permanent deformation due to accumulation of plastic strains.

5. RESULTS

5.1. Environmental Data

5.1.1. Meteorological and Subsurface Conditions

Air and soil temperatures between October 2004 and August 2008 are shown in Fig. 4. The air temperature ranged from -27 and 34^{o}C during the monitoring period, with sub-freezing temperatures occurring between December and April each year. Temperature of the fly-ash stabilized subgrade and the subgrade ranged between -7^{o}C and 32 ^{o}C and varied seasonally with the air temperature. The magnitude and frequency of variation diminishes with depth, which reflects the thermal damping provided by the pavement materials. The soil temperatures in the control section have similar distribution as in the fly ash section.

Frost penetrated to approximately 0.6 m below ground surface (bgs) each year, as illustrated by the drop in temperature below 0 ^{o}C at 330 mm below ground surface (bgs) and at 609 mm bgs and the drops in volumetric water content at 609 mm bgs when the soil temperature falls below 0 ^{o}C (volumetric water contents are not reported in Fig. 5 for periods when freezing was established). These apparent drops in water content reflect freezing of the pore water. The water content measured by WCRs (water content reflectometers) is determined by measuring the velocity of an electromagnetic wave propagated along the probe. The velocity of the wave varies with the apparent dielectric constant of the soil, which is dominated by the dielectric constant of the water phase. When the pore water freezes, the dielectric constant of the water phase drops significantly, which appears as a drop in water content in WCR data (Benson and Bosscher 1999).

Higher water contents were recorded in the FASS than the subgrade early on until 2006. However, in 2006 volumetric water content in the fly ash layer was reduced and comparable to that in the subgrade. This may be a consequence of hydration reactions as the 2006 precipitation record is similar to that of 2005 for the first 7 months of the year (Fig. 5a). The annual variation in water content is small, with the volumetric water content of the FASS varying within 8% and the subgrade within 3%. Higher water contents are recorded in the summer months, when greater precipitation occurs.

5.1.2. Trace Elements in Lysimeter Drainage

The drainage rates of the leachate collected in the three lysimeters are shown in Fig. 6. The volumes collected from the lysimeters show that peak drainage generally occurs in the spring and early summer with a secondary peak in autumn. The lowest drainage occurs during winter when the subgrade is often frozen, and during a period in July and/or August.

Peak drainage of 0.46 mm/day occurred in the East lysimeter in the month following installation, with maximum drainage rates during subsequent seasonal peak flows of 0.44, 0.40, 0.15 and 0.23 mm/day. Total drainage in the East lysimeter is 2.78 m^3, or 0.88 pore volumes of flow (PVF), with an average of 0.29 PVF/year. The West lysimeter had low drainage rates in the month following installation (0.00 to 0.07 mm/day), with maximum drainage rates during subsequent seasonal peak flows of 0.23, 0.40, 0.15 and 0.23 mm/day. Total drainage in the West lysimeter is 1.53 m^3, or 0.58 PVF, with an average of 0.19 PVF/year. Peak drainage of 0.40 mm/day occurred in the Control lysimeter in the month following installation, with maximum drainage rates during subsequent seasonal peak flows of 0.38, 0.40, 0.23 and 0.33 mm/day. Total drainage in the Control lysimeter is 2.48 m^3, or 2.33 pore volumes of flow (PVF), with an average of 0.78 PVF/year. The highest drainage rates occur in the Control lysimeter, with both fly ash stabilized subgrades having lower rates. The West lysimeter has the lowest drainage rate.

During the monitoring period, pH of the drainage of the East and West lysimeters has been mildly to moderately basic (7.1-9.1), and the Control lysimeter drainage has been mildly basic (6.9-7.5). The Control lysimeter drainage has been predominantly in oxidizing conditions (Eh of -41 to 241 mV). The West lysimeter has been consistantly in oxidizing conditions (Eh of 7 to 321 mV), while the East drainage has fluctuated between oxidizing and reducing conditions (Eh of -369 to 254 mV). A summary of the pH and Eh data is in Appendix C.

Trace element concentrations in the sampled drainage exceed at least once the WI NR 140.10 Groundwater Enforcement Standard for five elements (Fig. 7). Arsenic (As) exceeds the enforcement standard in both the fly ash test lysimeters and the control lysimeter; however the concentrations are higher in the lysimeters in the fly ash stabilized section. Lead (Pb) and Thallium (Tl) exceed the enforcement standard in one of the fly ash test lysimeters and the control lysimeter. Selenium (Se) and Nickel (Ni) exceed in one of the fly ash test lysimeters, but not in the control lysimeter.

Additionally, in both fly ash test lysimeters the non-enforcable WI NR 140.10 Groundwater Preventative Action Limits for Cadmium (Cd), Cobalt (Co), and Chromium (Cr) were exceeded (Fig. 8). The non-enforcable limit for Antimony (Sb) was exceeded in both fly ash test lysimeters and the control lysimeter.

All other elements tested did not exceed either any of the limits. Long-term leachate quality is also investigated at STH 60 where a number of working platform test sections were constructed using different materials and techniques including a fly ash working platform (subbase) constructed by stabilizing the existing subgrade with Class C fly ash (Appendix D). A similar observation to that of US 12, however, is also observed at STH 60 (Ag, Cd, and Se exceeded enforcement limits and Cr, As, Ba, Pb, Sb, and Tl exceeded preventive action limits). There was also exceedence of the limits at the control section lysimeters. These concentrations are the effluent concentrations, studies indicate attenuation occurs during transport through subsurface to the groundwater (Bin-Shaffique et al. 2002).

5.2. Mechanical Properties of Subgrade and Fly Ash Stabilized Subgrade

5.2.1. Laboratory Test Data

CBR, M_r, and q_u of the compacted untreated subgrade and field-mixed FASS are summarized in Table 3. M_r tests were also conducted on both the undisturbed subgrade and the undisturbed field-mixed FASS samples obtained 75-mm thin-wall Shelby tubes and are given in Table 3.

The CBRs of the compacted untreated subgrade and field-mixed FASS samples (after 7 days of curing) along the alignment of the project are shown in Fig. 9a. There is no systematic variation in CBR of the subgrade and the field-mixed FASS along the alignment, suggesting that the variability in the CBR is more likely due to heterogeneity in the material rather than systematic variation in site conditions or construction methods. CBR of the compacted untreated subgrade ranges from 2 to 48 (mean = 26), and the field-mixed FASS has CBRs between 5 and 26 (mean = 14). FASS has lower CBR than the subgrade compacted at its *in situ* water content to its *in situ* density. This finding is unusual, compared to other studies with fly ash stabilization (Bin-Shafique et al. 2004, Li et al. 2006, Edil et al. 2002, Edil et al. 2006, Trzebiatowski et al. 2004).

CBRs of the untreated subgrade and field-mixed FASS in the three groups of soil types are shown in Fig. 9b. The lower of CBR of the FASS is observed in every group of soil type including the fine-grained soils and coarser-grained soils, except in Station 590+00. There is not relation between the soil type of subgrade and the decreasing CBR trend after fly ash stabilization.

Fig. 10 shows the *in situ* water content and dry unit weight of the subgrade and the FASS samples. The *in situ* water content and dry unit weight of the subgrade were measured with nuclear density gauge prior to construction when dry weather was prevalent. However, during construction, there were several storms occurring before fly ash was spread on the graded subgrade. To avoid the construction on the wet ground, the fly ash was spread and mixed with subgrade and had to wait 24 hours after the heavy rains. Furthermore, a water tank was still connected to the road reclaimer perhaps providing additional water during mixing. Thus, some of water content of the FASS is higher than the subgrade, and even higher than the optimum water content (Fig. 10a). The dry unit weights of the subgrade and the FASS are comparable and reasonably uniform as shown in Fig. 10b, making water content the primary factor affecting mechanical properties in early stages of curing. Higher water content in the FASS compared to the natural subgrade persisted until 2006 as shown by the field monitoring data (Fig. 5b). The field-mixed FASS samples were prepared using the measured dry unit weight of FASS by the nuclear density gage.

Resilient moduli of subgrade and field-mixed FASS are summarized in Table 3 and shown in Fig. 11. These M_r correspond to a deviator stress of 21 kPa, which represents typical conditions within the base course of a pavement structure (Tanyu et al. 2003, Trzebiatowski et al. 2004). Complete M_r curves are included in Appendix E. As observed for CBR, there is no systematic variation in M_r along the alignment. Comparison of the M_r of the compacted subgrade and the field-mixed FASS in Fig. 11a indicates a similar trend as observed relative to CBR. For the subgrade, the M_r ranges between 58 and 219 MPa (mean = 100 MPa), whereas the field-mixed FASS has M_r between 37 and 130 MPa (mean = 73 MPa). M_r of the subgrade and field-mixed FASS in the three groups of soil type is shown in Fig. 11b. The lower M_r of the FASS samples is observed in every group of soil type including the fine-grained soils and coarser-grained soils, except in Station 590+00. As with CBR, the decrease in M_r can be attributed to higher water content of the FASS.

Resilient moduli, corresponding to a deviator stress of 21 kPa, of the undisturbed subgrade and undisturbed FASS samples are summarized in Table 3 and shown in Fig. 12. The undisturbed FASS samples were collected after 7-days of fly ash compaction. There is no systematic variation in M_r along the alignment. The M_r of the undisturbed subgrade samples is lower than that of the undisturbed FASS samples in Fig. 12a. For the undisturbed subgrade, the M_r ranges between 34 and 42 MPa (mean = 38 MPa), whereas it ranges between 60 and 129 MPa (mean = 82 MPa) for the undisturbed FASS. However, there is only limited number of undisturbed samples and only one set of the samples is from the same station (St. 594+00) the other samples are not paired. Therefore, a strong conclusion is hard to make based on this data. M_r of the undisturbed subgrade and undisturbed FASS in the three groups of soil types is shown in Fig. 12b. The higher M_r of the FASS is demonstrated in fine-grained soils (Group 1). Data in Group 2 and 3 are missing at some of the stations because some of the undisturbed subgrade and the undisturbed FASS samples were broken during extrusion from the thin-wall sampler.

The M_r of the field-mixed FASS samples (mean=71 MPa) is reasonably close to that of the undisturbed FASS samples (mean =82 MPa) within the context of the variation observed in each group (Table 3).. Thus, the field-mixed FASS can be considered to be an effective method of assessing the *in situ* soil stiffness.

Unconfined compressive strengths are summarized in Table 3 and shown in Fig. 13 for the compacted subgrade and the field-mixed FASS samples. As with CBR and M_r, there is no systematic variation in q_u along the alignment. Comparison of the q_u for the compacted subgrade and the undisturbed field-mixed FASS samples in Fig. 13a indicates that adding fly ash decreased the q_u in some soils but increased in others (Fig. 13b). For the subgrade, the q_u ranges between 92 and 458 kPa (mean = 194 kPa), whereas it ranges between 43 and 410 kPa (mean = 174 kPa) for the field-mixed FASS.

5.2.2. Parametric Study

To gain a clearer understanding of the effects of water content, fly ash content, and soil type on resilient modulus, a parametric study was conducted. Laboratory-mixed FASS samples were prepared with 12%, 15%, and 18% fly ash (dry weight) at the water content (based on the soil fraction) of 2% dry, 0%, 3% wet, and 7% wet of optimum water content. Subgrade soils were collected from Station 580+00 representing CL/CH soil (A-7-6), from Station 586+00 representing SC soil (A-6), and from Station 614+00 representing SM soil (A-2-6). The fly ash content and water content ranges include the field conditions encountered at US 12 but also encountered at other sites and projects.

Resilient moduli of the laboratory-mixed FASS as a function of water content and fly ash content are summarized in Table 4 and shown in Fig. 14. The lines drawn in the Fig. 14 are trend lines within the test range and do not imply relationships. For the CL soil, M_r decreases with increasing water content for 12% fly ash content; however, with higher fly ash contents the effect of moisture content is not perceptible (Fig. 14a). On the other hand, M_r increases with increasing water content for the SM soil for all fly ash contents (Fig. 14c). For Group 2 soil (SC, A-6), the M_r is not too sensitive to water content, except perhaps for the highest fly ash content

(18%). All together, the data in Fig. 14 reflects the complex interaction between base soil, fly ash content and amount of water available. In general, clay is more sensitive to water content and its stiffness and strength decreases with increasing water content and the mechanical properties of coarser-grained soils such as silty sand are much less sensitive to water content. Increasing amounts of fly ash requires increasing amounts of water up to a certain point to complete the hydration reactions and strength gain. These competing processes dictate the outcome of the strength and stiffness gain in fly ash stabilized soils. There is not a general rule available to predict this in advance without mix design tests.

In the previous section, the stiffness (M_r) and strength (CBR, q_u) of the FASS were lower than the original subgrade and this was attributed to the high water content during the fly ash construction that was caused by heavy rain compared to the relatively dry conditions prevailed during subgrade sampling. In Fig.14, the resilient modulus of the subgrade soil as obtained from the subgrade samples compacted to the field moisture and density at the corresponding station are also shown. These values should be compared to the 12% fly ash data for each soil. This comparison shows that fly ash addition results in higher M_r at the same moisture content (up to 2.5 times) than that of the untreated subgrade. For the SC soil (Fig. 14a), it is also apparent that a lower M_r than the subgrade M_r would be obtained with higher water content for the FASS. However, Fig.14c shows that higher M_r can be obtained with higher water content for the FASS with SM base soil. Fig. 11 shows that at Station 614+00 the M_r is indeed higher for the FASS than the subgrade although the FASS had a higher water content than the subgrade (see Fig. 10).

Unconfined compressive strength of the laboratory-mixed FASS as a function of water content and fly ash content are summarized in Table 4 and shown in Fig. 15. Although the strength increases with increasing fly content in general, the trends are not as clear with respect to moisture content. The FASS unconfined strengths are 2 to 3 times higher for the FASS (with 12% fly ash) at the same water than the subgrade.

5.2.3. Field Test Data

In situ stiffness measured with the SSG and dynamic penetration index (DPI) measured with the DCP are shown in Fig. 16 and 17 for the subgrade and the FASS (after 7 d of curing). Addition of the fly ash and compaction increased the strength and stiffness appreciably, with the DPI decreasing from 33 to 15 mm/blow, on average, and the stiffness increasing from 8 to 20 MN/m, on average. The DPI and stiffness of the stabilized and compacted subgrade are also less variable than those of the subgrade.

Maximum deflections from the FWD tests for the 40-kN drop are shown in Fig. 18. Maximum deflection, which is measured at the center of the loading plate, is a gross indicator of pavement response to dynamic load. FWD tests were conducted in August 2004, May 2005, august 2006, and June 2007 to define the as-built condition as well as the conditions after over several years of winter weather exposure. Overall similar deflections were measured initially (2004, 2005, and 2006 surveys), suggesting that the FASS has essentially maintained its integrity after two years although some differences are notable. For instance, some of the peak deflections recorded after construction disappeared a year later. It is also noted that the deflections in the control section are comparable to those in the fly ash stabilized test section. The survey in 2007 gave slightly higher deflections. However, there were somewhat higher peak deflections both in the fly ash test section and the control section in 2007. Deflections are also influenced with the surface layer so direct comparisons are not always indicative of the conditions so the back-analyzed elastic moduli need to be compared.

The back calculation of the modulus of the stabilized layer is difficult when a very stiff surface layer such as concrete pavement is present. Analyses were conducted with MODULUS software as well as an alternative approach named ANN (Artificial Neural Network) as summarized in Appendix F. Both analyses indicate a degradation of modulus in 2007. The ANN analysis indicates a more gradual decrease whereas the MODULUS indicates a more rapid decrease. Based on the ANN analysis, the average coefficient of subgrade reaction is 86, 300 MN/m^3 (318 $kips/in^3$) and the average modulus of the fly ash stabilized layer is 1,262 MPa (183 ksi).

5.2.4 Freeze-Thaw and Wet-Dry Cycling

Freeze-thaw study was conducted for laboratory-mixed FASS at Station 614+00. The procedure is described in Rosa (2006). The resilient modulus and unconfined compressive strength as a function of freeze-thaw cycles are shown in Fig. 20. The M_r increased from 37 MPa to 57 MPa as the freeze-thaw cycles increased from 0 to 5, which is an unusual behavior. It is attributed to strength gain during the freeze-thaw cycles beyond the initial 7-day curing. Similar tests were conducted to evaluate wet-dry cycling. However, the test results were inconclusive due to experimental problems. More research on freeze-thaw and wet-dry testing is needed.

Long-term impacts are also investigated at STH 60 where a number of working platform test sections were constructed using different materials and techniques including a fly ash working platform (subbase) constructed by stabilizing the existing subgrade with Class C fly ash. FWD surveys were conducted in Spring and Fall from 2000 to 2007. The back-analyzed fly ash working platform moduli are given in Appendix D. Fly ash subbase had higher moduli than all other platforms including the control sections built with crushed aggregate. The moduli was also retained over 7 years of winter freezing cycles.

5.3. Pavement Distress

The pavement distress data collected in 2009 and the associated graphics for USH 12 are given in Appendix G. Pavement distress data collected 2001-2008 at STH 60 and the associated graphics are given in Appendix D. STH 60 is a flexible pavement unlike USH 12 but it has fly ash stabilized subgrade and control sections stabilized by breaker run. The results are summarized in Table 5 and 6 for both highways. Table 7 gives a summary of how the pavement survey data can be interpreted for pavement management purposes. A review of the pavement distress surveys indicate that fly ash stabilized sections perform comparable to control sections stabilized with breaker run in USH 12 4 years after construction and also over 8 years of observation in STH 60.

6. CONCLUSIONS AND RECOMMENDATIONS

A case history involving a segment of US 12 is described where Class C fly ash was used to stabilize the subgrade during construction of a rigid pavement. Additionally, long-term monitoring data collected in a flexible pavement site (STH 60) and construction data collected at a rigid pavement site (STH 32) where the subgrade was stabilized with fly ash are reported. The main case (US12) involved a 1.2-km segment US 12, immediately west of Fort Atkinson where both the construction and the post-construction behavior have been monitored from 2 years. At STH 32 (about 3.7 km from East Sauk Road to I-43) in Port Washington, Wisconsin where only the construction was monitored (Appendix H). Finally, modulus and leachate data from STH 60 near Lodi, Wisconsin (Edil et al. 2002) collected over 8 years were reported (Appendix D). All of the information generated is used in making the conclusions.

At all sites, California Bearing Ratio (CBR), resilient modulus (M_r), and unconfined compressive (q_u) tests were conducted on the subgrade samples and the FASS mixed in the field and laboratory to assess the effect of adding fly ash to the subgrade. *In situ* testing was also conducted on the subgrade and on the FASS with a soil stiffness gauge (SSG), dynamic cone penetrometer (DCP), and falling weight deflectometer (FWD).

At US 12 and STH 60, post-construction monitoring involved installation of three pan lysimeters beneath the pavement to monitor the rate of drainage and the quality of the leachate. FWD surveys were also conducted annually since construction at US 12 and STH 60 and only after construction at STH 32. Field monitoring also involved collection of the meteorological and the subsurface temperature and moisture data on a continuous basis. Additionally, a parametric study was also conducted on the three soil groups encountered at US 12 test segment to assess the influence of fly ash content, water content, and soil type on stiffness and strength.

A laboratory freeze-thaw and wet-dry cycling study was conducted on a sample of FASS. A pavement distress survey was conducted in 2009 at USH 12 fly ash section and annually at STH 60 since construction.

The following observations are made based on this investigation:

1. All of the tests consistently indicated that the stiffness and strength of the subgrade were improved significantly by fly ash stabilization at STH 32. However, this was not readily apparent at US 12 as the subgrade was substantially drier than the FASS because of several rain storms that took place during construction. Observations during construction, however, clearly demonstrated the benefit of fly ash stabilization because once the fly ash is mixed and compacted in a window of dry weather the FASS remained stiff in subsequent rain events.

2. The FASS CBR was 85, M_r 21 MPa, and q_u 454 kPa, on average at STH 32. The corresponding numbers for US 12 were CBR = 14, M_r = 73 MPa, and q_u = 174 kPa, respectively. It is noted several significantly different soil types were encountered in both sites and the fly ash contents were 12% and 10% and the water contents 12-14% and 7-14% for US 12 and STH 32, respectively.
3. The gain in stiffness and strength are typically 2-3 times due to fly ash stabilization. *In situ* stiffness measured with the SSG and dynamic penetration index (DPI) measured with the DCP also illustrated that the addition of the fly ash and compaction increased the strength and stiffness appreciably. These findings suggest that fly ash stabilization of subgrade should be beneficial during construction and, if durable, in terms of increasing pavement capacity and service life in the long-term.
4. The data also indicates a complex relationship between base soil type, amount of fly ash, and water content. For instance, in some soils (e.g., clay) stabilized with fly ash M_r decreases

when the water content of the soil increases whereas in some other soils (e.g., sandy) it increases. Therefore, a careful mix design is needed for fly ash stabilization involving all potential subgrade soils. The results from both sites also confirm that fly ash stabilization results in a relatively stiffer layer irrespective of the type of soil.

5. The M_r, of the field-mixed FASS is close to that of undisturbed samples of FASS obtained by thin-wall tube sampler. Thus, the field-mixed FASS can be considered to be an effective method of assessing the *in situ* soil stiffness.
6. Analysis of the FWD data collected after several years of freeze-thaw cycles showed some degradation in the modulus after 2 winter seasons but the modulus at the US 12 site. However, long-term FWD data collected at STH 60 indicated that the fly ash section continued to gain stiffness and had a higher stiffness than the control section.
7. A review of the pavement distress surveys indicate that fly ash stabilized sections perform comparable to control sections stabilized with breaker run in USH 12 4 years after construction and also over 8 years of observation in STH 60.
8. Percolation from the pavement varies seasonally. The concentration of 5 elements at US 12 exceeded the Wisconsin enforcement limits and 4 elements exceeded the preventive action limit. All other elements were below these limits. A similar observation, however, is also observed at STH 60. There was also exceedence of the limits at the control section lysimeters. These concentrations are the effluent concentrations, studies indicate attenuation occurs during transport through subsurface to the groundwater (Bin-Shaffique et al. 2002).

7. REFERENCES

Benson, C.H. (2002), Containment systems: Lessons learned from North American failures, Environmental Geotechnics (4th ICEG), Swets and Zeitlinger, Lisse, pp. 1095-1112.

Benson, C.H. and Bosscher, P.J., 1999. Time-domain reflectometry in geotechnics: a review. In: W. Marr and C. Fairhurst (Editors), Nondestructive and Automated Testing for Soil and Rock Properties, ASTM STP 1350. ASTM International, West Conshohocken, PA, pp. 113-136.

Bin Shafique, S., Benson, C. H., and Edil, T. B. (2002). "Leaching of Heavy Metals from Fly Ash Stabilized Soils Used in Highway Pavements." Geo Engineering Report No. 02-14, Dept. of Civil and Environmental Engineering, University of Wisconsin-Madison.

Bin-Shafique, S., Benson, C.H., Edil, T.B. and Hwang, K., 2006. Leachate concentrations from water leach and column leach tests on fly-ash stabilized soils. Environmental Engineering 23(1), pp. 51-65.

Bin-Shafique, S., Edil, T.B., Benson, C.H. and Senol, A., 2004. Incorporating a fly-ash stabilised layer into pavement design. Geotechnical Engineering, Institution of Civil Engineers, United Kingdom, 157(GE4), pp. 239-249.

Crovetti, J.A., 2000. Construction and performance of fly ash-stabilized cold in-place recycled asphalt pavement in Wisconsin, Issues in Pavement Design and Rehabilitation. Transportation Research Record, pp. 161-166.

Edil, T.B., Acosta, H.A. and Benson, C.H., 2006. Stabilizing soft fine-grained soils with fly ash. Journal of Materials in Civil Engineering, 18(2), pp. 283-294.

Edil, T.B. et al., 2002. Field evaluation of construction alternatives for roadways over soft subgrade. Transportation Research Record, No. 1786: National Research Council, Washington DC, pp. 36-48.

FHWA, 2003. Fly Ash Facts for Highway Engineers. Federal Highway Administration, US Department of Transportation, Washington D.C., FHWA-IF-03-019.

Kim, K. and Benson, C.H. (2002), Water content calibrations for final cover soils, Geo Engineering Report 02-12, Geo Engineering Program, University of Wisconsin-Madison.

Li, L, Benson, C.H., Edil, T.B., and Hatipoglu, B., 2006. Sustainable construction case history: Fly ash stabilization of recycled asphalt pavement material. Transportation Research Record, in review.

Sauer, J.J., Benson, C. H. and Edil, T. B., 2005. "Metals Leaching from Highway Test Sections Constructed with Industrial Byproducts. Geo Engineering Report No. 05-21, Department of Civil & Environmental EngineeringUniversity of Wisconsin-Madison, Madison, Wisconsin.

Sawangsuriya, A. and Edil, T.B., 2005. Use of soil stiffness gauge and dynamic cone Penetration for pavement materials evaluation. Geotechnical Engineering, Institute of Civil Engineers, United Kingdom, Vol. 158, No. GE4, pp. 217-230.

Sawangsuriya, A., Edil, T.B. and Bosscher, P.J., 2003. Relationship between soil stiffness gauge modulus and other test moduli for granular soils. Transportation Research Record, No. 1849: National Research Council, Washington D.C., pp. 3-10.

Tanyu, B., Kim, W., Edil, T., and Benson, C., 2003. Comparison of laboratory resilient modulus with back-calculated elastic modulus from large-scale model experiments and FWD tests on granular materials. Resilient Modular Testing for Pavement Components, American Society for Testing and Materials, West Conshohocken, PA. STP 1437, pp. 191-208.

Trzebiatowski, B., Edil, T.B. and Benson, C.H., 2004. Case study of subgrade stabilization using fly ash: State Highway 32, Port Washington, Wisconsin. In: A. Aydilek and J. Wartman (Editors), Beneficial Reuse of Waste Materials in Geotechnical and Transportation Applications, GSP No. 127. ASCE, Reston, VA, pp. 123-136.

Turner, J.P., 1997. Evaluation of western coal fly ashes for stabilization of low-volume roads, Testing Soil Mixed with Waste or Recycled Materials. American Society for Testing and Materials, West Conshohocken, PA. STP 1275, pp. 157-171.W

TABLES

Table 1. Properties of subgrade soils

Soil Group	Station Number	LL	PI	% Fines	GI	Classification		w_N (%)	CBR	γ_d (kN/m^3)	w_{opt} (%)	γ_{dmax} (kN/m^3)
						USCS	AASHTO					
Group 1	580+00	46	29	71	19	CL	A-7-6	NA	NA	18.9	15.0[1]	19.6[1]
	594+00	65	49	65	29	CH	A-7-6	7.8	48	20.0		
	602+00	42	23	56	10	CL	A-7-6	NA	NA	17.6		
	606+00	41	24	57	10	CL	A-7-6	12.2	16	18.6		
Group 2	586+00	40	27	43	6	SC	A-6	7.8	36	22.0	10.0[2]	20.2[2]
	598+00	38	21	50	4	SC	A-6	10.8	22	20.1		
	612+00	27	12	44	2	SC	A-6	14.3	2	17.6		
	616+00	33	18	47	5	SC	A-6	6.0	26	20.0		
	618+00	36	22	48	6	SC	A-6	7.9	28	18.9		
	620+00	25	11	47	2	SC	A-6	7.5	38	21.3		
Group 3	582+00	29	14	33	1	SP-SM	A-2-6	4.8	48	21.3	10.0[3]	19.9[3]
	590+00	31	17	35	1	SP-SM	A-2-6	14.7	4	18.2		
	610+00	27	15	35	1	SP-SM	A-2-6	6.7	NA	17.7		
	614+00	26	11	34	0	SP-SM	A-2-6	8.1	21	20.7		

LL = Liquid Limit, PI = Plasticity Index, GI = Group Index, USCS = Unified Soil Classification System, AASHTO = Association of American State Highway and Transportation Officials, w_N = In situ water content, CBR = California Bearing Ratio, γ_d = Dry unit weight, w_{opt} = Optimum water content, $\gamma_{d,max}$ = Maximum dry unit weight. Standard proctor compaction test at 594+00 (1), at 620+00 (2), and at 614+00 (3). NA is no available.

Table 2. Chemical composition and physical properties of Columbia fly ash and typical Class C and F fly ashes

Parameter	Percent of Composition			Specifications	
	Columbia	Typical* Class C	Typical* Class F	ASTM C 618 Class C	AASHTO M 295 Class C
$SiO_2 + Al_2O_3 + Fe_2O_3$, %	55.5	63	88	50 Min	50 Min
CaO (calcium oxide), %	23	24	9		
SO_3 (sulfur trioxide), %	3.7	3	1	5 Max	5 Max
Loss on Ignition, %	0.7	6	6	6 Max	5 Max
Moisture Content, %	0.09	-	-	3 Max	3 Max
Specific Gravity	2.7	-	-		

*from FHWA (2003)

Table 3. CBR, M_r, and q_u of subgrade and fly-ash stabilized subgrade (FASS)

Station Number	CBR		M_r (MPa)				q_u (kPa)	
	Subgrade	Field-Mix FASS	Subgrade	Field-Mix FASS	Undistrubed Subgrade#	Undistrubed Field-Mix FASS#	Subgrade	Field-Mix FASS
580+00	-	-	-	-	-	-	-	-
582+00	48	19	69.2	58.5*	-	-	117.8	NA
584+00	-	7	-	92.7	-	-	-	143
586+00	36	26	64.3	54.6*	-	-	138.4	NA
588+00	-	19	-	75.8	-	-	-	170.3
590+00	4	15	88.2	89.4		81.6*	91.7	168.6
592+00	-	9	-	59.1	-	87.1	-	193.9
594+00	48	19	219.3	59.9	34.1	70.3	238.5	175.9
596+00	-	19	-	74.9	-	128.9	-	185.7
598+00	22	11	121.7	80.5	37.0	-	458.0	282.5
600+00	-	16	-	130.1	-	82.4*	-	410.4
602+00	-	-	-	-	-	-	-	-
604+00	-	7	-	70.4	42.3	-	-	139.0
606+00	16	5	101.8	50.9	-	-	236.4	135.5
608+00	-	5	-	36.7	-	-	-	110.7
610+00	-	9	-	90.8	-	59.7	-	194.5
612+00	2	-	-	43.7	-	68.9*	-	124.2
614+00	21	15	57.8	NA	-	91.0	91.5	
616+00	26	18	93.3	60	-	64.8*	118.4	128.5
618+00	28	12	101.2	73.8	-	-	224.9	43.1
620+00	38	-	86.2	NA	-	-	222.7	NA

Notes: CBR = California bearing ratio, M_r = resilient modulus reported at 21 kPa deviator stress, q_u = unconfined compressive strength, hyphen indicates test not conducted, * = aspect ratio is 1:1, # = sampled by undisturbed thin-wall sampler, NA = not available because specimen damaged.

Table 4. CBR, M_r, and q_u of lab-mix fly-ash stabilized subgrade

Station Number	Soil Classification USCS	Soil Classification AASHTO	FA (%)	w (%)	w - w_{opt} (%)	M_r (MPa)	q_u (kPa)
580+00	CL	A-7-6	12	13	-2	242	450
				15	0	115	510
				18	3	98	240
				22	7	61	350
			15	13	-2	192	570
				15	0	144	360
				18	3	172	570
				22	7	105	440
			18	13	-2	122	510
				15	0	109	470
				18	3	347	990
				22	7	130	590
582+00	SC	A-2-6	12	7	-2	102	430
				9	0	134	360
				12	3	161	480
				16	7	178	650
			15	7	-2	163	660
				9	0	183	520
				12	3	303	480
				16	7	160	660
			18	7	-2	366	600
				9	0	253	1160
				12	3	208	1010
				16	7	130	850
614+00	SP-SM	A-2-6	12	8	-2	152	310
				10	0	167	1120
				13	3	153	730
				17	7	207	430
			15	8	-2	111	720
				10	0	164	1330
				13	3	241	1280
				17	7	264	810
			18	8	-2	94	430
				10	0	129	1090
				13	3	195	2390
				17	7	178	1100

Notes: FA = fly ash content by weight, w = water content, w_{opt} = optimum water content from standard proctor curve, M_r = resilient modulus reported at 21 kPa deviator stress, q_u = unconfined compressive strength.

Table 5. Comparison of Fly Ash and Control Section Pavement Conditions in 2009 at USH 12

Section	**PDI**		**IRI (m/km)**		**Rut Depth (in)**	
	East	West	East	West	East	West
Fly Ash Stabilized St. 573+75 to 622+00	6.1-11.7	0-10.9	1.77-1.97	1.40-1.86	0.06-0.09	0.06-0.09
Control 1 St. 568+00 to 573+75	6.1	6.1	1.52	1.42	0.05	0.08
Control 2 627+93 to 633+68	3.4	3.4	1.42	1.69	0.06	0.07

Table 6. Comparison of Fly Ash and Control Section Pavement Conditions in 2008 at STH 60

Section	**PDI**		**IRI (m/km)**		**Rut Depth (in)**	
	East	West	East	West	East	West
Fly Ash Stabilized	28	13	1.06	0.92	0.19	0.14
Adjacent Control	34	13	1.04	1.37	0.13	0.14
Other Two Controls	20.3-48.1	7-20.3	0.9-1.2	0.63-0.88	0.16-0.17	0.12-0.18

Table 7. Critical Pavement Distress Quantities

Pavement Type	Rating	PDI	IRI (m/km)	Rut Depth (in)
HMA Flexible (STH 60)	Satisfactory	<20	<1.5	<0.15
	Failed	>70	>2.2	>0.3
PCC Dowelled Rigid (US 12)	Satisfactory	<20	<2	<0.15
	Failed	>50	>2.6	>0.3

Source: David Friedrichs, Pavement Decision Support System, WisDOT

FIGURES

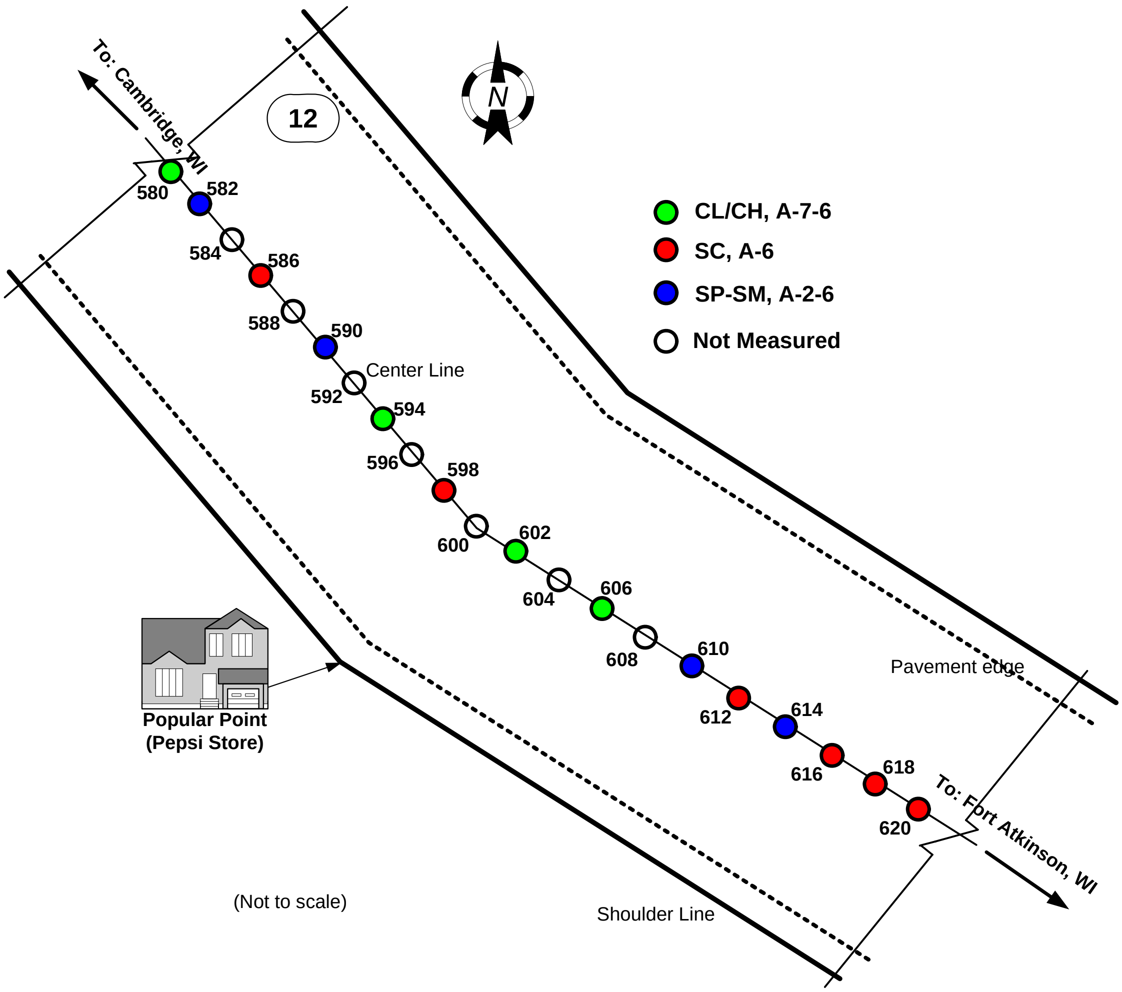


Fig. 1. Layout of test section at US12 between Cambridge and Fort Atkinson, Wisconsin (numbers represent project stationing).

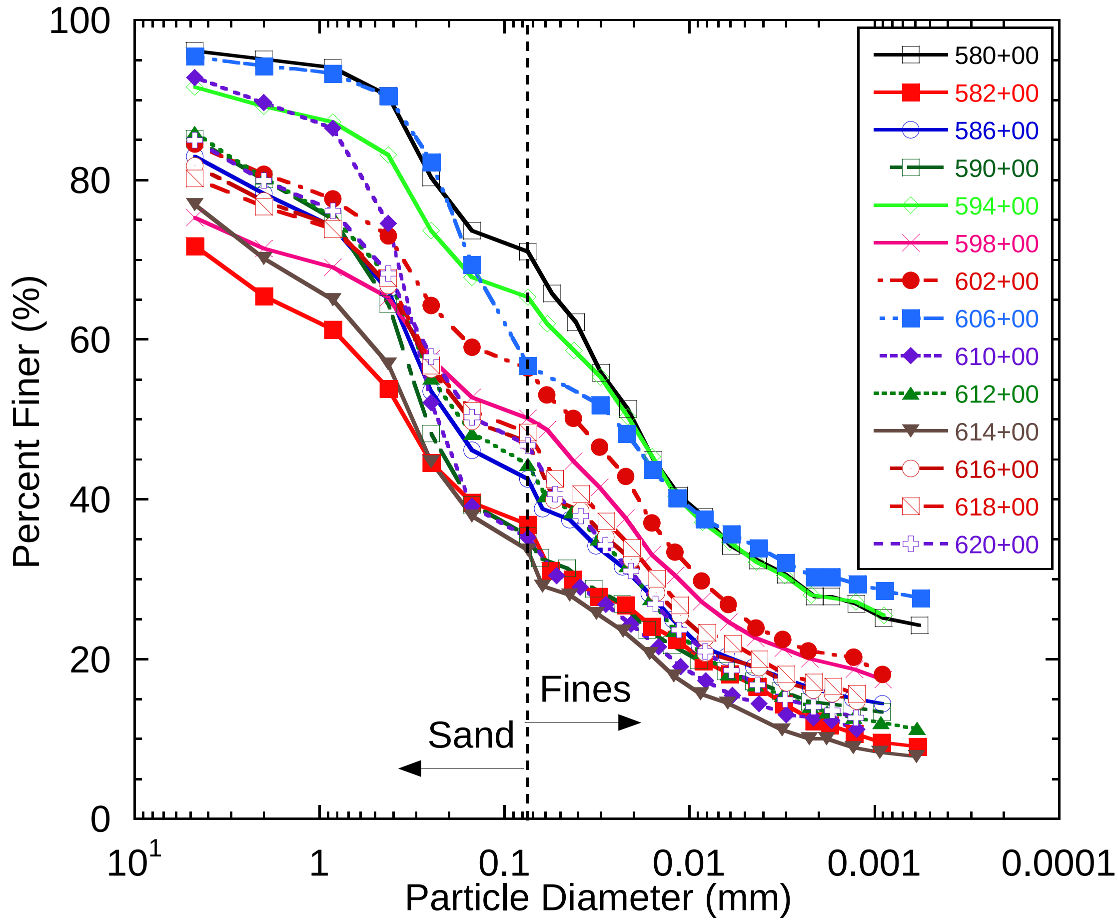


Fig. 2. Particle size distributions of the subgrade soil

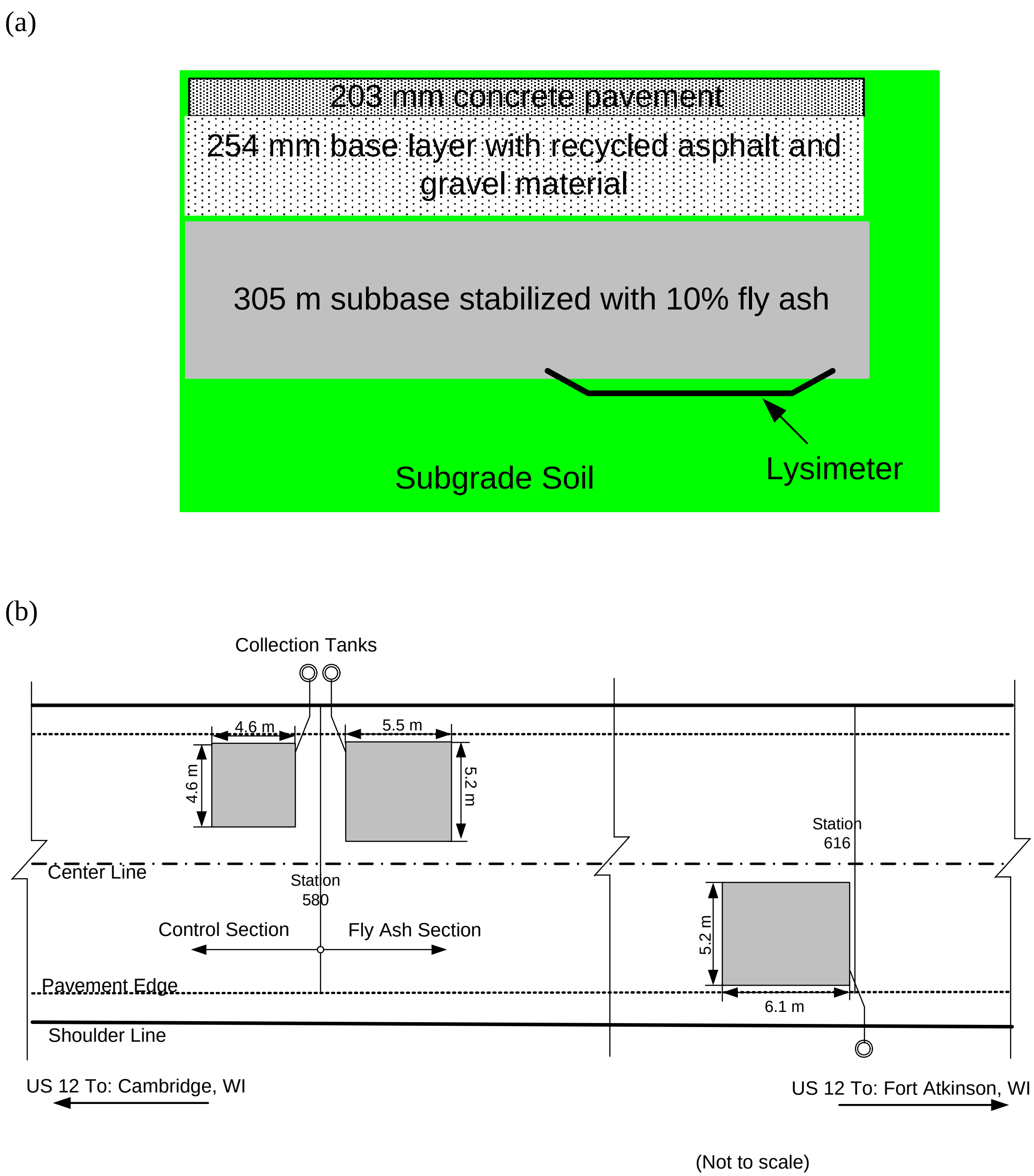


Fig. 3. Fly ash test sections at US12 site: (a) profiles of pavement structure and (b) layout of three lysimeters (positions are approximate).

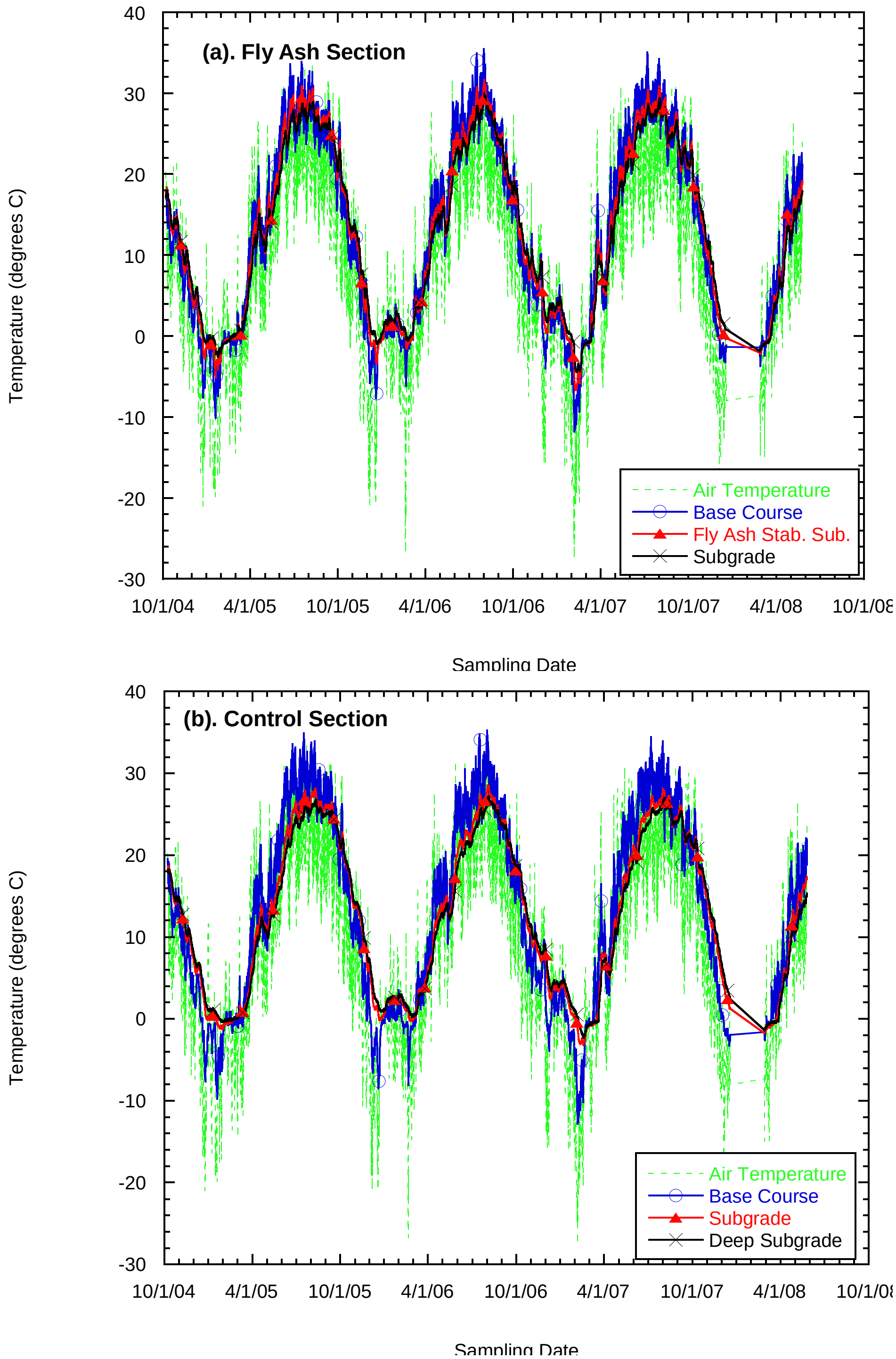


Fig. 4. Air and soil temperatures of the base course, subgrade, and fly-ash stabilized subgrade at fly ash section (a) and control section (b). Air temperature is shown in black dash line. Soil temperature measured at three depths at the fly ash section: 330 mm bgs (mid-depth in recycled asphalt), 609 mm bgs (mid-depth of fly-ash stabilized subgrade) and 914 mm bgs (subgrade). Soil temperature measured at three depths at the control section: 335 mm bgs (mid-depth in recycled asphalt), 810 mm bgs (subgrade) and 1117 mm bgs (subgrade).

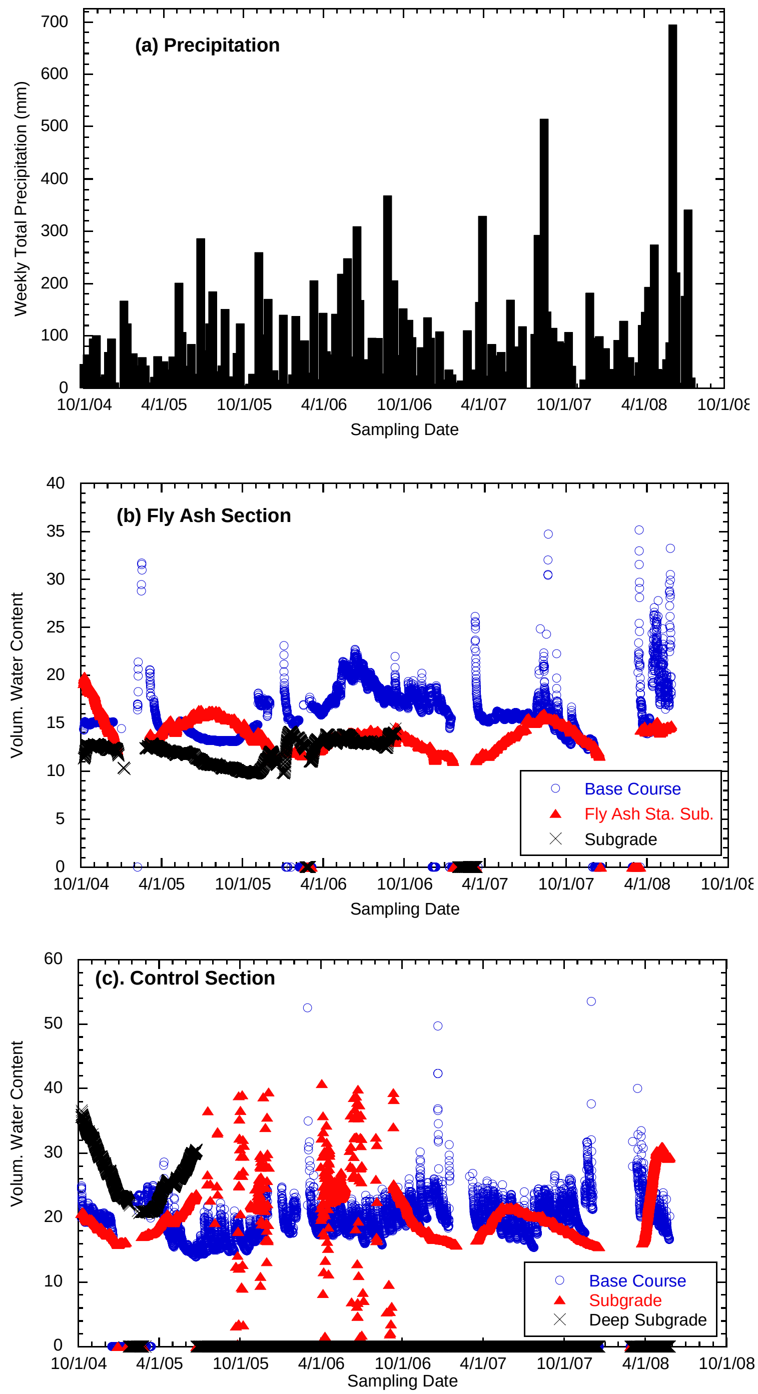


Fig. 5. Weekly Total Precipitation at Fort Atkinson, WI (a), volumetric water content of the base course, subgrade, and fly-ash stabilized subgrade at fly ash section (b) and control section (c). Volumetric water content measured at three depths at the fly ash section: 330 mm bgs (mid-depth in recycled asphalt), 609 mm bgs (mid-depth of fly-ash stabilized subgrade) and 914 mm bgs (subgrade). Volumetric water content measured at three depths at the control section: 335 mm bgs (mid-depth in recycled asphalt), 810 mm bgs (subgrade) and 1117 mm bgs (subgrade).

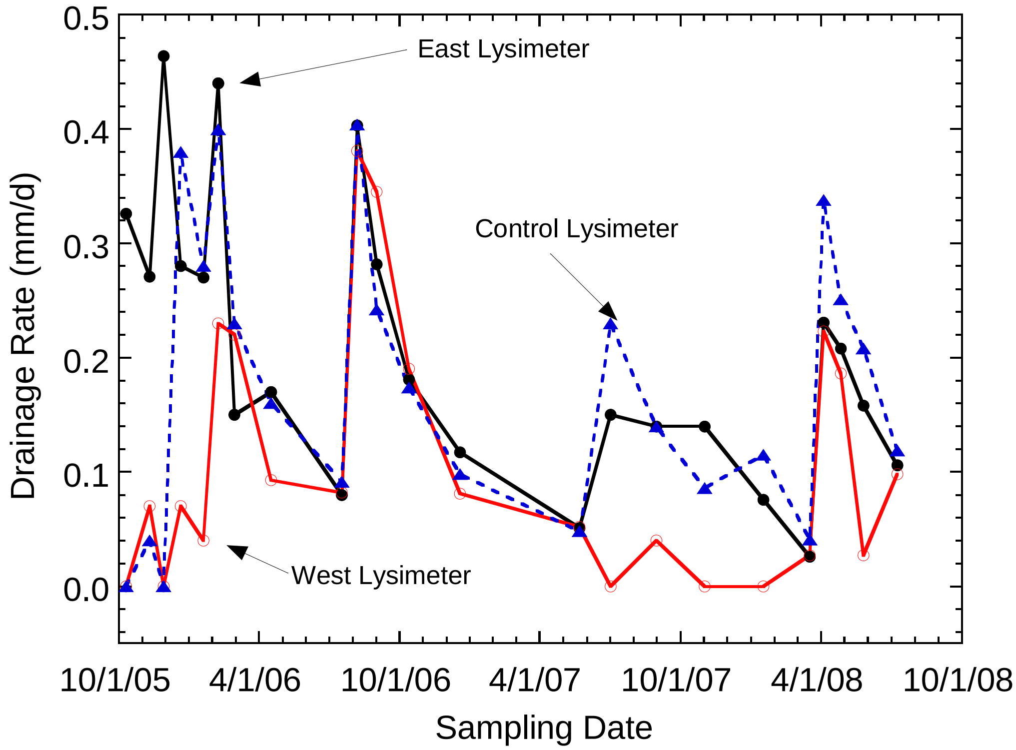


Fig. 6. Drainage from the pavement collected in the three lysimeters. Base of the two lysimeters is located at the bottom of the fly-ash stabilized subgrade layer in the west and east side, and base of the lysimeter at the control section is located at the 30 cm below base course.

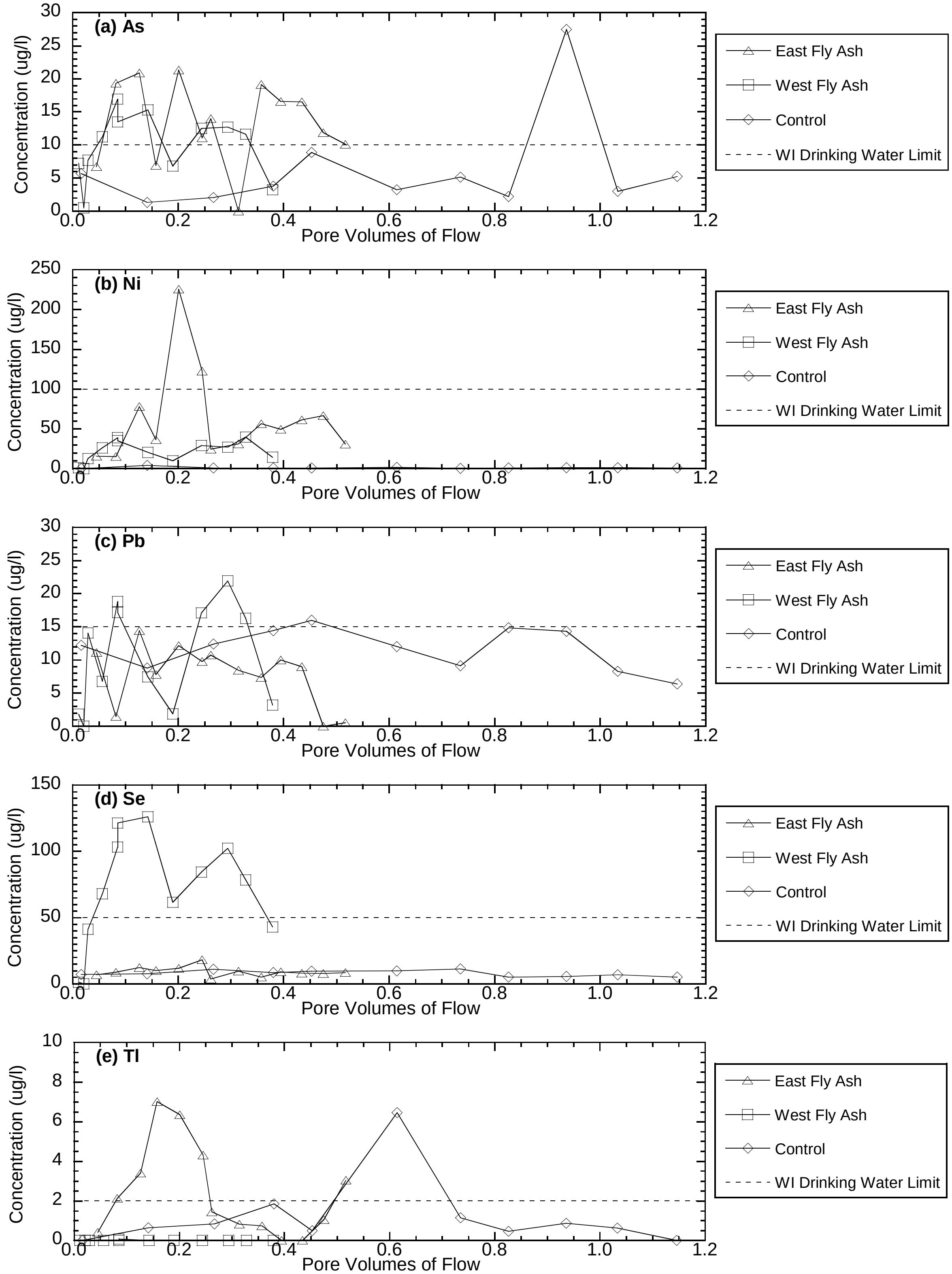


Fig. 7. Concentrations of trace elements in leachate collected in lysimeters that exceeded WI NR 140.10 Groundwater Enforcement Standard; (a) As, (b) Ni, (c) Pb, (d) Se, and (e) Tl.

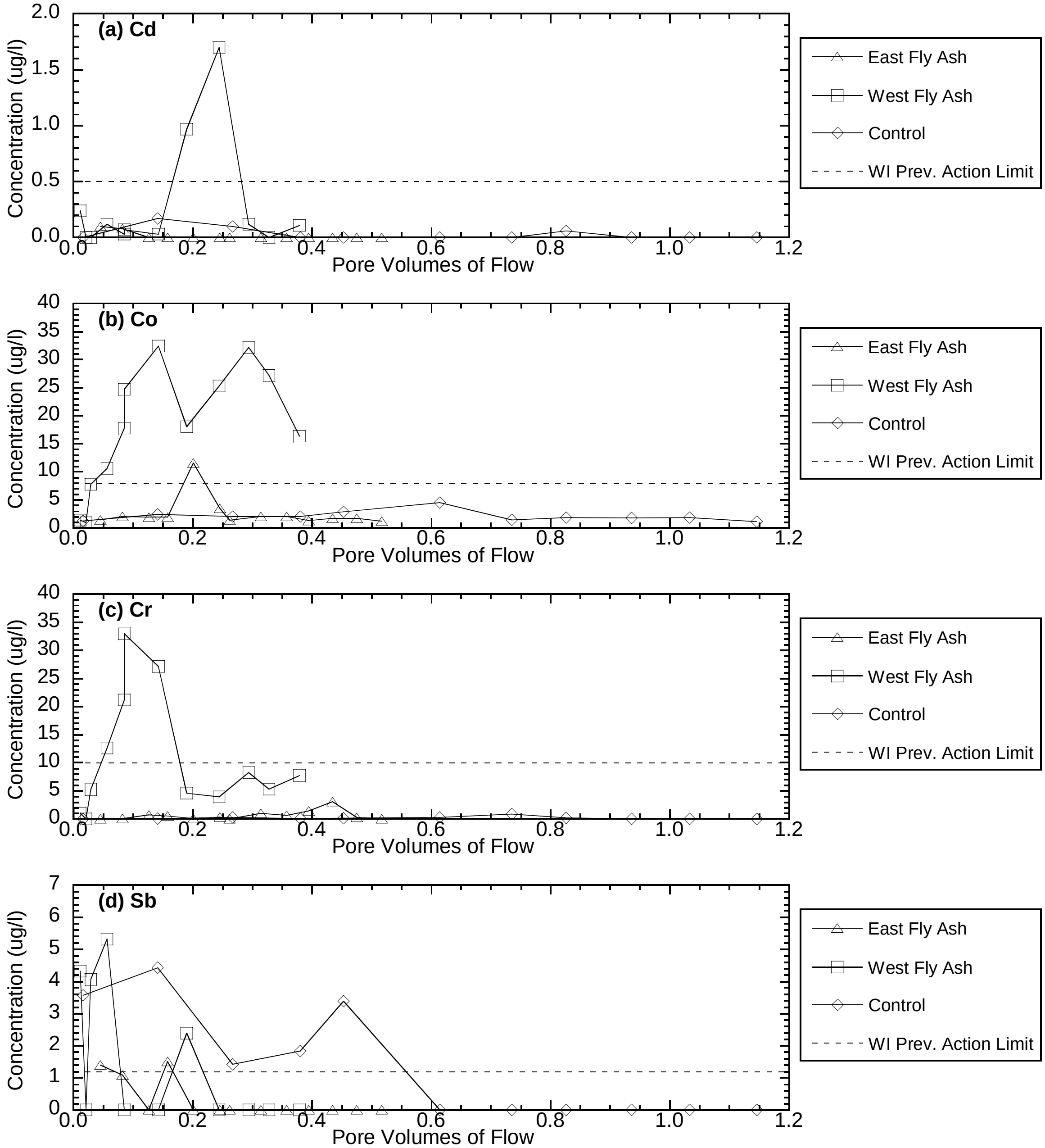


Fig. 8. Concentrations of trace elements in leachate collected in lysimeters that exceeded WI NR 140.10 Groundwater Prventative Action Limit (non-enforcable); (a) Cd, (b) Co, (c) Cr, and (d) Sb.

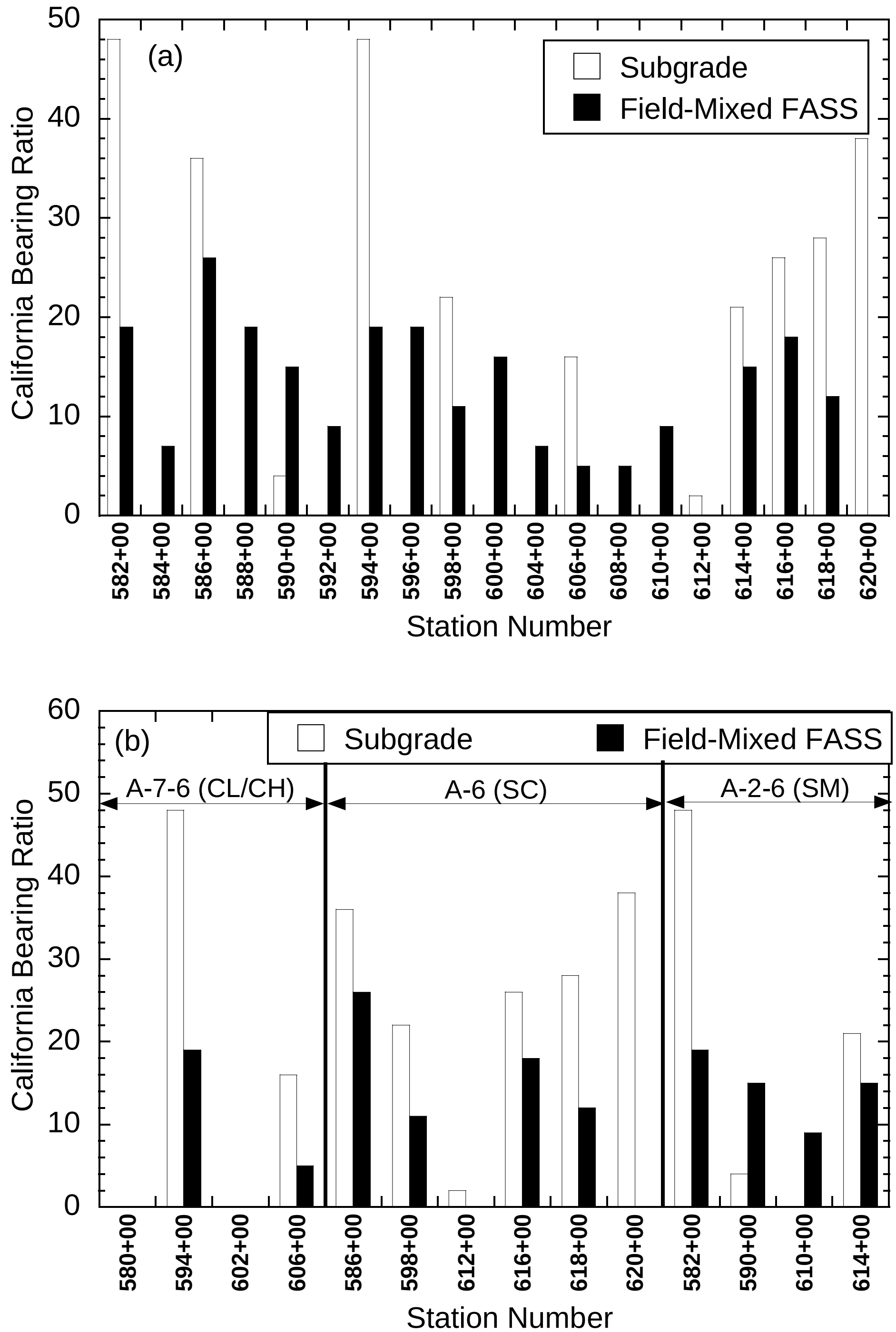


Fig. 9. California bearing ratio of subgrade and field-mixed FASS (after 7 d of curing) (a) and CBR gain at the three groups after fly ash stabilization (b).

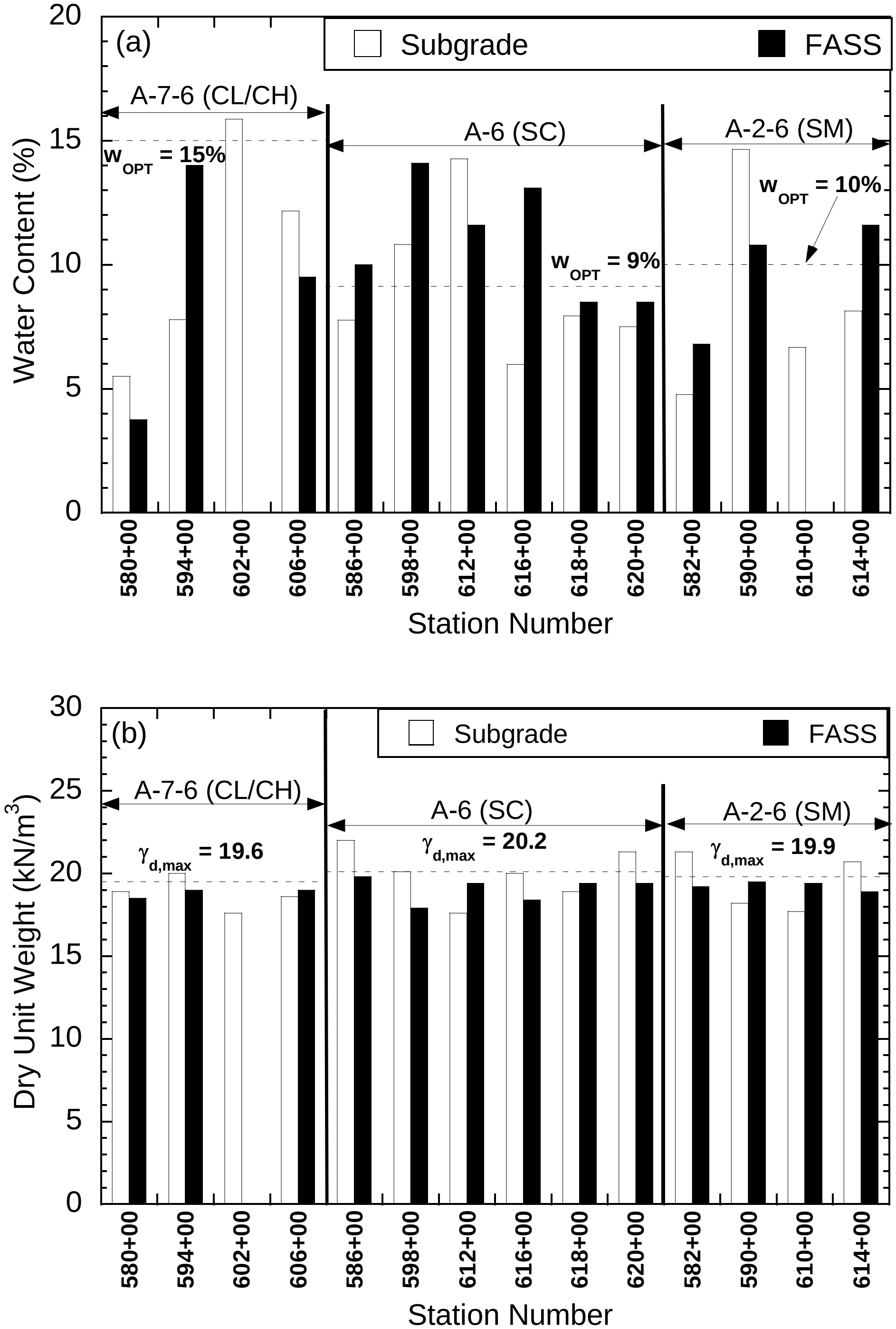


Fig. 10. Water content (a) and dry unit weight (b) of subgrade and *in situ*FASS.. The water content and dry unit weight were measured with nuclear density gauge. The water content and dry unit weight of *in situ* FASS were measured 1-3 hrs after the field compaction.

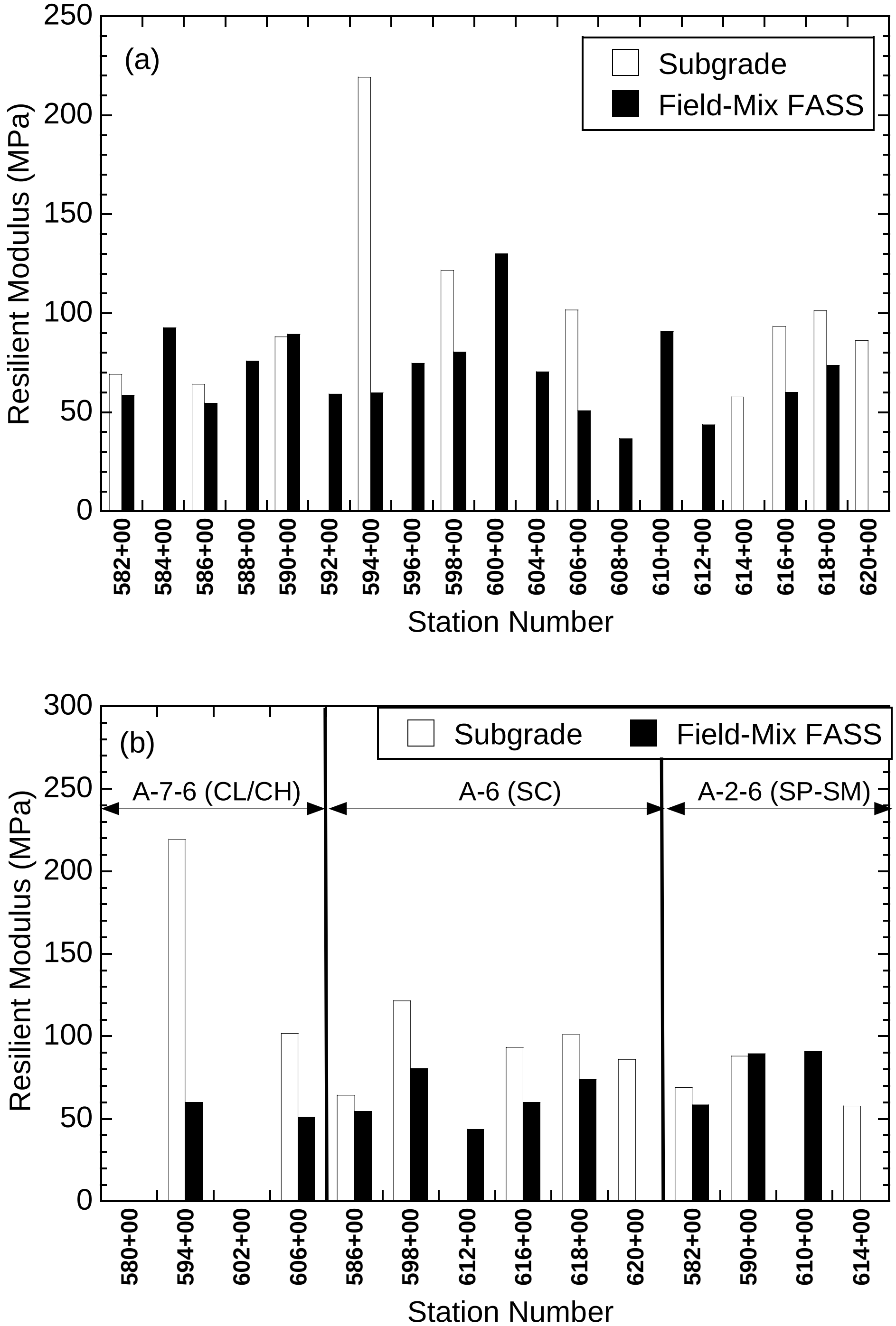


Fig. 11. Resilient modulus of subgrade and field-mix fly-ash stabilized subgrade at 21 kPa deviator stress (a) and M_r gain at the three groups after fly ash stabilization (b). Specimens cured for 14 d. For stations 582+00 and 586+00, specimens had a 1:1 height-to-diameter ratio.

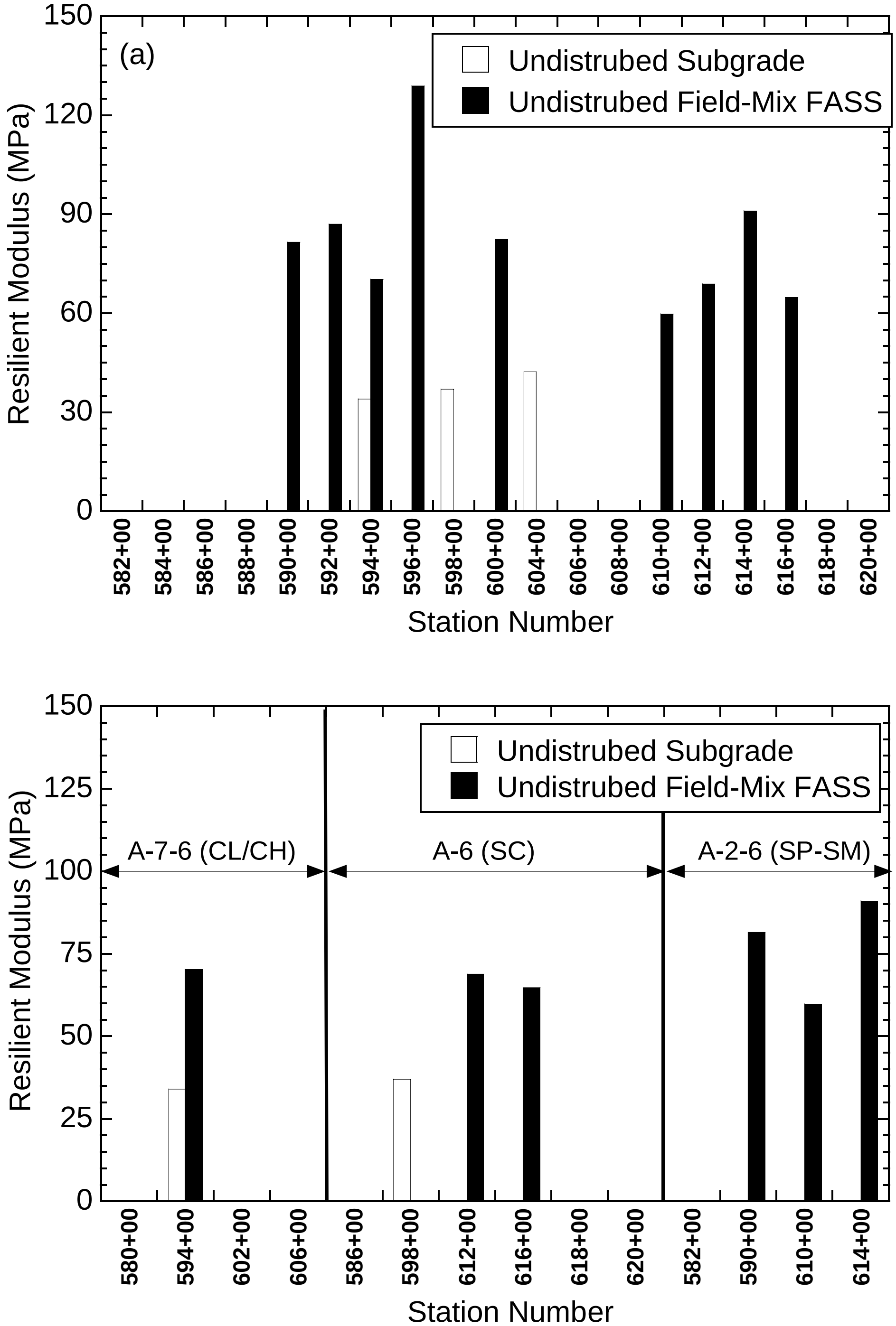


Fig. 12. Resilient modulus of undisturbed subgrade and undisturbed field-mix fly-ash stabilized subgrade at 21 kPa deviator stress (a) and M_r gain at the three groups after fly ash stabilization (b). The undisturbed subgrade and undisturbed field-mix fly-ash stabilized subgrade were collected using thin-wall sampler. Specimens cured for 14 d.

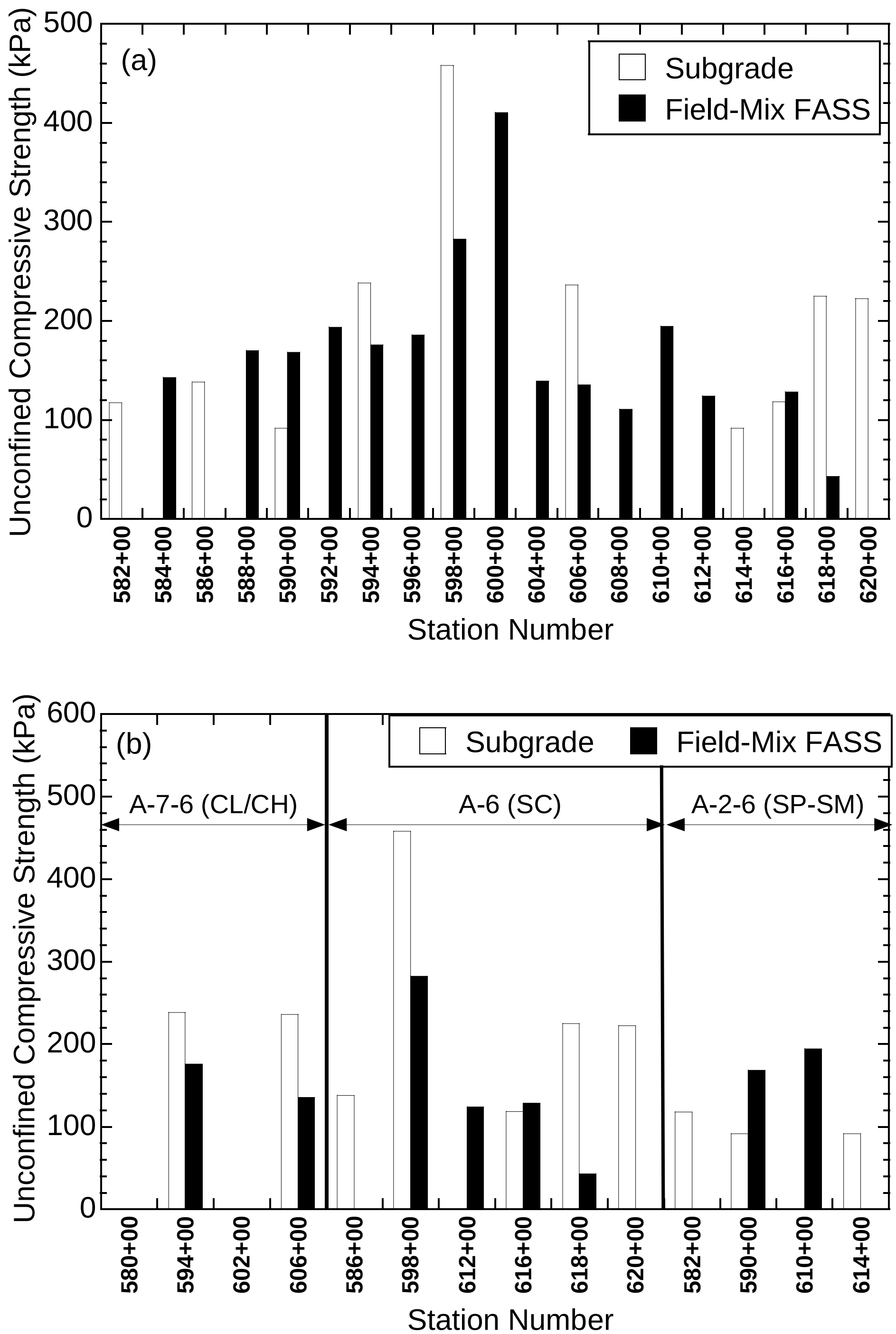


Fig. 13. Unconfined compression strength (q_u) of subgrade and field-mix fly-ash stabilized subgrade after 14 d of curing (a) and q_u gain at the three groups after fly ash stabilization (b).

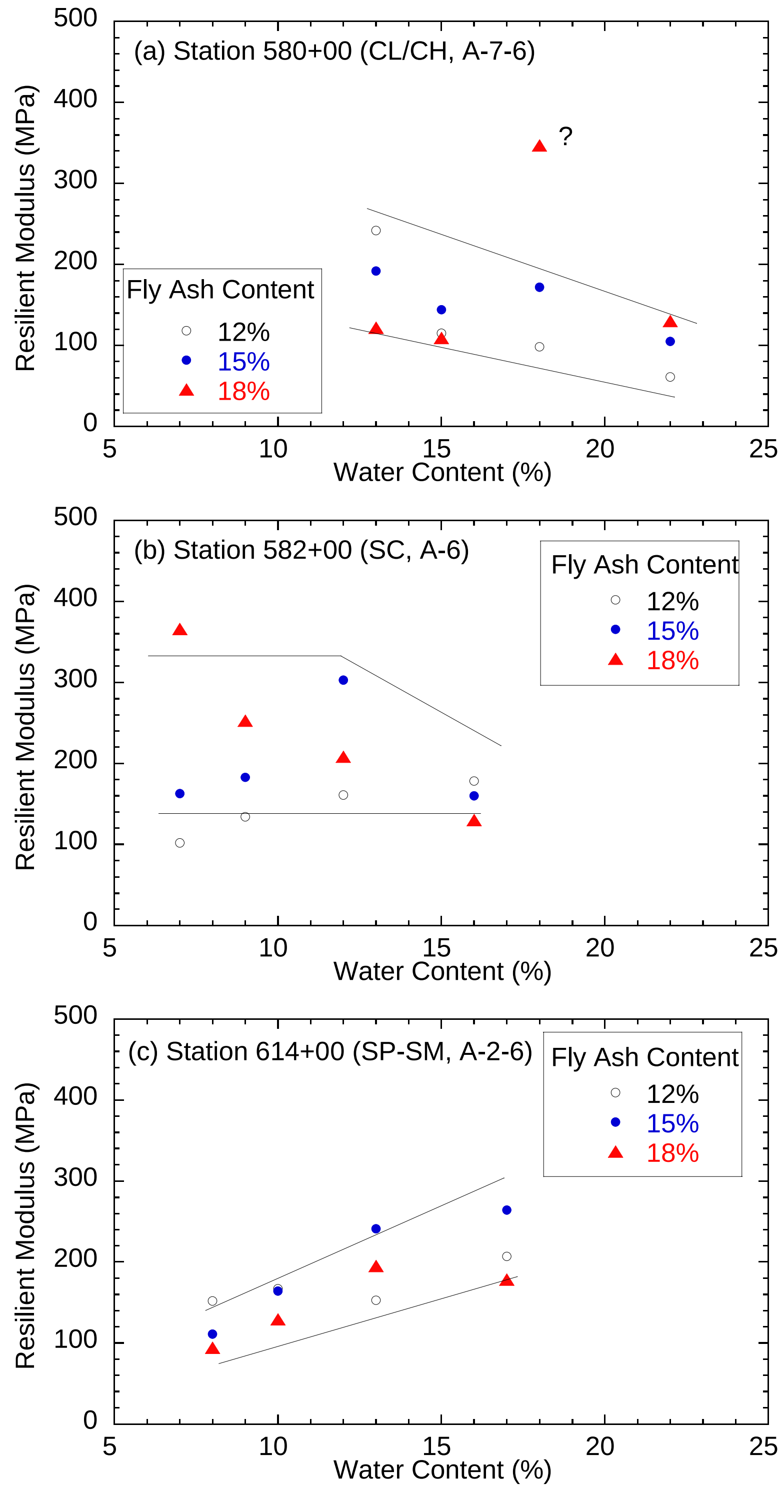


Fig. 14. Resilient modulus of laboratory-mixed fly-ash stabilized subgrade as a function of water content at 12%, 15%, and 18% fly ash content in Station 580+00 (a), Station 582+00 (b), and Station 614+00. Specimens cured for 14 d. Resilient modulus was at 21 kPa deviator stress.

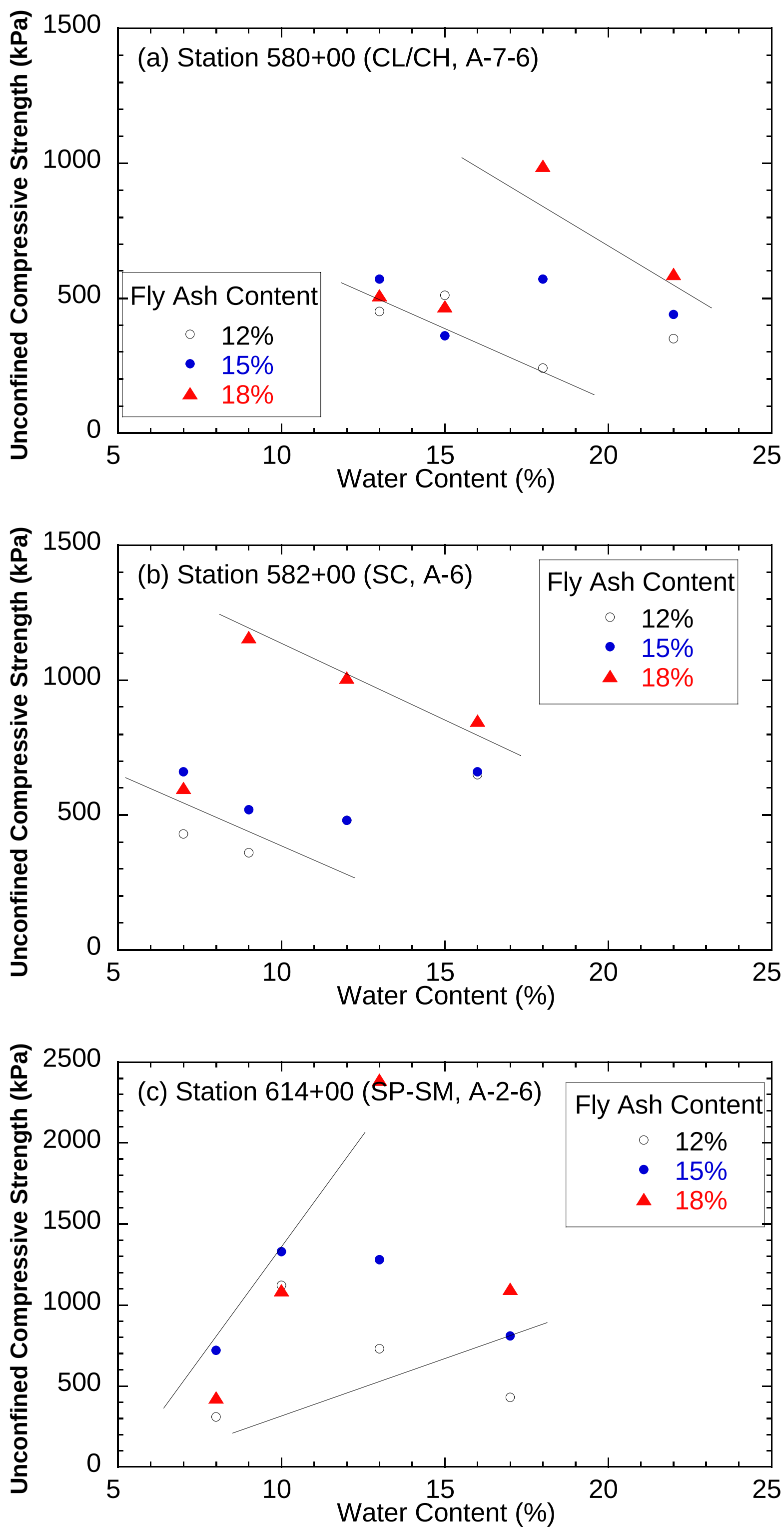


Fig. 15. Unconfined compression strength of laboratory-mixed fly-ash stabilized subgrade as a function of water content at 12%, 15%, and 18% fly ash content in Station 580+00 (a), Station 582+00 (b), and Station 614+00.

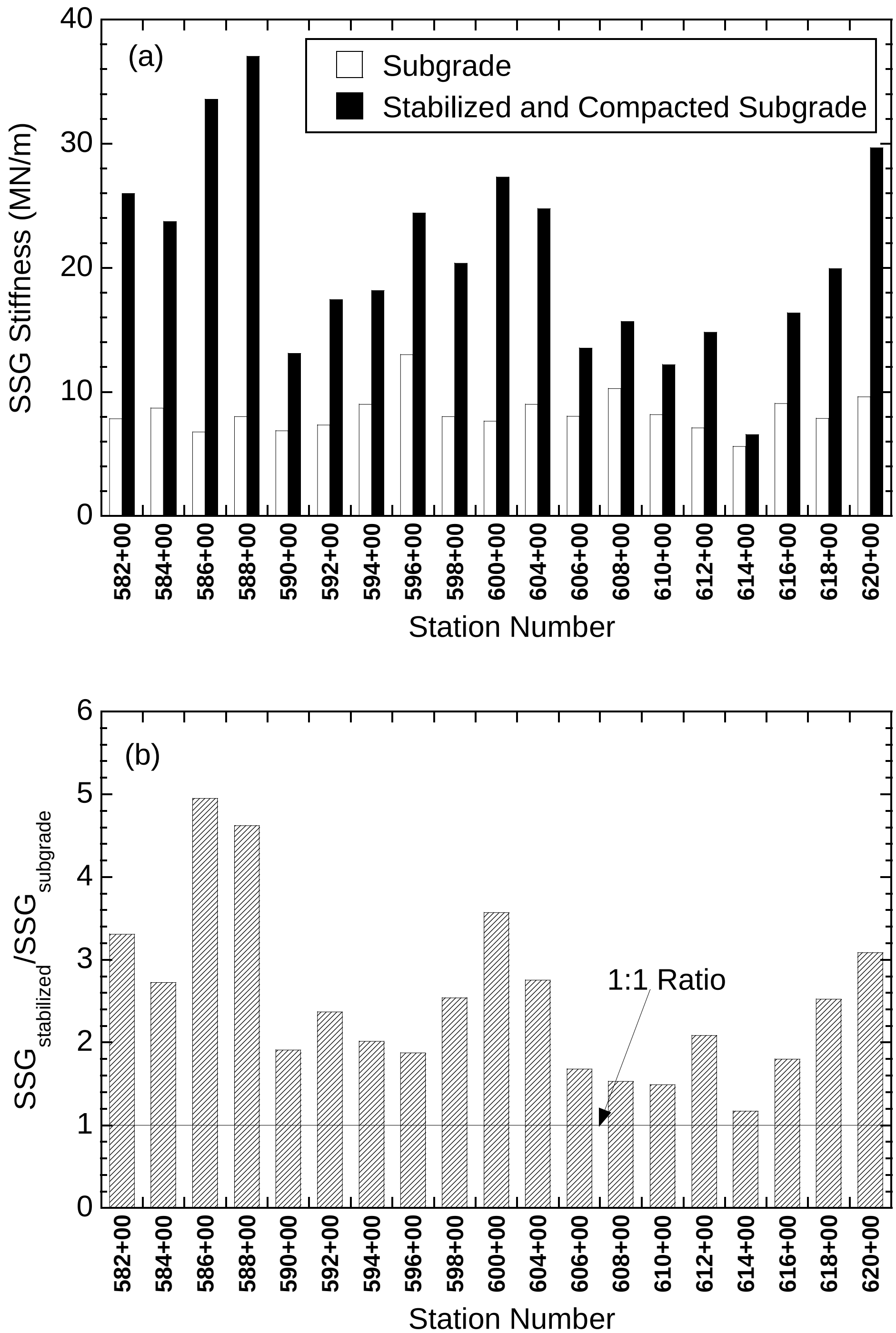


Fig. 16. Stiffness of subgrade and stabilized-compacted-subgrade after 7 d of curing (a) and stiffness gain after fly ash stabilization (b). Stiffness was measured with a SSG.

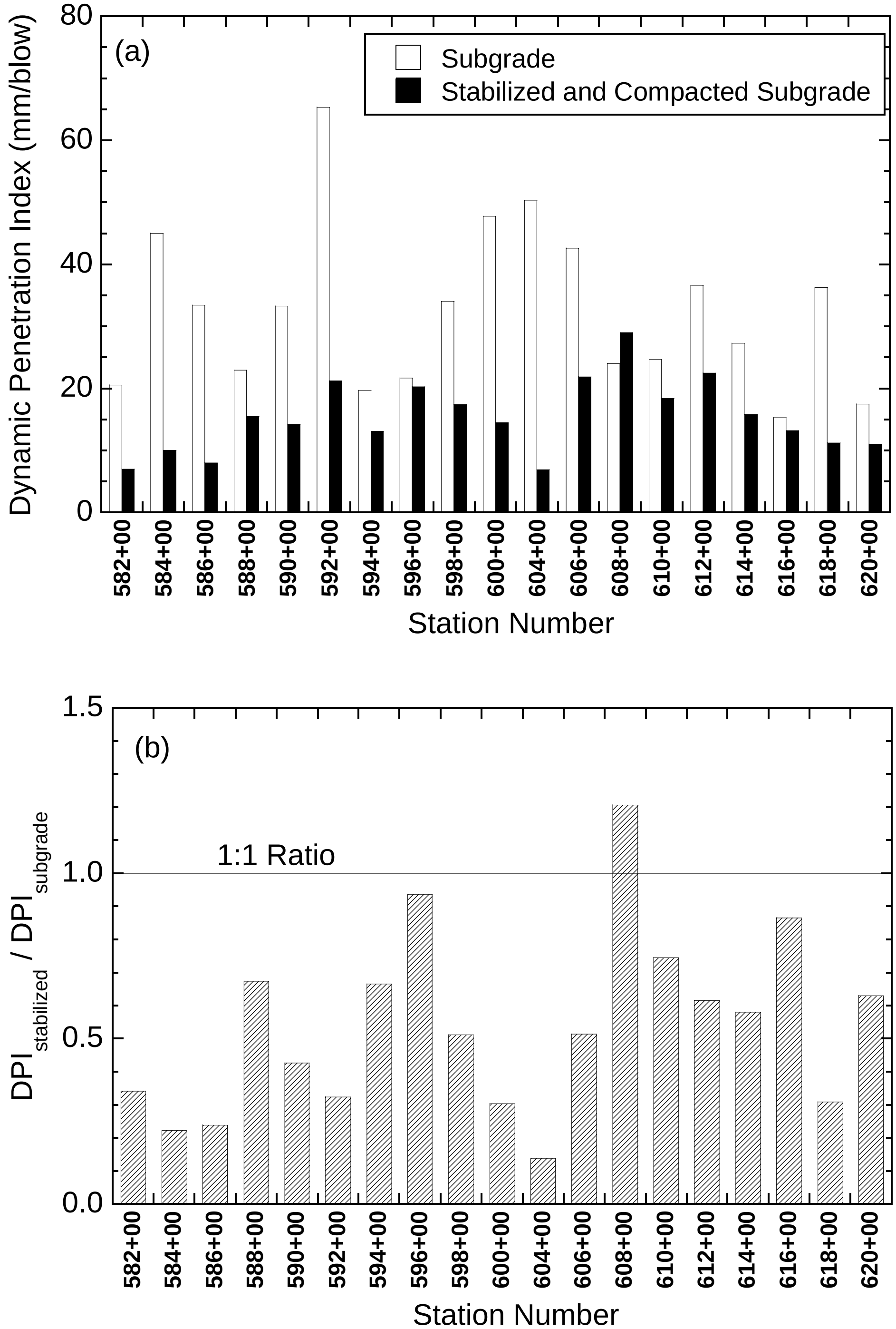
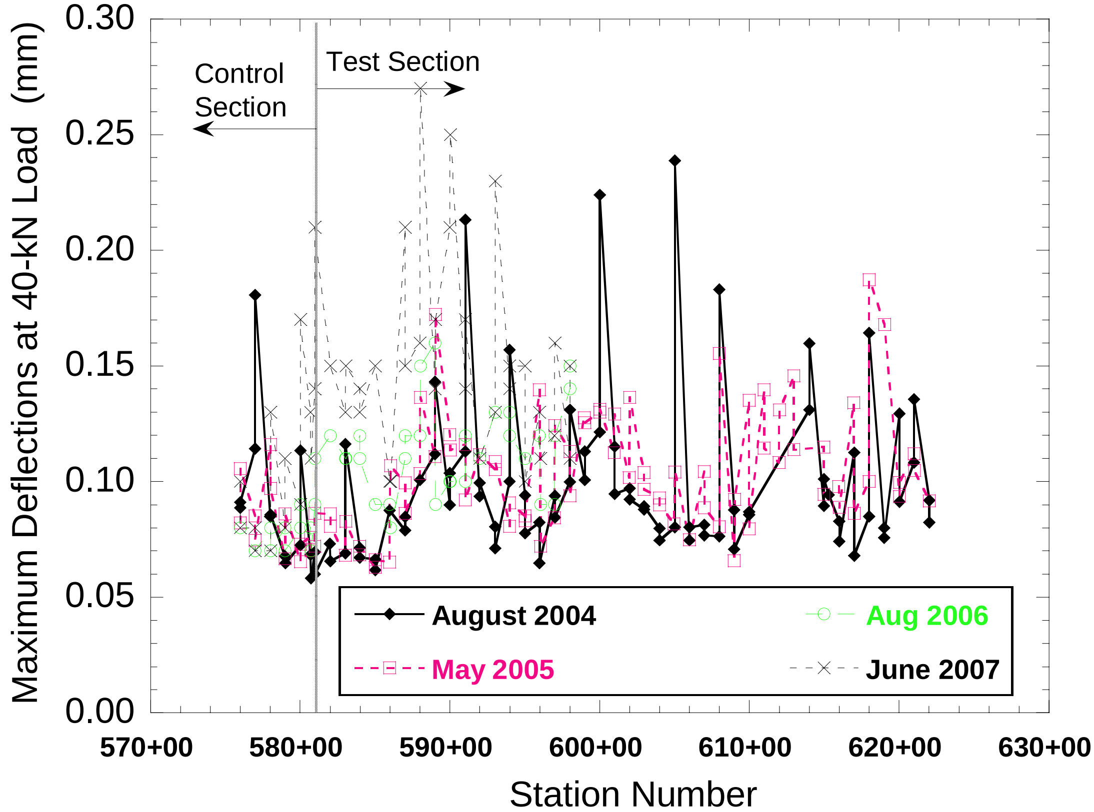


Fig. 17. Dynamic penetration index (DPI) of subgrade and stabilized-compacted-subgrade after 7 d of curing (a) and DPI gain after fly ash stabilization (b). DPI was measured with a DCP.



z

Fig. 18. Maximum deflection from the 40-kN drop for FWD tests conducted in August 2004 until June 2007

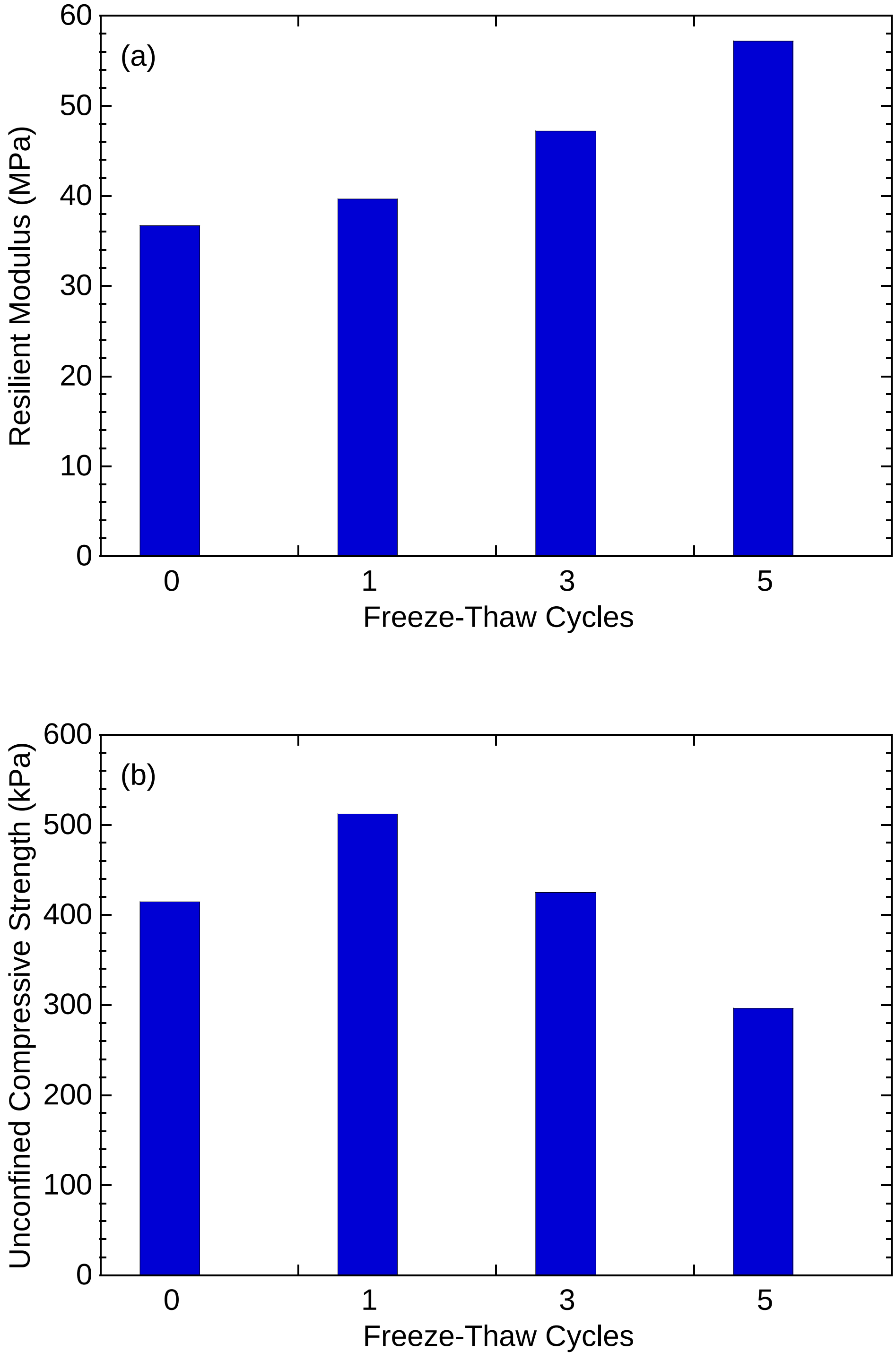


Fig. 19. Resilient modulus (a) and unconfined compressive strength (b) of laboratory-mixed FASS as a function of freeze-thaw cycles. The fly ash content is 12% by weight. The subgrade was sampled at Station 614+00.

APPENDICES

APPENDIX A

CONSTRUCTION PHOTOGRAPHS

Fig. A1. Subgrade before placement of fly ash.

Fig. A2 Lay-down truck placing fly ash on subgrade.

Fig. A3. Water truck and road-reclaimer blending fly ash, water, and subgrade.

Fig. A4. Surface of stabilized subgrade after compaction.

Fig. A5. Mid-section of road-reclaimer used to blend fly ash, water, and subgrade soil.

Fig. A6. Collecting a sample of mixture of fly ash and subgrade soil for use in laboratory testing.

Fig. A7. Measuring water content and unit weight of stabilized subgrade after compaction.

Fig. A8. Measuring stiffness using SSG

Fig. A9. Falling weight deflectometer (KUAB 2 m FWD).

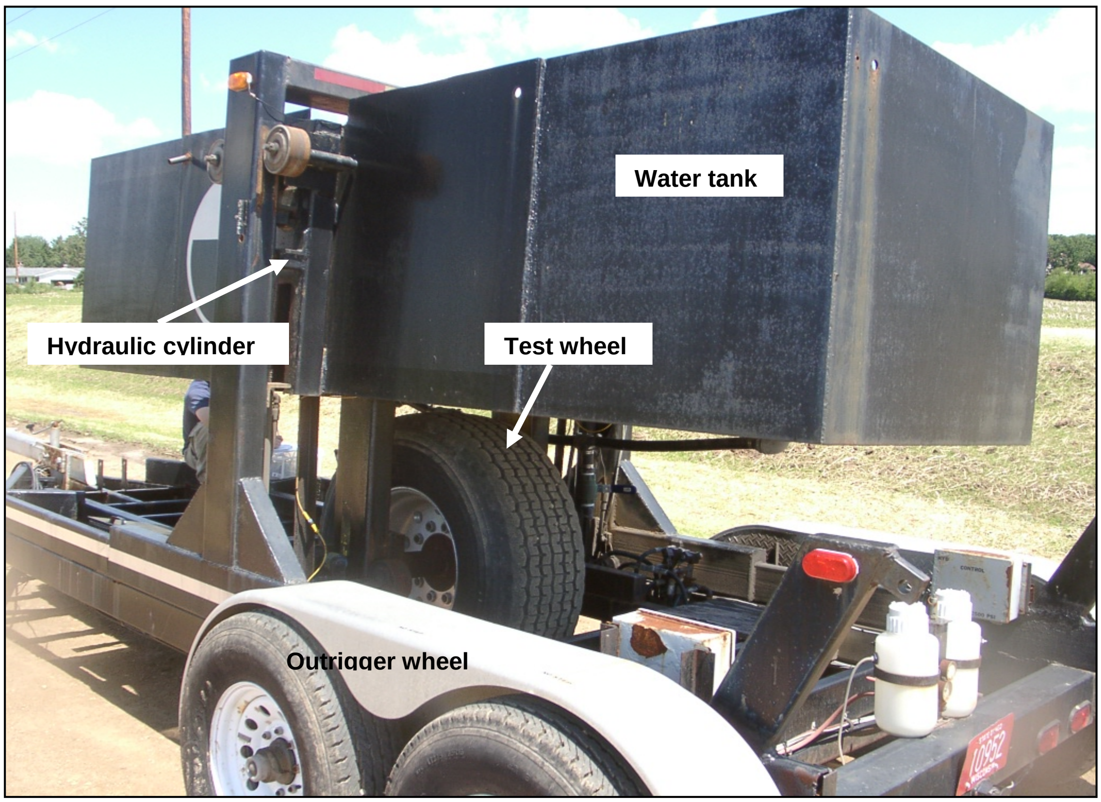


Fig. A10. Rolling Weight Deflectometer test apparatus.

APPENDIX B

PHOTOGRAPHS OF
LYSIMETER CONSTRUCTION
AND
INSTALLATION OF INSTRUMENTS

Fig. B1. Installing geomembrane for lysimeter.

Fig. B2. Installing collection tank for lysimeter.

Fig. B3. Installing water content reflectometer in subgrade.

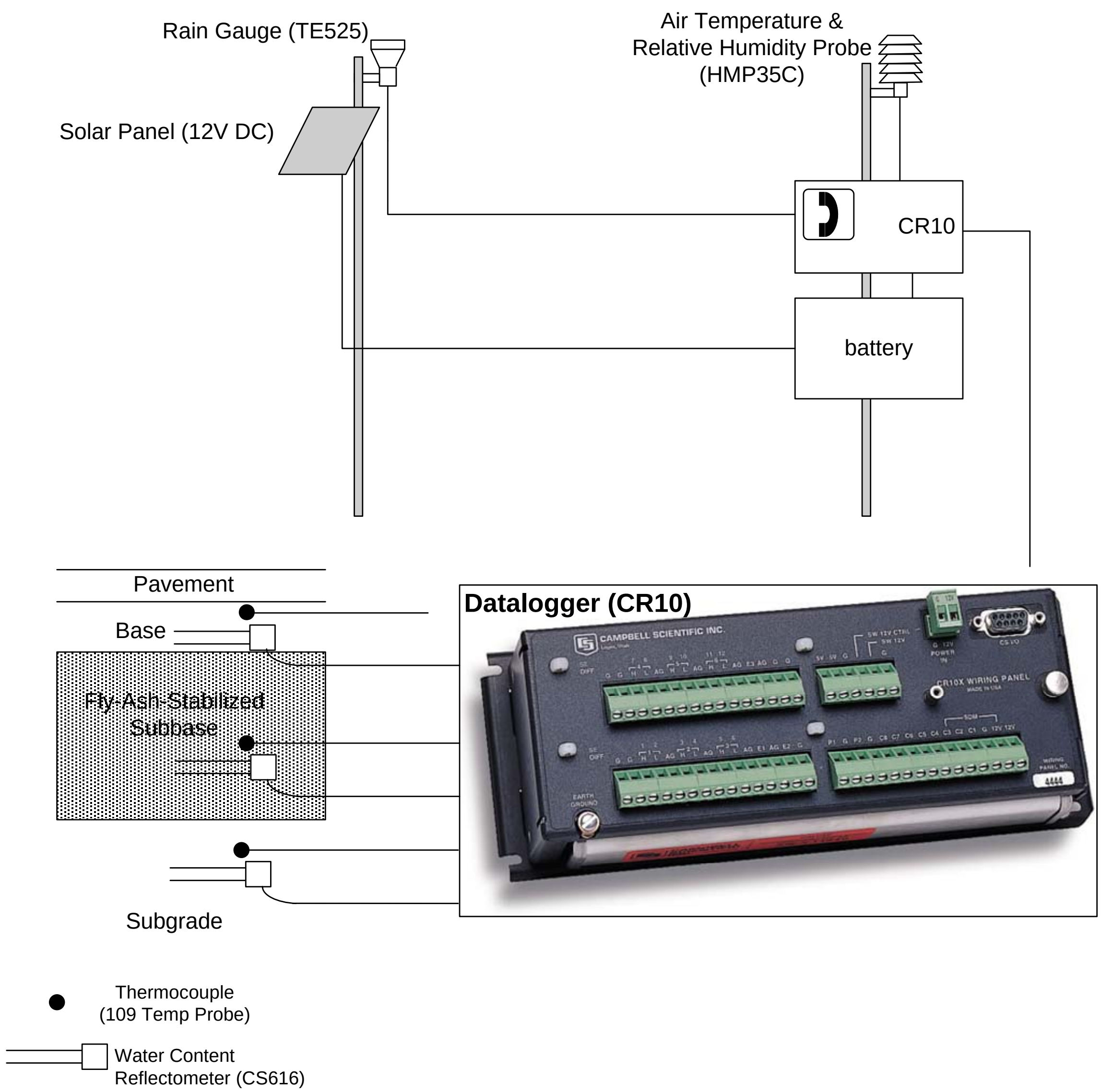


Fig. B4. Layout of field instrumentation.

APPENDIX C

LYSIMETER MONITORING DATA

Table C1. Summary of lysimeter data.

Date	East Side Fly Ash Section				West Side Fly Ash Section				Control Section			
	Drainage Rate (mm/d)	pH	Eh	EC	Drainage Rate (mm/d)	pH	Eh	EC	Drainage Rate (mm/d)	pH	Eh	EC
10/10/2005	0.33	6.92	69.80	8.86	0.00	-	-	-	0.00	-	-	-
11/10/2005	0.27	7.10	32.30	9.54	0.07	6.81	90.40	0.28	0.04	7.38	83.10	3.09
11/28/2005	0.46	7.19	-48.80	9.17	0.00	-	-	-	0.00	-	-	-
12/20/2005	0.28	7.02	225.6	9.5	0.07	7.13	234.4	4.94	0.38	7.05	240.9	4.46
1/19/2006	0.27	6.75	-90.1	8.61	0.04	6.87	321.4	5.16	0.28	6.98	310.1	4.09
2/7/2006	0.44	6.83	-	8.88	0.23	7.18	-	5.65	0.40	7.15	-	4.47
2/28/2006	0.15	7.21	117.3	7.7	0.22	7.25	120.0	5.62	0.23	7.53	160.6	2.5
4/2/2006	0.17	7.31	-92.6	6.74	0.09	7.45	142.6	4.32	0.16	7.19	-28.5	3.86

\- No Sample

APPENDIX D

RESULTS OF LONG-TERM MONITORING AT STH 60 WORKING PLATFORM (SUBBASE) TEST SECTIONS:

A. **CONCENTRATION OF ELEMENTS COLLECTED IN THE LYSIMETER UNDER THE FLY ASH STABILIZED SECTION AND THE CRUSHED AGREGATE CONTROL SECTION**

B. **BACKCALCULATED MODULUS OF FLY ASH AND CONTROL SECTION FROM FWD TESTS**

C. **PAVEMENT DISTRESS SURVEY DATA**

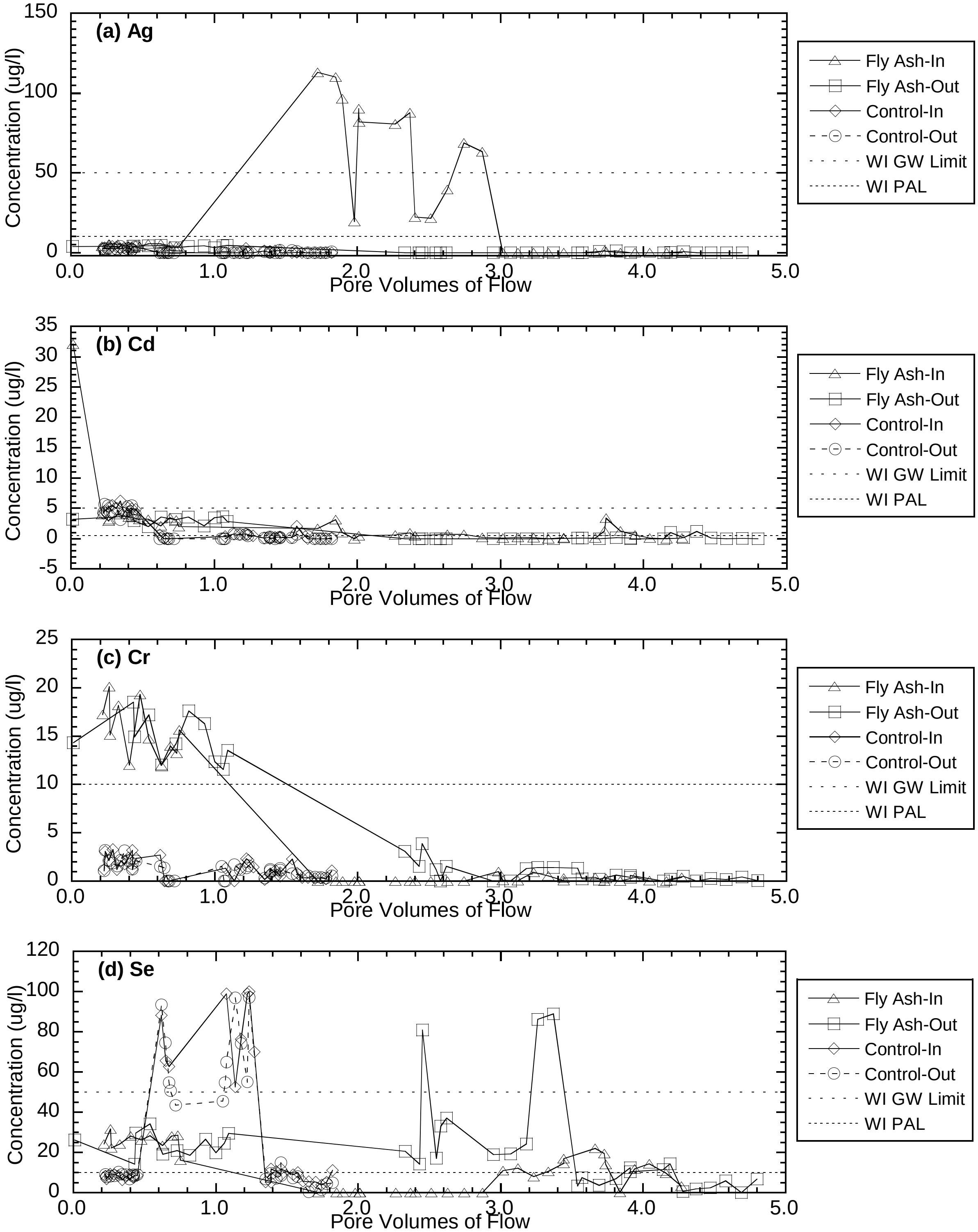


Fig. DA-1. Concentrations of four trace elements of concern in lysimeter leachate that were analyzed for during entire project; (a) Ag, (b) Cd, (c) Cr, and (d) Se.

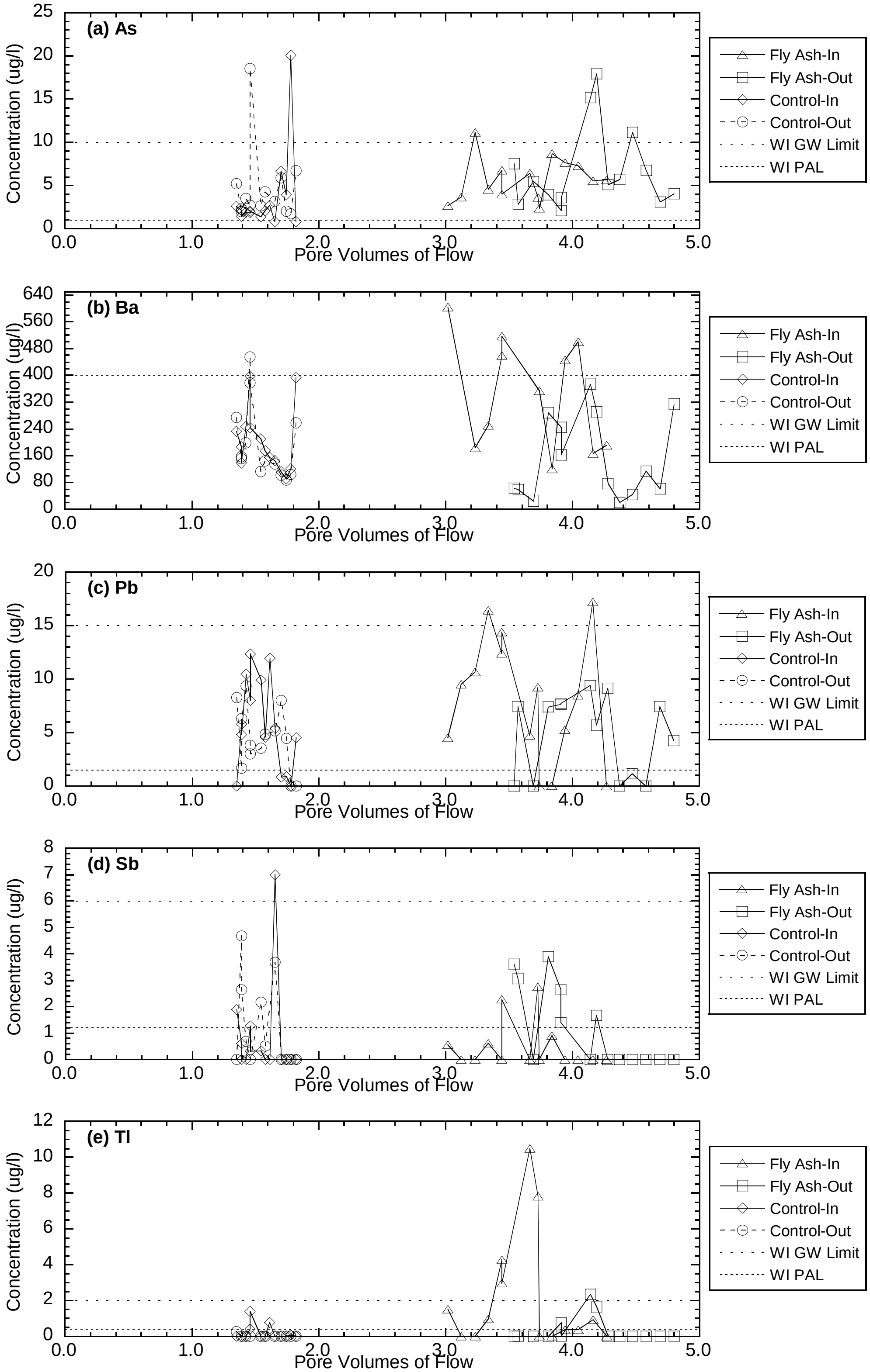


Fig. DA-2. Concentrations of other elements of concern in lysimeter leachate that were analyzed for beginning 5 years after installation; (a) As, (b) Ba, (c) Pb, (d) Sb, and (e) Tl.

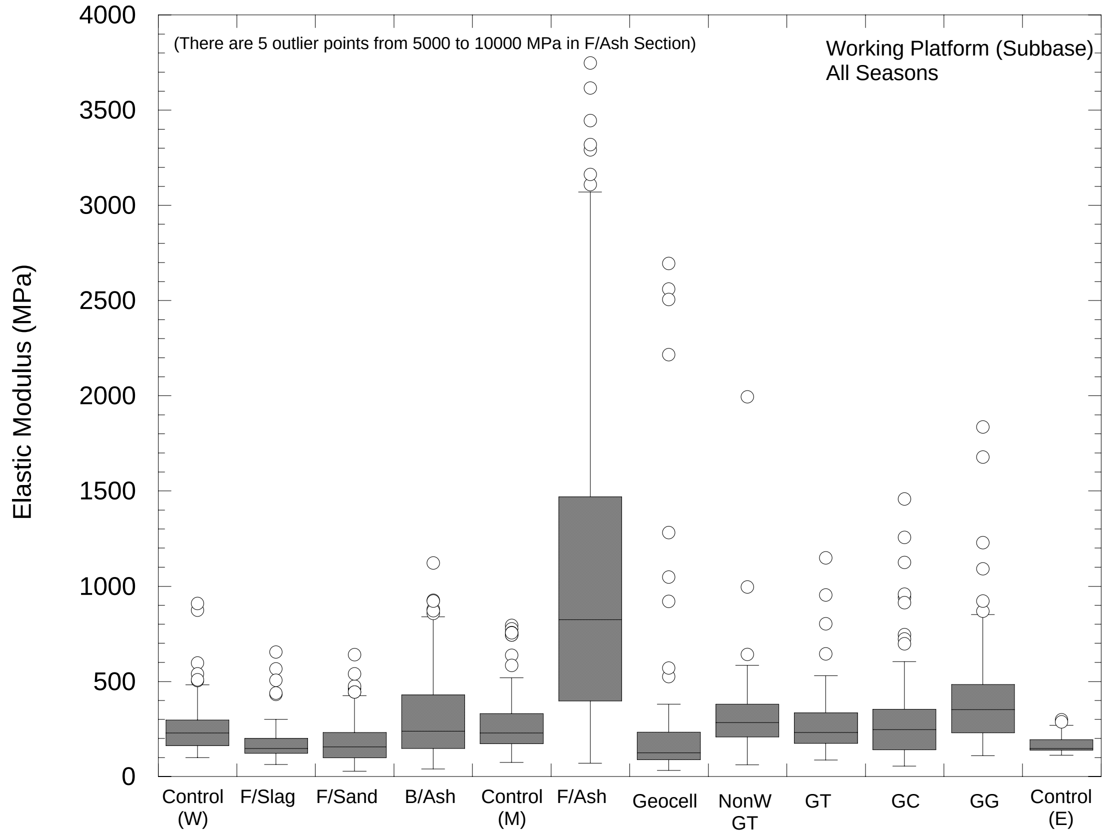


Fig. DB-1. STH 60 Test Working Platforms (Subbase) Moduli Averaged Over All Season 2000-2007

Control: Crushed Aggregate — B/Ash: Bottom Ash

F/Slag: Foundry Slag — F/Sand: Foundry Sand

F/Ash: Fly Ash — Geocell

NonW GT: Nonwoven Geotextile Reinforced Crushed Aggregate

GT: Woven Geotextile Reinforced Crushed Aggregate

GC: Geocomposite Drain under Crushed Aggregate

GG: Geogrid Reinforced Crushed Aggregate

Ref: T. B. Edil, C. H. Benson, M. S. Bin-Shafique, B. F. Tanyu, W. H. Kim, and A. Senol, "Field Evaluation of Construction Alternatives for Roadway over Soft Subgrade," *Transportation Research Record*, No. 1786, Paper No. 02-3808, National Research Council, Washington D. C., 2002, pp. 36-48

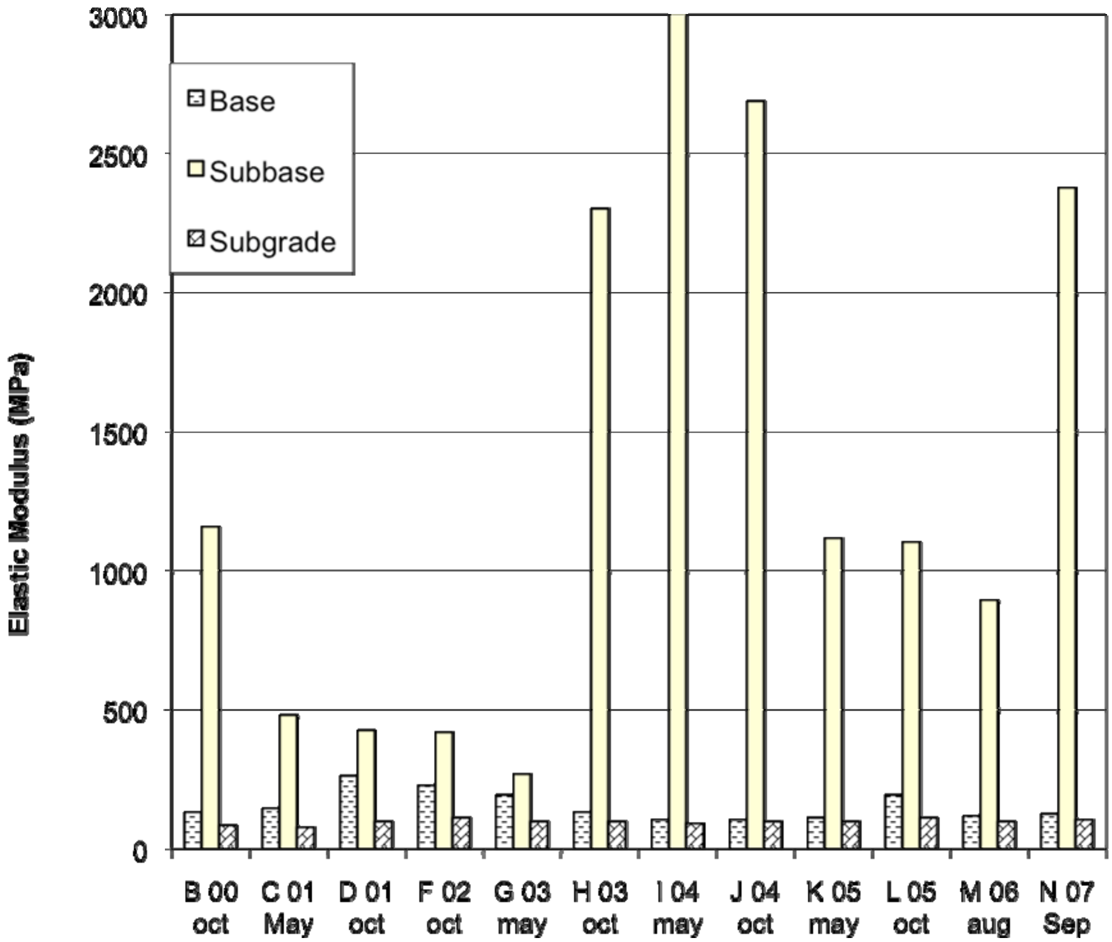


Fig. DB-2. STH 60 Fly Ash Working Platform (Subbase) Moduli 2000-2007

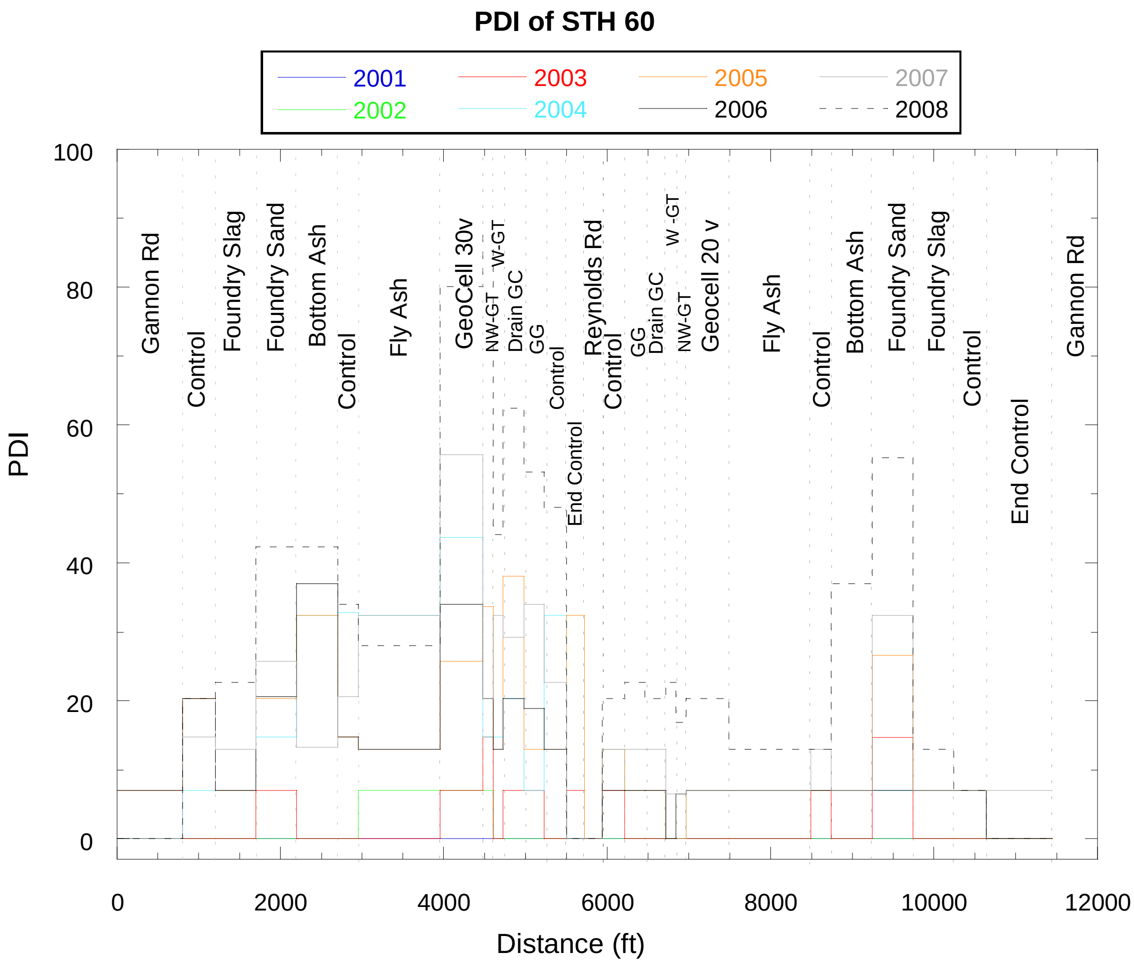


Fig. DC-1. STH 60 Pavement Distress Index (DPI) for 2001-2008

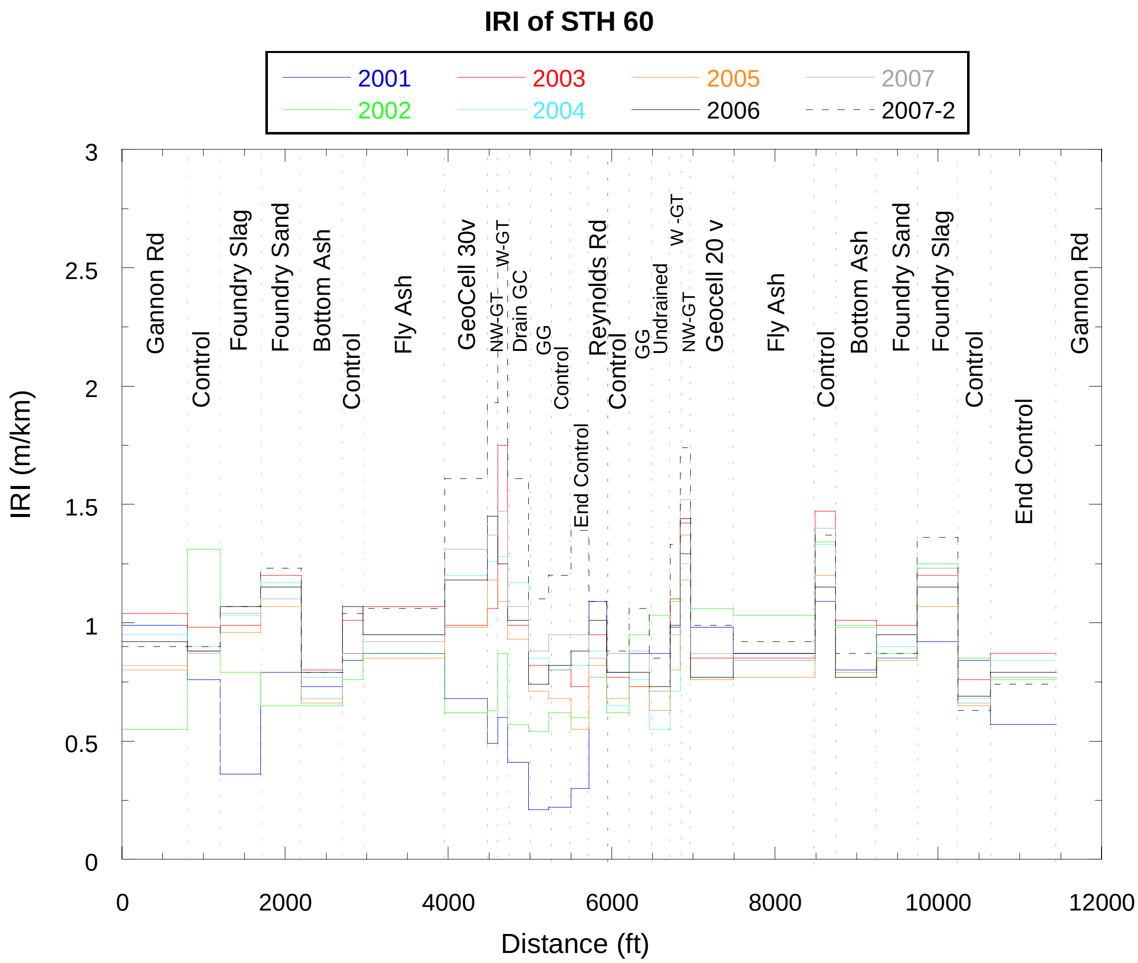


Fig. DC-2. STH 60 International Roughness Index (IRI) for 2001-2008

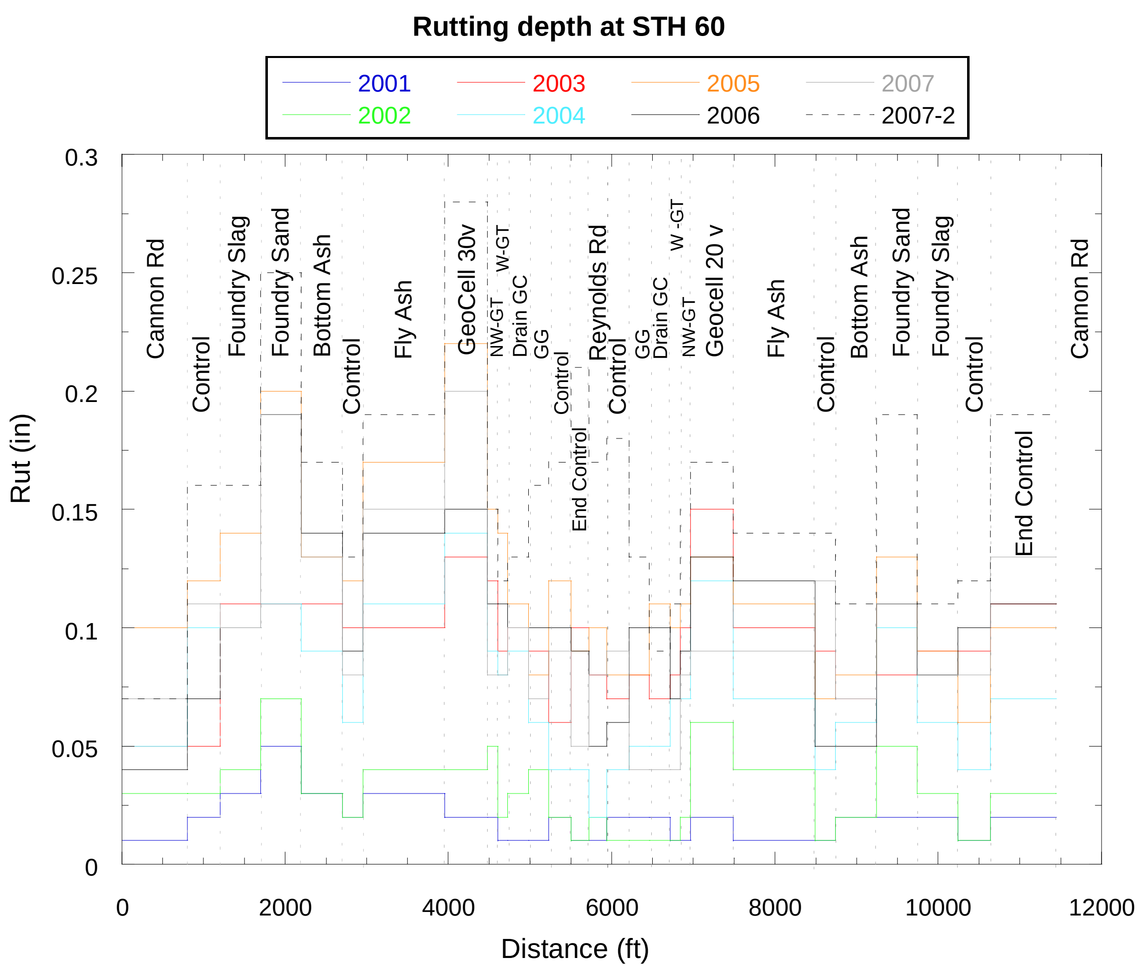


Fig. DC-3. STH 60 Rut Depth for 2001-2008

Recycled Sub-Base Study - Years 2001 through 2008

County: Columbia
Highway: STH 60
Project Number:
Data Collected For: Prof. Tuncer Edil - UW Madison

****Note: Section Lengths are measured in feet.**

						Data Collection Year							
Line	Road Name	RP From Feature	RP To Feature	Section Length	Pavement Type	**2001** PDI	**2002** PDI	**2003** PDI	**2004** PDI	**2005** PDI	**2006** PDI	**2007** PDI	**2008** PDI
1	STH 60E	Gannon Rd	Control	803.0	A	0.0	0.0	0.0	0.0	7.0	7.0	0.0	0.0
2	STH 60E	Control	Foundry Slag	400.0	A	0.0	0.0	0.0	7.0	20.3	20.3	14.8	20.3
3	STH 60E	Foundry Slag	Foundry Sand	500.0	A	0.0	0.0	0.0	7.0	7.0	7.0	13.0	22.7
4	STH 60E	Foundry Sand	Bottom Ash	500.0	A	0.0	0.0	7.0	14.8	20.3	20.6	25.7	42.3
5	STH 60E	Bottom Ash	Control	500.0	A	0.0	0.0	0.0	32.4	32.4	37.0	13.3	42.3
6	STH 60E	Control	Fly Ash	250.0	A	0.0	0.0	0.0	32.8	14.8	14.8	20.6	34.0
7	STH 60E	Fly Ash	Geocell - 30v	1000.0	A	0.0	7.0	0.0	32.4	13.0	13.0	32.4	28.0
8	STH 60E	Geocell - 30v	NW - GT	525.0	A	0.0	7.0	7.0	43.7	25.7	34.0	55.7	80.1
9	STH 60E	NW - GT	W - GT	125.0	A	0.0	7.0	14.8	14.8	33.7	20.3	20.3	87.5
10	STH 60E	W - GT	Drained	125.0	A	0.0	0.0	0.0	14.8	13.0	13.0	32.4	44.1
11	STH 60E	Drained	CG	250.0	A	0.0	0.0	7.0	20.3	38.1	20.3	29.2	62.5
12	STH 60E	CG	Control	250.0	A	0.0	0.0	7.0	7.0	13.0	18.9	34.0	53.2
13	STH 60E	Control	END Control	275.0	A	0.0	0.0	0.0	32.4	13.0	13.0	22.7	48.1
14	STH 60E	END Control	Reynolds Rd	219.0	A	0.0	0.0	7.0	0.0	32.4	0.0	0.0	0.0
15	STH 60W	Reynolds Rd	Control	219.0	A	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	STH 60W	Control	CG	275.0	A	0.0	0.0	7.0	13.0	13.0	7.0	13.0	20.3
17	STH 60W	CG	UnDrained	250.0	A	0.0	0.0	0.0	7.0	7.0	7.0	13.0	22.7
18	STH 60W	UnDrained	W - GT	250.0	A	0.0	0.0	0.0	7.0	7.0	7.0	13.0	20.3
19	STH 60W	W - GT	NW - GT	125.0	A	0.0	0.0	0.0	0.0	0.0	0.0	6.5	22.7
20	STH 60W	NW - GT	Geocell - 20v	125.0	A	0.0	0.0	0.0	0.0	0.0	6.5	6.5	16.9
21	STH 60W	Geocell - 20v	Fly Ash	525.0	A	0.0	0.0	0.0	7.0	7.0	7.0	7.0	20.3
22	STH 60W	Fly Ash	Control	1000.0	A	0.0	0.0	0.0	7.0	7.0	7.0	7.0	13.0
23	STH 60W	Control	Bottom Ash	250.0	A	0.0	0.0	7.0	7.0	7.0	7.0	13.0	13.0
24	STH 60W	Bottom Ash	Foundry Sand	500.0	A	0.0	0.0	0.0	7.0	7.0	7.0	7.0	37.0
25	STH 60W	Foundry Sand	Foundry Slag	500.0	A	0.0	0.0	14.7	7.0	26.6	7.0	32.4	55.3
26	STH 60W	Foundry Slag	Control	500.0	A	0.0	0.0	0.0	7.0	7.0	7.0	7.0	13.0
27	STH 60W	Control	END Control	400.0	A	0.0	0.0	0.0	7.0	7.0	7.0	7.0	7.0
28	STH 60W	END Control	Gannon Rd	803.0	A	0.0	0.0	0.0	0.0	0.0	0.0	7.0	0.0

SPRQ 145

Recycled Sub-Base Study - Years 2001 through 2008

County: Columbia
Highway: STH 60
Project Number:
Data Collected For: Prof. Tuncer Edil - UW Madison

***Note: IRI is measured in Meters per Kilometer.**
****Note: Section Lengths are measured in feet.**

						Data Collection Year							
	Road			Section	Pavement	**2001**	**2002**	**2003**	**2004**	**2005**	**2006**	**2007**	**2008**
Line	Name	RP From Feature	RP To Feature	Length	Type	IRI L m	IRI L m	IRI L m	IRI L m	IRI L m	IRI L m	IRI L m	IRI L m
1	STH 60E	Gannon Rd	Control	803.0	A	0.99	0.55	1.04	0.95	0.80	0.92	0.82	0.90
2	STH 60E	Control	Foundry Slag	400.0	A	0.76	1.31	0.98	0.90	0.88	0.88	0.87	0.90
3	STH 60E	Foundry Slag	Foundry Sand	500.0	A	0.36	0.79	0.99	1.03	0.96	1.07	1.04	1.07
4	STH 60E	Foundry Sand	Bottom Ash	500.0	A	0.79	0.65	1.20	1.17	1.07	1.15	1.10	1.23
5	STH 60E	Bottom Ash	Control	500.0	A	0.73	0.65	0.80	0.77	0.66	0.79	0.68	0.79
6	STH 60E	Control	Fly Ash	250.0	A	0.84	0.76	1.01	0.87	0.87	1.07	0.87	1.04
7	STH 60E	Fly Ash	Geocell - 30v	1000.0	A	0.87	0.87	1.07	0.95	0.85	0.95	0.92	1.06
8	STH 60E	Geocell - 30v	NW - GT	525.0	A	0.68	0.62	0.99	1.20	0.98	1.18	1.31	1.61
9	STH 60E	NW - GT	W - GT	125.0	A	0.49	0.63	1.06	1.26	1.18	1.45	1.37	1.93
10	STH 60E	W - GT	Drained	125.0	A	0.60	0.87	1.75	1.28	1.09	1.25	1.47	2.53
11	STH 60E	Drained	CG	250.0	A	0.41	0.57	0.99	1.17	0.93	1.01	1.07	1.61
12	STH 60E	CG	Control	250.0	A	0.21	0.54	0.82	0.85	0.71	0.74	0.88	1.10
13	STH 60E	Control	END Control	275.0	A	0.22	0.62	0.80	0.80	0.68	0.82	0.95	1.20
14	STH 60E	END Control	Reynolds Rd	219.0	A	0.30	0.60	0.73	0.82	0.55	0.88	0.95	1.39
15	STH 60W	Reynolds Rd	Control	219.0	A	1.09	0.77	0.95	0.88	0.82	1.01	0.85	1.09
16	STH 60W	Control	CG	275.0	A	0.79	0.62	0.77	0.65	0.68	0.79	0.87	0.88
17	STH 60W	CG	UnDrained	250.0	A	0.87	0.95	0.73	0.76	0.73	0.79	0.88	1.06
18	STH 60W	UnDrained	W - GT	250.0	A	0.87	1.03	0.71	0.55	0.63	0.73	0.71	0.85
19	STH 60W	W - GT	NW - GT	125.0	A	0.98	1.09	1.10	0.71	0.80	0.99	0.95	1.33
20	STH 60W	NW - GT	Geocell - 20v	125.0	A	1.29	1.18	1.42	1.25	1.37	1.44	1.52	1.74
21	STH 60W	Geocell - 20v	Fly Ash	525.0	A	0.98	1.06	0.85	0.76	0.76	0.77	0.87	0.99
22	STH 60W	Fly Ash	Control	1000.0	A	0.87	1.03	0.85	0.84	0.77	0.87	0.84	0.92
23	STH 60W	Control	Bottom Ash	250.0	A	1.09	1.34	1.47	1.33	1.20	1.15	1.40	1.37
24	STH 60W	Bottom Ash	Foundry Sand	500.0	A	0.80	0.99	1.01	0.79	0.79	0.77	0.98	0.87
25	STH 60W	Foundry Sand	Foundry Slag	500.0	A	0.85	0.87	0.99	0.90	0.84	0.95	0.85	0.87
26	STH 60W	Foundry Slag	Control	500.0	A	0.92	1.25	1.20	1.23	1.07	1.15	1.23	1.36
27	STH 60W	Control	END Control	400.0	A	0.84	0.85	0.76	0.68	0.65	0.69	0.66	0.63
28	STH 60W	END Control	Gannon Rd	803.0	A	0.57	0.76	0.87	0.84	0.77	0.79	0.77	0.74

Recycled Sub-Base Study - Years 2001 through 2008

County: Columbia
Highway: STH 60
Project Number:
Data Collected For: Prof. Tuncer Edil - UW Madison

***Note: Rutting is measured in Inches.**
****Note: Section Lengths are measured in feet.**

						Data Collection Year							
	Road			Section	Pavement	**2001**	**2002**	**2003**	**2004**	**2005**	**2006**	**2007**	**2008**
Line	Name	RP From Feature	RP To Feature	Length	Type	RUT Re	RUT Re	RUT Re	RUT Re	RUT Re	RUT Re	RUT Re	RUT Re
1	STH 60E	Gannon Rd	Control	803.0	A	0.01	0.03	0.05	0.05	0.10	0.04	0.07	0.07
2	STH 60E	Control	Foundry Slag	400.0	A	0.02	0.03	0.05	0.10	0.12	0.07	0.11	0.16
3	STH 60E	Foundry Slag	Foundry Sand	500.0	A	0.03	0.04	0.11	0.10	0.14	0.10	0.10	0.16
4	STH 60E	Foundry Sand	Bottom Ash	500.0	A	0.05	0.07	0.11	0.11	0.20	0.19	0.19	0.25
5	STH 60E	Bottom Ash	Control	500.0	A	0.03	0.03	0.11	0.09	0.13	0.14	0.13	0.17
6	STH 60E	Control	Fly Ash	250.0	A	0.02	0.02	0.10	0.06	0.12	0.09	0.08	0.13
7	STH 60E	Fly Ash	Geocell - 30v	1000.0	A	0.03	0.04	0.10	0.11	0.17	0.14	0.15	0.19
8	STH 60E	Geocell - 30v	NW - GT	525.0	A	0.02	0.04	0.13	0.14	0.22	0.15	0.20	0.28
9	STH 60E	NW - GT	W - GT	125.0	A	0.02	0.05	0.12	0.09	0.15	0.11	0.08	0.15
10	STH 60E	W - GT	Drained	125.0	A	0.01	0.02	0.09	0.08	0.14	0.11	0.08	0.12
11	STH 60E	Drained	CG	250.0	A	0.01	0.03	0.10	0.09	0.11	0.10	0.10	0.13
12	STH 60E	CG	Control	250.0	A	0.01	0.04	0.09	0.06	0.08	0.10	0.07	0.16
13	STH 60E	Control	END Control	275.0	A	0.02	0.02	0.06	0.04	0.12	0.10	0.09	0.17
14	STH 60E	END Control	Reynolds Rd	219.0	A	0.01	0.01	0.10	0.04	0.09	0.09	0.05	0.21
15	STH 60W	Reynolds Rd	Control	219.0	A	0.01	0.02	0.08	0.02	0.10	0.05	0.08	0.17
16	STH 60W	Control	CG	275.0	A	0.02	0.01	0.07	0.04	0.08	0.06	0.09	0.18
17	STH 60W	CG	UnDrained	250.0	A	0.02	0.01	0.08	0.05	0.08	0.10	0.04	0.13
18	STH 60W	UnDrained	W - GT	250.0	A	0.02	0.01	0.07	0.05	0.11	0.10	0.04	0.09
19	STH 60W	W - GT	NW - GT	125.0	A	0.01	0.01	0.08	0.07	0.10	0.07	0.04	0.11
20	STH 60W	NW - GT	Geocell - 20v	125.0	A	0.01	0.02	0.10	0.07	0.11	0.09	0.08	0.15
21	STH 60W	Geocell - 20v	Fly Ash	525.0	A	0.02	0.06	0.15	0.12	0.13	0.13	0.09	0.17
22	STH 60W	Fly Ash	Control	1000.0	A	0.01	0.04	0.10	0.07	0.11	0.12	0.09	0.14
23	STH 60W	Control	Bottom Ash	250.0	A	0.01	0.01	0.09	0.04	0.07	0.05	0.12	0.14
24	STH 60W	Bottom Ash	Foundry Sand	500.0	A	0.02	0.02	0.07	0.06	0.08	0.05	0.07	0.11
25	STH 60W	Foundry Sand	Foundry Slag	500.0	A	0.02	0.05	0.08	0.10	0.13	0.11	0.11	0.19
26	STH 60W	Foundry Slag	Control	500.0	A	0.02	0.03	0.09	0.06	0.09	0.08	0.08	0.11
27	STH 60W	Control	END Control	400.0	A	0.01	0.01	0.09	0.04	0.06	0.10	0.08	0.12
28	STH 60W	END Control	Gannon Rd	803.0	A	0.02	0.03	0.11	0.07	0.10	0.11	0.13	0.19

APPENDIX E

RESILIENT MODULUS OF FIELD-MIX FLY ASH STABILIZED SUBGRADE

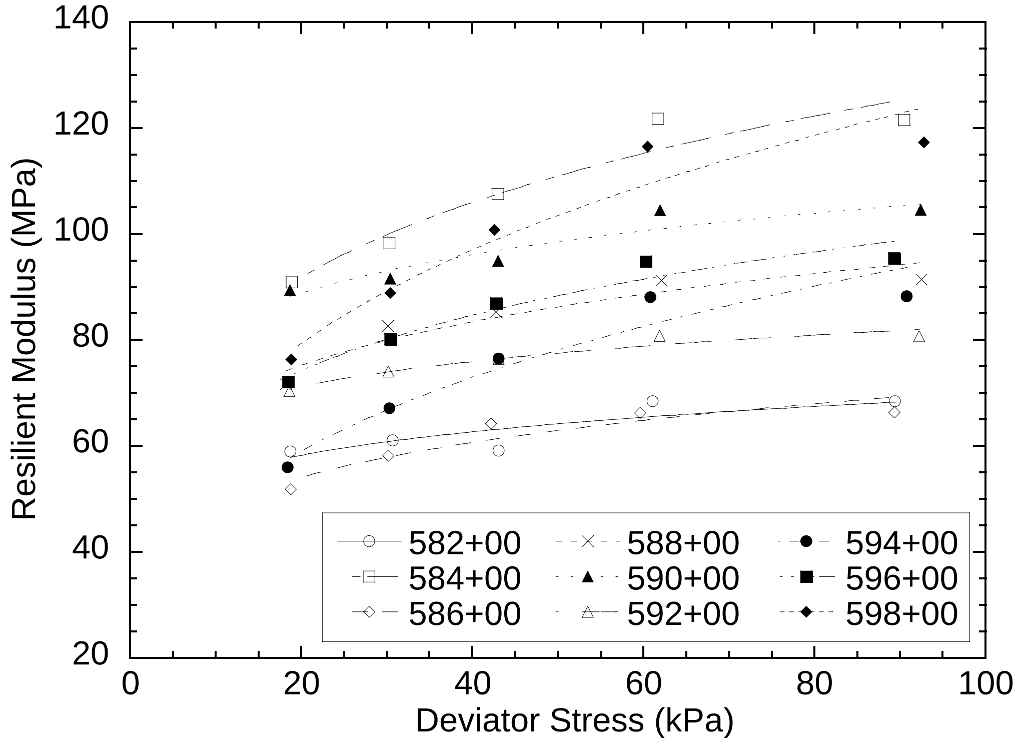


Fig. E-1. Specimens cured for 14 d. For stations 582+00 and 586+00, specimens had a 1:1 height-to-diameter ratio.

APPENDIX F

BACK ANALYSIS OF MODULUS USING ARTIFICIAL NEURAL NETWORK METHOD

One of the challenges in the MODULUS software back-calculation process is the need to define upper and lower limits for the modulus value for each layer. This iterative process can be very time-consuming. Bayrak and Ceylan (2008) proposed an alternative approach in back-calculating the modulus values for concrete pavement using the Artificial Neural Network (ANN). Bayrak and Celyan indicated that the ANN modulus back-calculation process is significantly faster than the MODULUS software approach. In analyzing US12 FWD data, the analysis was done using by varying 2 different variables:

1. The number of FWD sensors and deflections considered in the analysis:
   Two sets of deflections were considered; the first set use deflections from 4 sensors (0 inches, 12 inches, 24 inches, and 36 inches); the second set use deflections from 6 sensors (0 inches, 12 inches, 24 inches, 36 inches, 48 inches, and 60 inches).
2. The thickness of the base layer :
   In this analysis, there are 3 base layer thicknesses considered; 0 inches, 10 inches, and 22 inches.

Figure 1 is an example of ANN analysis results and Table 1 is the summary of ANN analysis. K is the Coefficient of Subgrade Reaction (in psi/inch), E is the PCC layer modulus (in ksi), and I is the radius of relative stiffness (in inches). The summary indicated the following:

1. The coefficient of sub-grade reaction (K) decreases from year to year indicating that there is an accumulation of damage in the base material) or in the sub-base material (fly-ash stabilized material)
2. The PCC surface layer modulus (E) is shown to increase from 2004 until 2006 then drop from 2006 to 2007.

The averaged value of deflection measured by the FWD continuously increase from the 2004 measurement until the 2007 measurement. This is an indication of damage being accumulated in the pavement structure. The ANN approach considers this increase in the deflection by reporting an increase in the PCC surface layer modulus (E) and as a decrease in the value of the coefficient sub-grade reaction (K). In the MODULUS analysis, the changes in deflection values are reflected in the change of base and sub-base modulus values.

Table 2 compares the average value of the ANN output and the MODULUS output. Like the ANN analysis, two different MODULUS analyses were conducted: 22 inches combined base, and a combination of 10 inches base and 12 inches sub-base.

1. Degradation of material
   Based on the output, the ANN analysis reported a slower degradation in the base layer material (24% in the first year) over time when compared with the MODULUS output (53% in the first year)
2. Consistency
   The ANN analysis consistently show that the material degradation over time. The MODULUS analysis does not show continuous degradation.

It is difficult to say whether ANN or MODULUS analysis is better considering they have different approaches. A comparison between back-calculated modulus with direct measurements in the field is required to check which approach is best.

Table F-1 Summary of ANN approach in back-calculating modulus

				2004	2005	2006	2007	Average
4-Model	D0-D12-D24-D36	None	K	467	356	342	217	318
			E	5677	6263	6860	6278	6409
			I	27	30	31	35	32
		10" Base	K	467	356	342	217	318
			E	5483	6054	6633	6066	6194
			I	27	30	31	35	32
		22" Base	K	467	356	342	217	318
			E	4316	4776	5257	4784	4983
			I	27	30	31	35	32
6-Model	D0-D12-D24-D36-D48-D60	None	K	390	311	299	193	277
			E	7259	7383	8176	7125	7565
			I	30	32	34	37	34
		10" Base	K	390	311	299	193	277
			E	7015	7132	7902	6883	7309
			I	30	32	34	37	34
		22" Base	K	390	311	299	193	277
			E	5535	5618	6234	5420	5762
			I	30	32	34	37	34

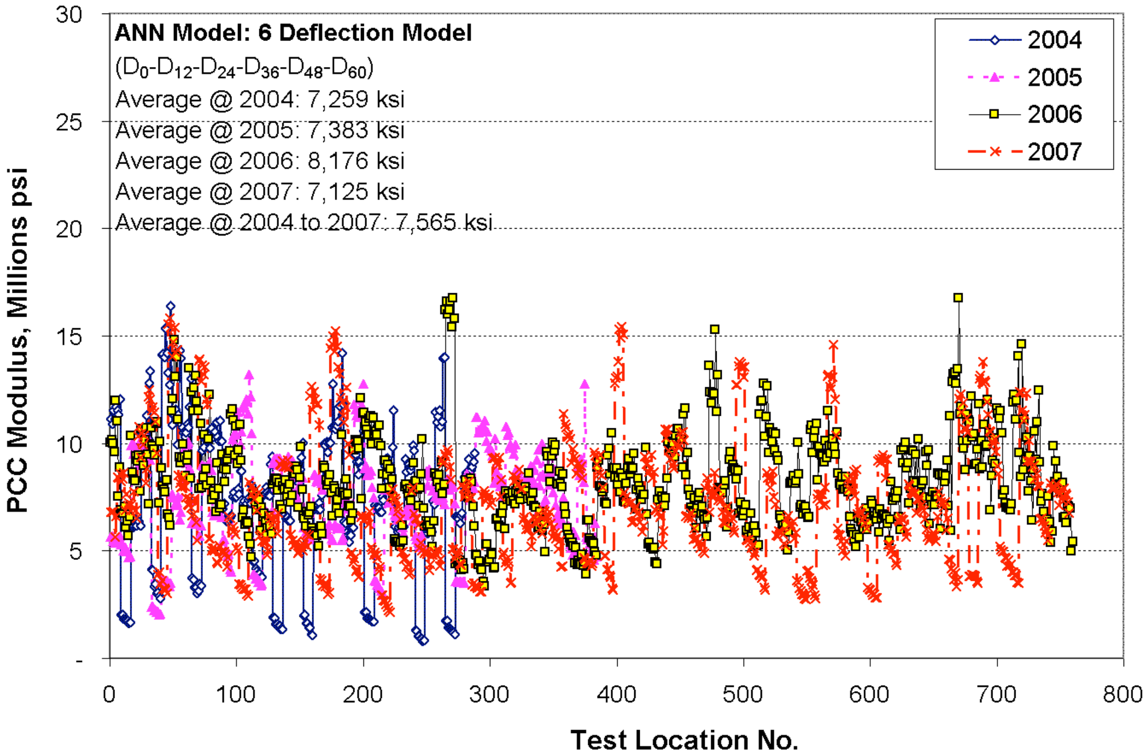


Fig. F-1 Example of ANN Analysis

TABLE F-2 Comparison between ANN and MODULUS output

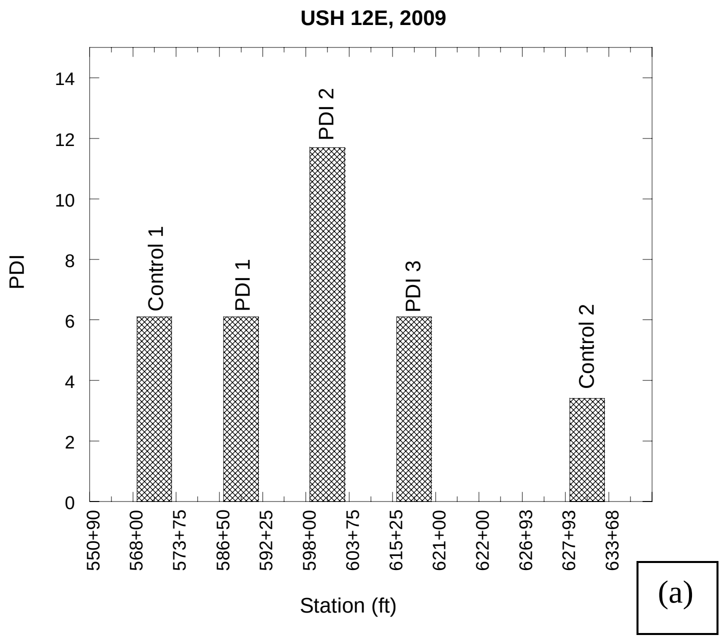
FWD Year	22-inch base model		10-inch base model		
	ANN K (ksi/i)	MOD-Base E (ksi)	ANN K (ksi/i)	MOD-Base E (ksi)	MOD-Subbase E (ksi)
2004	467.00	283.56	467.00	182.90	392.52
2005	356.00	133.50	356.00	86.17	166.39
2006	342.00	67.38	342.00	112.37	61.30
2007	217.00	69.84	217.00	96.72	111.06

References:

Bayrak, M.B., and H. Ceylan. (2008) “A Neural Network-Based Approach for the Analysis of Rigid Pavement Systems Using Deflection Data” Submitted to the 87th Transportation Research Board Annual Meeting, Washington, D.C

APPENDIX G

PAVEMENT SURVEY DATA AND GRAPHS FOR USH 12 – 2009



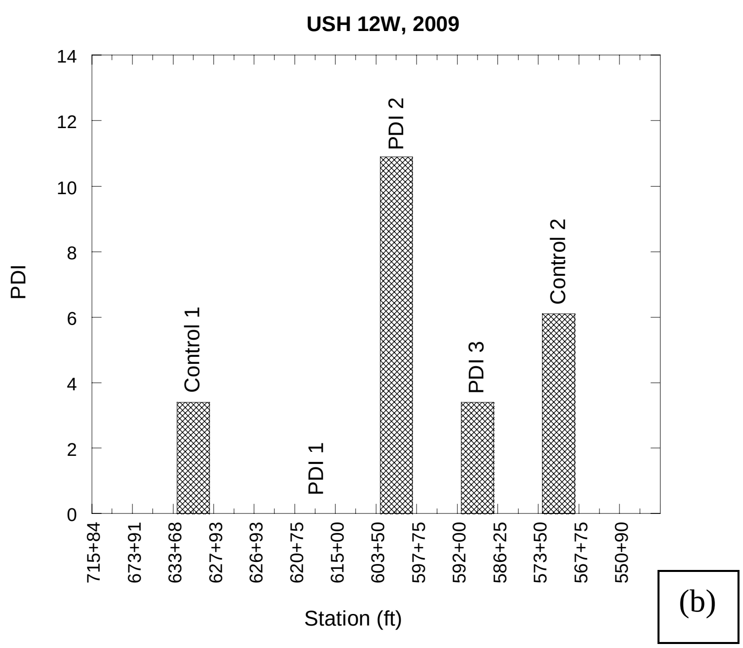
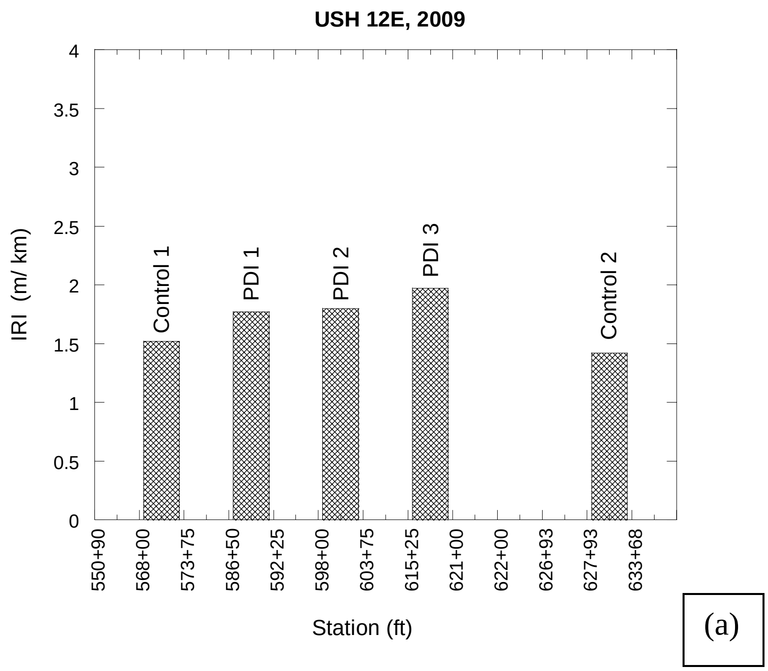


Fig. G1. Pavement Distress Index of Fly Ash (PD 1-3) and Control Sections in 2009 at USH 12: (a) East and (b) West Bound Lanes



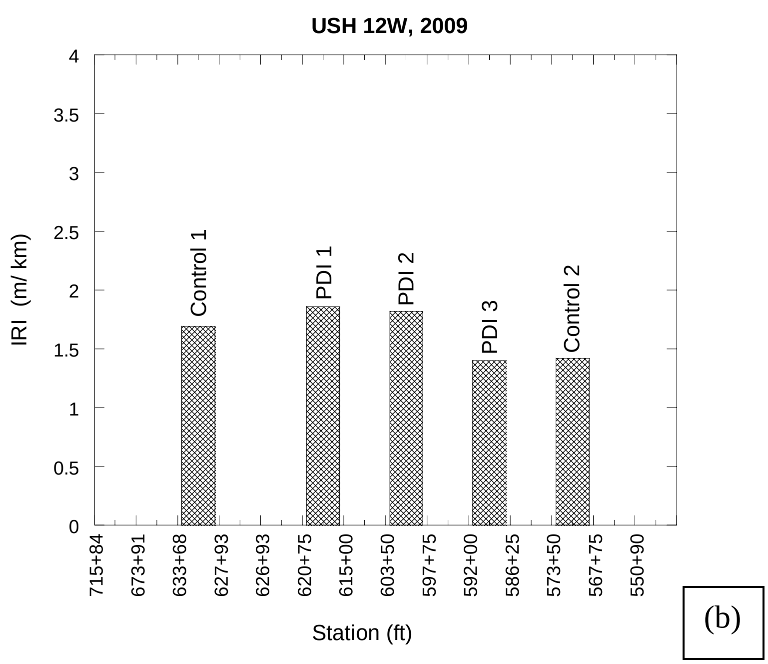
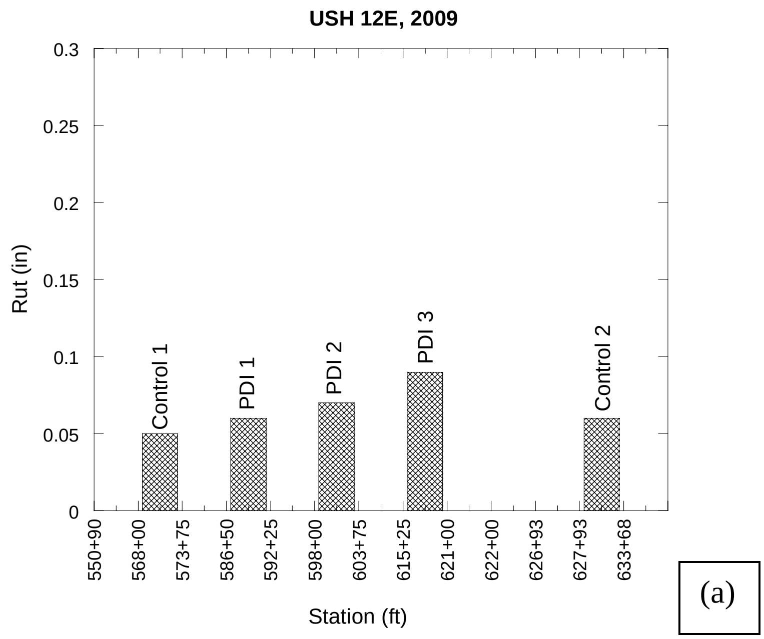


Fig. G2. International Roughness Index of Fly Ash (PD 1-3) and Control Sections in 2009 at USH 12: (a) East and (b) West Bound Lanes



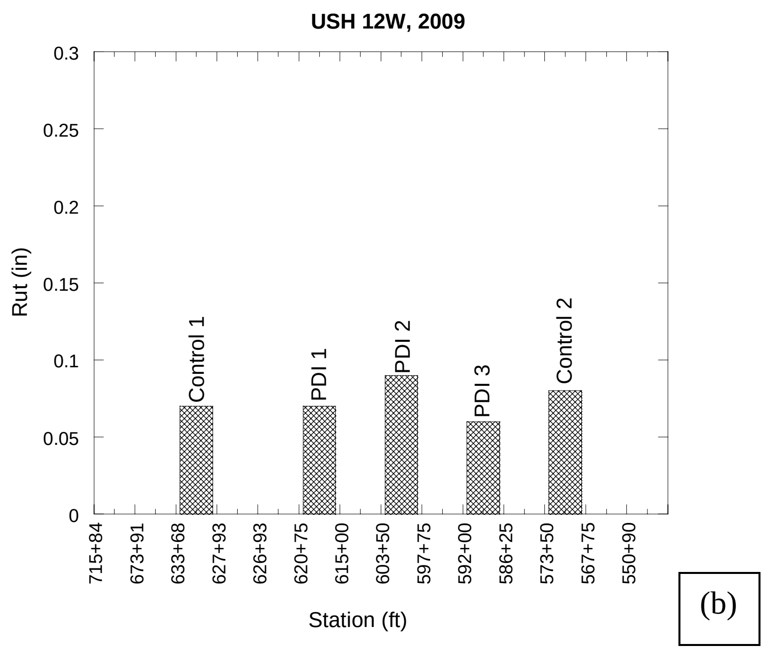


Fig. G3. Rut Depth of Fly Ash (PD 1-3) and Control Sections in 2009 at USH 12:
(a) East and (b) West Bound Lanes

PCC Fly Ash Study - 2009

County: Jefferson
Highway: USH 12
Data Collected For: Prof. Tuncer Edil - UW Madison

***Note: IRI is measured in meters per kilometer.**
****Note: Section Lengths are measured in feet.**
*****Note: Rutting is measured in decimal fractions of an inch**
******Note: WIsDOT pavement type 8 corresponds to a Jointed Plain Concrete Pavement with Dowels (JPCP)**

Line No.	Road Name	Section A or B	Station From	Station To	RP From Feature	RP To Feature	Section Length	IRI L m [m/km]	RUT Re [in]	PDI	Pavement Type	Surface Year
1	12 E		550+90	568+00	CTH G	Control 1	1710.0				8	2004
2	12 E		568+00	573+75	Control 1	End Control 1	575.0	1.52	0.05	6.1	8	2004
3	12 E		573+75	586+50	End Control 1	PDI 1	1275.0				8	2004
4	12 E	A	586+50	592+25	PDI 1	End PDI 1	575.0	1.77	0.06	6.1	8	2004
5	12 E		592+25	598+00	End PDI 1	PDI 2	575.0				8	2004
6	12 E	A	598+00	603+75	PDI 2	End PDI 2	575.0	1.80	0.07	11.7	8	2004
7	12 E		603+75	615+25	End PDI 2	PDI 3	1150.0				8	2004
8	12 E	B	615+25	621+00	PDI 3	End PDI 3	575.0	1.97	0.09	6.1	8	2004
9	12 E		621+00	622+00	End PDI 3	End Flyash	100.0				8	2004
10	12 E		622+00	626+93	End Flya	USh 12 Beg Div	493.0				8	2004
11	12 E		626+93	627+93	USH 12 Beg Div	Control 2	100.0				8	2004
12	12 E		627+93	633+68	Control 2	End Control 2	575.0	1.42	0.06	3.4	8	2004
13	12W		715+84	673+91	ROBERT S	BANKER R	4193.0				8	2004
14	12W		673+91	633+68	BANKER R	Control 1	4022.5				8	2004
15	12W		633+68	627+93	Control 1	End Control 1	575.0	1.69	0.07	3.4	8	2004
16	12W		627+93	626+93	End Control 1	USH 12W End Div	100.0				8	2004
17	12W		626+93	620+75	USH 12W End Div	PDI 1	618.0				8	2004
18	12W	B	620+75	615+00	PDI 1	End PDI 1	575.0	1.86	0.07	0.0	8	2004
19	12W		615+00	603+50	End PDI 1	PDI 2	1150.0				8	2004
20	12W	A	603+50	597+75	PDI 2	End PDI 2	575.0	1.82	0.09	10.9	8	2004
21	12W		597+75	592+00	End PDI 2	PDI 3	575.0				8	2004
22	12W	A	592+00	586+25	PDI 3	End PDI 3	575.0	1.40	0.06	3.4	8	2004
23	12W		586+25	573+50	End PDI 3	Control 2	1275.0				8	2004
24	12W		573+50	567+75	Control 2	End Control 2	575.0	1.42	0.08	6.1	8	2004
25	12W		567+75	550+90	End Control 2	CTH G	1685.0				8	2004

APPENDIX H

CASE STUDY OF SUBGRADE STABILIZATION USING FLY ASH: STATE HIGHWAY 32, PORT WASHINGTON, WISCONSIN

CASE STUDY OF SUBGRADE STABILIZATION USING FLY ASH: STATE HIGHWAY 32, PORT WASHINGTON, WISCONSIN

By Bert D. Trzebiatowski[1], Tuncer B. Edil[2], and Craig H. Benson[2]
Geo Institute Members

Abstract: This paper describes a case history where Class C fly ash, an industrial byproduct of electric power production, was used to stabilize a sandy clay highway subgrade so that a firm working platform could be provided for pavement construction. California bearing ratio (CBR), resilient modulus (M_r), and unconfined compressive strength (UC) tests were conducted on the soil alone and the soil-fly ash mixture to assess how fly ash improves the bearing resistance, stiffness, and shear strength of the soil. Field tests were also conducted during construction using a soil stiffness gauge (SSG) and a falling weight deflectometer (FWD) to assess the stiffness and modulus of the stabilized soil in situ. CBRs ranging between 46 and 150 were obtained for the stabilized soil after 7 d of curing, whereas the soil typically has CBR near 0 in its naturally moist condition (~5% wet of optimum water content). M_r of the stabilized soil ranged between 11 and 28 MPa after 7 d of curing and between 17 and 68 MPa after 28 d of curing, whereas the un-stabilized clay was too soft to determine its M_r using conventional methods. SSG tests indicated that the in situ stiffness of the stabilized soil ranged between 19 and 31 MN/m after 7 d of curing, whereas the unstabilized soil had a stiffness between 8 and 21 MN/m. Moduli computed from the stiffness measurements made with the SSG were in good agreement with those determined from the FWD data, but both were about 10x higher than those from the M_r tests. Unconfined compressive strengths (q_u) of the stabilized soil ranged between 276 and 607 KPa after 7 d of curing and between 304 and 683 KPa after 28-d of curing, whereas q_u of the un-stabilized soil was less then 200 kPa.

INTRODUCTION

Soft subgrade is a problematic issue associated with constructing highways in northern tier states in the US. For example, in Wisconsin, 60% of the existing subgrade soils are considered 'poor' in terms of their ability to support construction

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traffic as well as the long-term repetitive loading associated with the service life of a pavement (Edil et al. 2002). In most cases, approximately 1-2 m of the soft subgrade is removed and the excavated region is backfilled with crushed rock to form a sturdy working platform for pavement construction. This procedure, while commonplace in Wisconsin and other states, is costly, time consuming, and has environmental impacts (i.e., destruction of land associated with quarrying rock and displacement of large quantities of fine-grained soil). Consequently, the Wisconsin Department of Transportation (WisDOT) is seeking alternative methods for dealing with soft subgrades. One of the methods under consideration is stabilizing the soft soil in situ using cementitious fly ash derived from electric power generating plants.

This paper describes a case history where a sandy clay subgrade was stabilized using class C fly ash to create a working platform for construction of a Portland cement concrete pavement. Prior to stabilization, samples of the subgrade were collected and tested to determine their index properties and how addition of fly ash would affect the California bearing ratio (CBR), resilient modulus (M_r), and unconfined compressive strength (q_u) of the soil. Tests were also conducted on samples of the in situ stabilized soils to determine if similar improvements in properties were obtained during construction. A soil stiffness gauge (SSG) was also used to measure the stiffness near the surface of the stabilized subgrade and falling weight deflectometer tests (FWD) were conducted to evaluate the overall improvement in stiffness achieved through stabilization.

BACKGROUND

Site

Stabilization was conducted in the two southbound lanes of Wisconsin State Trunk Highway 32 (STH-32) in Port Washington, Wisconsin. The stabilized subgrade extended from East Sauk Road to Interstate Highway 43 (I-43), a length of approximately 3.7 km. Five stations (611+50, 612+50, 613+50, 614+50, and 615+50) in a 150-m segment of STH-32 were evaluated in this study. All measurements were made and all samples were collected along the centerline between the two lanes in the study area.

Soil and Fly Ash

Five grab samples (≈ 20 kg each) of the subgrade soil were collected from a depth between 0-0.5 m at the aforementioned stations. Tests were conducted on these samples to determine index properties, compaction characteristics, CBR, M_r, and q_u. A summary of the index properties is in Table 1. Particle size distribution curves for the soils are shown in Fig. 1. All five soils are broadly graded. Gravel content generally is the factor that differentiates the soils.

Table 1. Index properties of subgrade soils.

Sample	LL	PI	% Fines	% Clay (<2μm)	Classification		In Situ Water Content (%)	w_{opt} (%)	$\gamma_{d,max}$ (kN/m^3)	
					USCS	AASHTO			Un-corrected	Gravel Corrected
615+50	23	10	53	46	CL	A-4	10.0	9.8	20.6	20.9
614+50	23	9	46	35	SC	A-4	8.1	11.0	20.0	20.9
613+50	23	9	43	33	SC	A-4	9.9	10.6	20.4	21.5
612+50	34	18	66	50	CL	A-6	11.5	13.6	18.7	19.1
611+50	28	14	35	27	GC	A-2-6	7.6	9.8	21.3	23.5

Notes: LL = Liquid Limit, PI = Plasticity Index, USCS = Unified Soil Classification System, AASHTO = Association of American State Highway and Transportation Officials, w_{opt} = optimum water content per ASTM D 698, $\gamma_{d,max}$ = maximum dry unit weight per ASTM D 698.

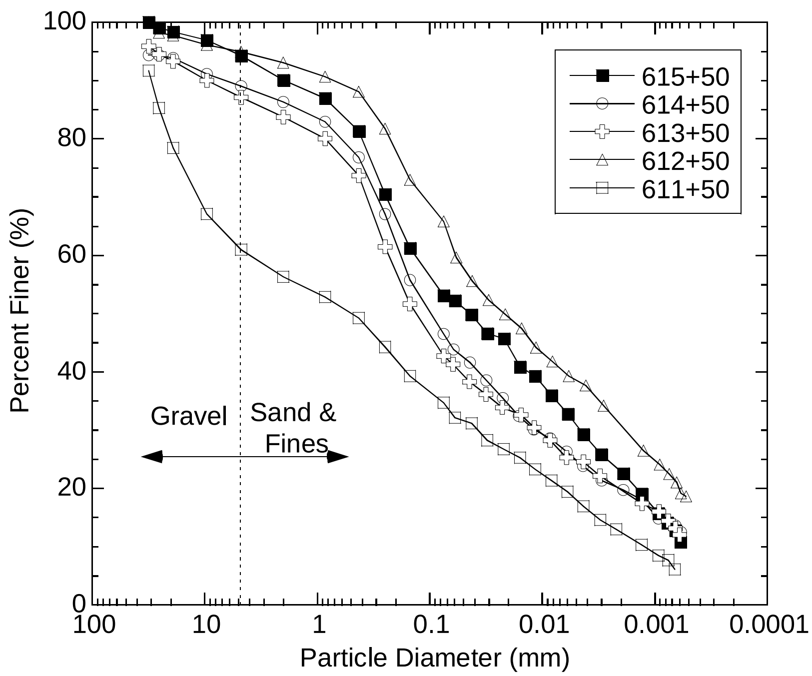

Fig. 1. Particle-size distribution of subgrade soils.

The soils classify as sandy lean clay (612+50 and 615+50), clayey sand (614+50 and 613+50), or clayey gravel with sand (611+50) according to ASTM D 2487. All five soils have similar Atterberg limits and are of low plasticity, although the soil from 612+50 is slightly more plastic than those from the others four stations. The average in situ water content of the soils at the time of construction was 9.4% ± 1.6%, which is unusually low for this site. Finer-grained subgrades in Wisconsin typically have water contents approximately 5-7% wet of optimum water content (Tanyu et al.

2003). The water contents were much lower than anticipated due to an unusual two-month drought at the time of construction.

Class C fly ash from the Pleasant Prairie (PP) Power Station in Pleasant Prairie, WI was used for stabilization. Chemical and physical analyses of the fly ash were conducted by LaFarge North America in Lockport, Illinois. Chemical composition of the fly ash is summarized in Table 2 along with the composition of typical class C and F fly ashes and typical Portland cement. The composition of the PP fly ash is very similar to the 'typical' class C ash.

Table 2. Chemical composition of PP fly ash along with typical compositions for class C and F ashes and Portland cement.

Chemical Species	**Percent of Composition**			
	PP Fly Ash	**Typical* Class C Fly Ash**	**Typical* Class F Fly Ash**	**Typical* Portland Cement**
CaO (lime)	21	24	9	64
SiO_2 (Silicon Dioxide)	38	40	55	23
Al_2O_3 (Aluminum Oxide)	21	17	26	4
Fe_2O_3 (Iron Oxide)	5	6	7	2
MgO (Magnesium Oxide)	4	5	2	2
SO_3 (Sulfur Trioxide)	2	3	1	2
SO_3	2	< 5	< 5	-
$SiO_2 + Al_2O_3 + Fe_2O_3$ (%)	64	> 50	> 70	-
Loss on Ignition	0.60	< 6	< 6	-

*Bin-Shafique et al. (2002).

Mix Design

WisDOT and Midwest Engineering Services, Inc., of Waukesha, WI evaluated mix-designs prior to construction to identify a suitable fly ash and the necessary fly ash content. Compaction and CBR tests were conducted using a range of fly ash contents. Results of the CBR tests indicated a 10% fly ash mixture would provide a sturdy working platform for construction equipment. Based on these test results, WisDOT selected a design consisting of a 0.3-m-thick stabilized layer with a fly ash content of 10% and a water content (based on fly ash and soil solids) of 12-14%, which is 1% wet of optimum water content (standard Proctor effort) for the soil-fly ash mixture.

Construction

Fly ash was spread onto the subgrade in a 0.1-m thick layer using a lay-down truck that was specially designed for fly ash application with minimal dust generation (Bin-Shafique et al. 2002). The fly ash was placed in strips that were 200 m long and 2.5 m wide until the width of the section was covered. A Wirtgen WR 2500 road reclaimer was used to mix the fly ash into the subgrade to a depth of 0.3 m. This reclaimer includes a water boom in front of the mixing tines that is used to add water to the mixture as needed to achieve the desired water content (12-14% in this case). Following mixing, the mixture was compacted using 4-6 passes of a self-propelled tamping foot compactor. After compaction, the surface was smoothed with a motor grader and compacted again using a vibratory compactor with a steel drum. Compaction was continued until the dry unit weight of the mixture exceeded 18.8 kN/m^3. Compaction was completed within 1 to 2 h after mixing. A nuclear density gage was used to monitor the water content and dry unit weight during compaction.

Construction of the overlying pavement layers began 1 d after the subgrade was stabilized. The overlying layers consisted of (bottom to top) 0.15 m of WisDOT Grade 2 crushed aggregate, 0.05 m of recycled asphalt pavement (RAP), and 0.23 m of Portland cement concrete.

EVALUATION METHODS

Laboratory tests were conducted to determine the CBR, q_u, and M_r of the mixture prepared in the field and mixtures prepared in the laboratory using soil and fly ash collected in the field during construction. Non-destructive field tests were conducted to determine the stiffness and modulus of the subgrade. The field tests were conducted directly on the subgrade using a soil stiffness gage (SSG) and on the completed pavement structure using a falling weight deflectometer (FWD)

Two methods were attempted to collect samples of the in situ mixture for laboratory testing. One method consisted of collecting a grab sample of the mixture immediately after mixing. This mixture was compacted in a mold to the same dry unit weight achieved in the field at the sampling location, as measured by the nuclear density gage. These samples were compacted in the field immediately after compaction occurred in the field, which typically was about 2 hr after mixing. The other method, which was unsuccessful, consisted of coring the stabilized subgrade after 7 d with a coring tool used to collect samples of concrete. The stabilized soil was greatly disturbed during the coring process, and the samples that were retrieved by coring were too disturbed to warrant testing.

Laboratory Methods

Specimens were tested following the methods described in ASTM D 3668 (CBR), AASHTO T 292 (M_r), and ASTM D 5102 (q_u). CBR specimens were prepared in a mold with a height of 114 mm and a diameter of 152 mm. The CBR specimens were not soaked prior to testing and were tested at a strain rate of 1.3 mm/min. M_r specimens were prepared in a mold that was 102 mm in diameter and 203 mm high. A confining pressure of 21 kPa and a seating load of 13.8 kPa were used for the M_r tests, and the testing sequence for cohesive soils was followed. Tests to determine q_u were conducted on the same specimens used for the M_r tests immediately after the M_r testing was conducted (the relatively small strains in the M_r test were assumed to have little effect on the peak undrained strength of the mixtures). A strain rate of 0.21%/min was used for the unconfined compression tests per ASTM D 5102. Standard method D 5102 is for compacted soil-lime mixtures rather than soil-fly ash mixtures. No standard method currently exists for unconfined compression testing of soil-fly ash mixtures.

Laboratory-mixed specimens of soil and fly ash retrieved during construction were prepared 5% wet of the optimum water content with 10% fly ash content. These tests were conducted to assess what may have occurred if the subgrade had been as moist as anticipated during design (i.e., if the drought had not occurred). The specimens were compacted 2 h after mixing to simulate the delay between stabilization and compaction in the field. Specimens of soil alone were also prepared 5% wet of optimum water content using the same procedure to simulate the wet and soft subgrade conditions that typically exist prior to stabilization. Soil specimens were also compacted at optimum water content for comparative purposes. The soil and fly ash used to prepare the specimens were used as is (i.e., no sieving or processing of either material was conducted prior to preparing the specimen) so that a mixture representative of field conditions would be tested. All specimens were sealed in plastic and cured for 7 to 28 d at 25°C and 100% relative humidity prior to testing.

Soil Stiffness Gauge

Stiffness of the subgrade was measured in situ using a Humboldt SSG using procedures described in Sawangsuriya (2001). Three measurements of the subgrade stiffness were made at each station within a 0.6-m radius. These measurements deviated by less than 10%. The average of the three stiffness measurements at each location is reported herein.

Falling Weight Deflectometer

FWD tests were conducted at the five stations using a KUAB model 2m-33 FWD. Deflections measured with the FWD were used to estimate the stiffness of the pavement layers, including the stabilized subgrade. The analysis was conducted using the program EVERCALC 5.0 (Washington State Department of Transportation), which is available at www.wsdot.wa.gov. A multilayer linear elastic pavement system is assumed when using EVERCALC to determine moduli from FWD data.

RESULTS OF EVALUATION

Tests were conducted on three soils and soil-fly ash mixtures. One of the three soils was a composite of the samples collected from stations 613+50, 614+50, and 615+50. These samples were combined because the soils had similar particle size distribution and Atterberg limits (Table 1, Fig. 1) The other soils are from stations 612+50 and 611+50, and represent higher (611+50) and lower (612+50) gravel contents. The composite soil (613-615+50) represents an intermediate gravel content.

California Bearing Ratio

CBRs of the unstabilized and stabilized soils are summarized in Table 3. CBRs of the unstabilized soil are very low (<3), particularly for the soils compacted 5% wet of optimum water content (CBR = 0-0.4). At both water contents, the unstabilized soils would be considered as poor and unacceptable subgrades prone to extensive rutting by construction traffic (Acosta et al. 2003). Larger CBRs were obtained for the stabilized soils, particularly for the field mixtures. The 7-d laboratory mixtures, prepared 5% wet of optimum water content, have CBRs ranging between 7-9, whereas the CBRs of the field mixtures, mixed at approximately optimum water content, range between 46-150 (more than 10x higher than the CBR of soil alone compacted at optimum water content). The high CBR of the field mixtures indicates that these stabilized soils should be excellent subgrades (Acosta et al. 2003). However, had the water content been much higher at the time of construction (i.e., had the drought not occurred), a softer material would likely have been obtained. However, even if the water content had been elevated, addition of fly ash would have resulted in an appreciable increase in the bearing strength of the subgrade.

Table 3. CBR of unstabilized and stabilized soils prepared in the laboratory and field.

Station	California Bearing Ratio (CBR)				
	Unstabilized		Stabilized		
	5% Wet of Optimum	Optimum	7-d Lab. Mixture (5% Wet of Optimum)	7-d Field Mixture (≈ Optimum)	28-d Field Mixture (≈ Optimum)
615+50	0.0	2	9	57	58
614+50				124	-
613+50				46	63
612+50	0.4	3	9	46	-
611+50	0.2	2	7	150	-
Average	0.2	2	8	85	60
Std. Dev.	0.2	1	1	49	3

Notes: Hyphen indicates no specimen was tested. Optimum water contents are in Table 1.

Comparison of the CBRs corresponding to 7 d and 28 d of curing indicates that most of the effect on bearing strength is realized within 7 d. No consistent increase in CBR is evident when comparing the 7-d and 28-d CBRs and, in some cases, the 28-d CBR is lower than the 7-d CBR (this effect is believed to be due to specimen variability rather than a true decrease in CBR). Acosta et al. (2003) also report that the CBR of soil-fly ash mixtures changes little between 7 and 28-d of curing.

Resilient Modulus

Results of the M_r tests are summarized in Table 4. Data are only provided for unstabilized soils compacted at optimum water content. Unstabilized soils prepared 5% wet of optimum water content failed during the conditioning phase of the M_r test. The unstabilized specimen prepared with the soil from 611+50 at optimum water content also failed during the conditioning phase. In fact, the specimen prepared with soil from 611+50 began to fail after the seating load was applied, indicating that this soil is very weak, even at optimum water content.

Typical curves relating resilient modulus to deviator stress are shown in Fig. 2 (laboratory prepared specimens of unstabilized and stabilized soil) and Fig. 3 (field mixtures). Some of the curves contain fewer data points than is common for resilient modulus tests because the specimens failed at higher stresses, precluding completion of the M_r test. For both the stabilized and unstabilized soils, the resilient modulus is modestly sensitive to deviator stress, with the stabilized soils exhibiting less sensitivity to deviator stress due to the cementing provided by the fly ash.

Table 4. Resilient modulus of soil alone and soil-fly ash mixtures prepared in the laboratory or collected from the field. Resilient moduli correspond to a deviator stress of 20 kPa.

<table>
<tr><th rowspan="3">Station</th><th colspan="4">Resilient Modulus (MPa) (Deviator Stress of 20 kPa)</th></tr>
<tr><th>Unstabilized</th><th colspan="3">Stabilized</th></tr>
<tr><th>Optimum</th><th>7-Day Lab Mixture (5% Wet of Optimum)</th><th>7-Day Field Mixture (≈ Optimum)</th><th>28-day Field Mixture (≈ Optimum)</th></tr>
<tr><td>615+50</td><td rowspan="3">15.3</td><td rowspan="3">10.0*</td><td>11.3</td><td>16.7</td></tr>
<tr><td>614+50</td><td>28.1</td><td>40.9</td></tr>
<tr><td>613+50</td><td>21.8</td><td>57.0</td></tr>
<tr><td>612+50</td><td>9.59</td><td>19.4</td><td>21.8</td><td>67.5</td></tr>
<tr><td>611+50</td><td>F</td><td>10.8</td><td>22.1</td><td>-</td></tr>
<tr><td>Average</td><td>12.4</td><td>13.4</td><td>21.0</td><td>45.5</td></tr>
<tr><td>Std. Dev.</td><td>4.02</td><td>5.23</td><td>6.05</td><td>22.1</td></tr>
</table>

Notes: F = specimen failed during conditioning sequence. Hyphen indicates no test conducted. *Extrapolated from last three points. See Table 1 for optimum water contents for each station.

The M_r of the stabilized specimens mixed in the laboratory using a water content 5% wet of optimum water content is nearly the same, on average, as the M_r of the unstabilized specimens prepared at optimum water content (Table 4). This suggests that fly ash stabilization can result in comparable improvement in stiffness as moisture-conditioning (i.e., drying) and compaction. The M_r of the stabilized field specimens is higher than those mixed in the laboratory because of the lower water content that existed in the field. As with CBR, the water content at which the mixture is prepared significantly affects the resilient modulus of the soils. Acosta et al. (2003) also report decreasing resilient modulus with increasing water content of soil-fly ash mixtures. Comparison of the in situ stiffness of the unstabilized and stabilized soil (as measured with the SSG) also demonstrates the efficiency of stabilization (Fig. 4). The in situ stabilized soil is 1.9x stiffer, on average, than the unstabilized compacted soil.

In contrast to the findings for CBR, M_r increased as curing continued (Fig. 3, open symbols = 7 d and closed symbols = 28 d). On average, the M_r after 28 d of curing was 2.2x higher than the M_r after 7 d of curing. The sensitivity to deviator stress also increased modestly as the mixture cured (Fig. 3). Moreover, for the specimens prepared with soil from 613+50 and 614+50, the slope of the relationship between M_r and deviator stress changed sign (positive to negative) between 7 and 28 d of curing.

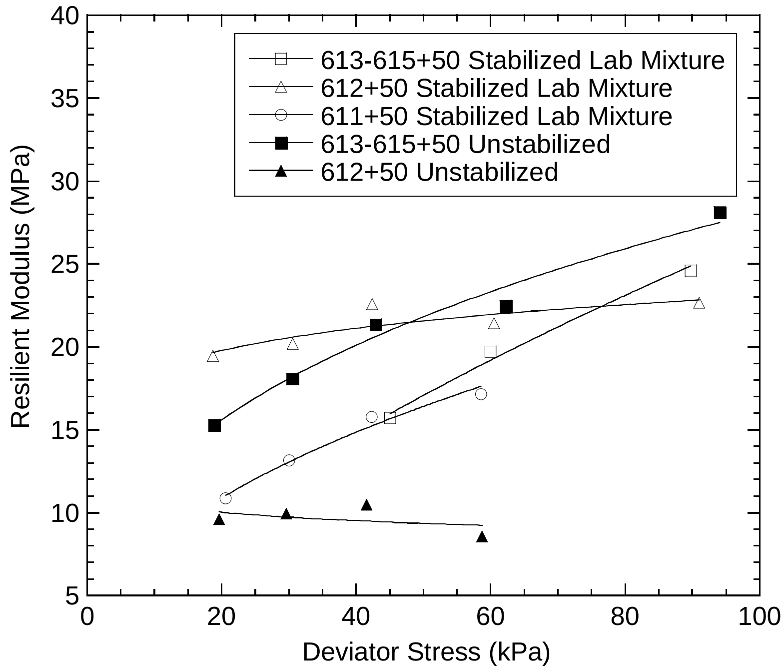

Fig. 2. Resilient modulus of unstabilized soil compacted at optimum water content and stabilized subgrade mixed in the laboratory using soil and fly ash collected during construction and prepared 5% wet of optimum water content using standard Proctor effort.

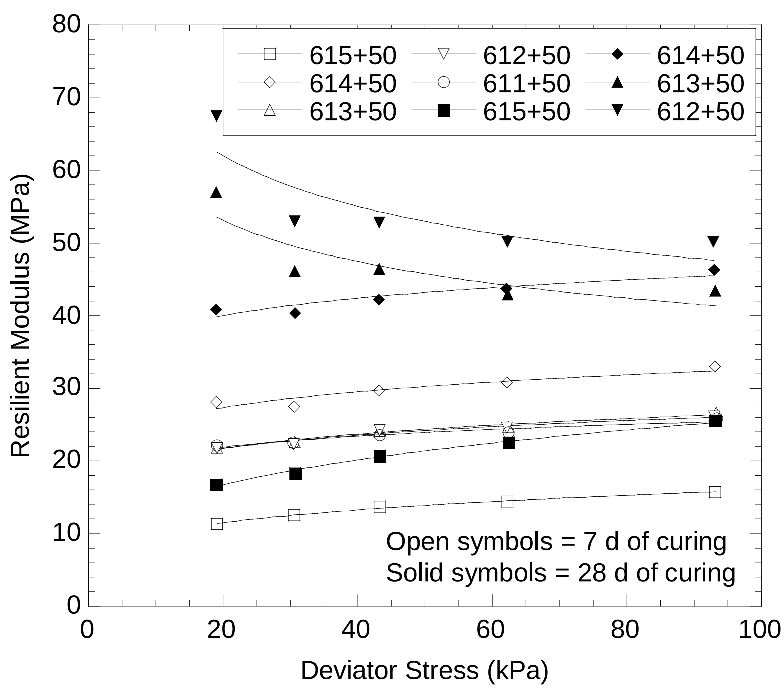

Fig. 3. Resilient modulus of stabilized subgrade compacted in the field. Specimens cured for 7 d (open symbols) and 28 d (closed symbols).

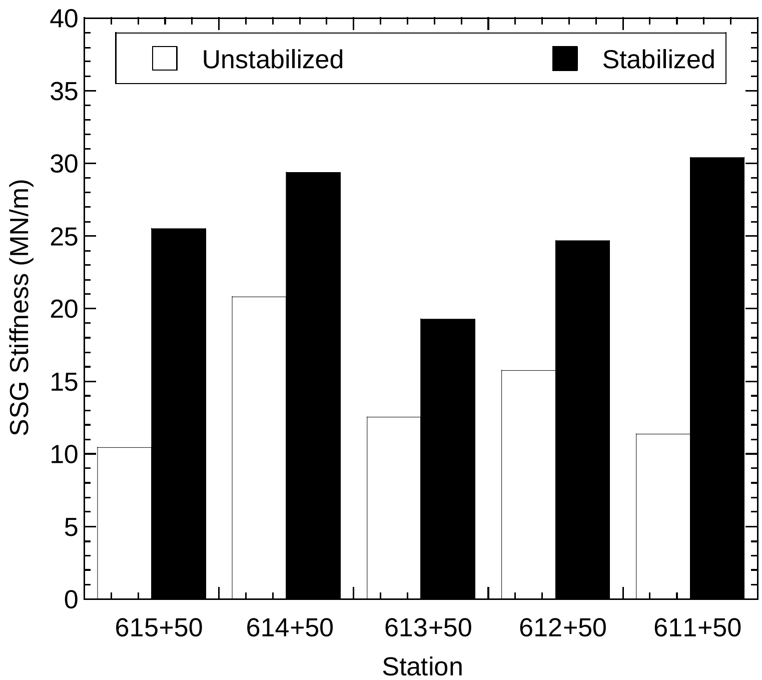

Fig. 4. Stiffness of unstabilized and stabilized subgrade measured in the field using a soil stiffness gage. Tests on stabilized soils conducted after 7 d of curing.

Unconfined Compressive Strength

Results of the unconfined compression tests are summarized in Table 5 and Fig. 5. Unconfined compressive strengths are not reported for unstabilized specimens compacted 5% wet of optimum water content because these specimens failed during resilient modulus testing.

Table 5. Unconfined compressive strength of unstabilized and stabilized soil. Laboratory mixture was prepared using soil and fly ash collected during construction.

Station	Unconfined Compressive Strength (kPa)			
	Unstabilized	Stabilized		
	Optimum	7-d Lab. Mixture (5% Wet of Optimum)	7-d Field Mixture (≈ Optimum)	28-d Field Mixture (≈ Optimum)
615+50	200	250*	448	497
614+50			607	303
613+50			448	517
612+50	145	283	490	683
611+50	110*	200*	276	D
Average	163	244	454	500
Std. Dev.	47.3	41.8	119.0	155.0

Notes: D = disturbed sample that could not be tested. Asterisks = estimated from M_r test specimen because specimen failed during resilient modulus testing.

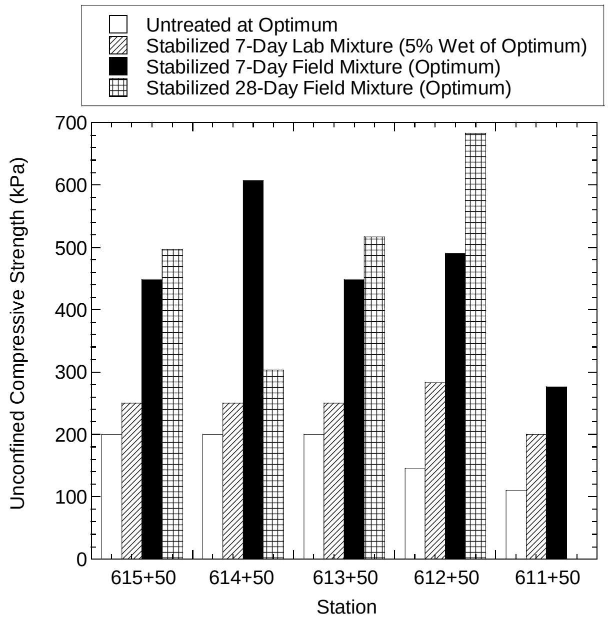

Fig. 5. Unconfined compressive strength of unstabilized and stabilized subgrade prepared in the laboratory and field after 7 and 28 d of curing.

Stabilization had a marked effect on unconfined compressive strength. The stabilized specimens prepared using laboratory mixtures at 5% wet of optimum water content had unconfined compressive strengths nearly 50% larger than the unstabilized soil prepared at optimum water content. Much larger unconfined compressive strengths were obtained for the field mixtures (compacted near optimum water content). Unconfined compressive strengths of the field mixtures (after 7 d of curing) were 2.7x higher, on average, than the unconfined compressive strengths of the unstabilized specimens compacted at optimum water content in the laboratory.

Comparison of the unconfined compressive strengths after 7 and 28 d of curing (Table 5 and Fig. 5) indicates that the strength gain from additional curing is modest (1.1x, on average). This finding is analogous to that obtained from the CBR test (Table 3), which is reasonable given that both tests are measure of shear strength, rather than stiffness.

Falling Weight Deflectometer

Back analyses of the FWD data using EVERCALC were conducted assuming three-layer profile consisting of 0.23 m of concrete, 0.20 m of base course, and an infinitely thick stabilized layer. The modulus of the concrete was allowed to vary between 7000 to 48,500 MPa and the Poisson's ratio was set at 0.20. The base course was assumed to have a Poisson's ratio of 0.30 and the modulus was allowed to vary between 100 to 7000 MPa. The stabilized layer was assumed to have a Poisson's ratio of 0.30.

Elastic moduli obtained from back-analysis of the FWD data are summarized in Table 6 along with moduli obtained from the M_r and SSG tests. Elastic moduli (E) were computed from the stiffness measured with the SSG using (Sawangsuriya 2001):

$$E = \frac{K\left(1 - \upsilon^2\right)}{1.77\ R} \tag{1}$$

where R is the outside radius of the SSG foot and υ is Poisson's ratio. The elastic moduli obtained from the SSG and the FWD back-analysis are comparable, and mostly in the range of 200 and 250 MPa. In contrast, the M_r measured after 7 d of curing are approximately one order of magnitude lower than the moduli from the SSG and FWD back-analysis. Tanyu et al. (2003) report similar differences between in situ measurements of modulus and M_r, and attribute the differences to strain level imposed in each test. The strain during a SSG test ranges between 0.001and 0.01%, whereas typically the strain in the M_r test typically is between 0.01 and 0.1% (Sawangsuriya et al. 2003).

Table 6. Elastic moduli from M_r, SSG, and FWD tests.

Station	**Center Deflection at 90 kN (mm)**	**Resilient Modulus (MPa)**		**SSG Elastic Modulus (MPa)**	**FWD Elastic Modulus (MPa) 3-layer**
		7 d	**28 d**	**7 d**	**31-Day**
615+50	0.18	11	17	242	241.3
614+50	0.16	28	41	279	206.8
613+50	0.19	22	57	183	241.3
612+50	0.21	22	68	234	206.8
611+50	0.19	22	D*	288	241.3
Average	0.19	21.0	45.8	245.2	227.5
Std. Dev.	0.02	6.16	22.14	41.77	18.90

*D = disturbed sample that could not be tested.

Although the moduli back-calculated from the FWD data are consistent with those measured with the SSG, there is considerable uncertainty regarding the validity of the moduli from the FWD tests. The center deflections obtained with FWD were very small (< 0.2 mm), which probably minimized deformation of the stabilized soil. Greater deflections might have been obtained had a larger load been used, but small loads were required to prevent damage to the new pavement surface. Moreover, some of the deflection may reflect the movement between adjacent slabs. The slabs are saw cut and are tied together by steel dowels.

SUMMARY AND CONCLUSIONS

This paper has described a case history where Class C fly ash was used successfully to stabilize a sandy clay subgrade so that a firm working platform could be provided for pavement construction. Laboratory tests were conducted on subgrade soil alone and the soil-fly ash mixtures to assess how fly ash stabilization improves the California bearing ratio (CBR), resilient modulus (M_r), and unconfined compressive strength (q_u) of the soil. Field tests were also conducted using a soil stiffness gauge (SSG) and a falling weight deflectometer (FWD) to measure the stiffness of the stabilized soil in situ.

All of these tests indicated that the stiffness and strength of the soil were improved significantly by fly ash stabilization. The average increased CBR from 2 to 85, the average M_r increased from near zero to 21 MPa, and q_u increased from less than 200 kPa to 454 kPa, on average. The test data also showed that the effectiveness of stabilization decreases when the water content of the soil increases. Analysis of the field tests results also confirmed that fly ash stabilization resulted in a relatively stiff subgrade.

Tests on specimens cured for 7 d and 28 d showed that curing affects the modulus, but not strength. M_r increases as curing occurs, and appears to become more sensitive to the stress level during the M_r test. In contrast, CBR or q_u, remain essentially unchanged after 7 d of curing.

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REFERENCES

Acosta, H., Edil, T., and Benson, C. (2003). Soil Stabilization and Drying Using Fly Ash. Geo Engineering Report No. 03-03, University of Wisconsin-Madison, Madison, Wisconsin.

Bin-Shafique, S., Benson, C., and Edil, T. (2002). Leaching of Heavy Metals from Fly Ash Stabilized Soils Used in Highway Pavements, Geo Engineering Report 02-14, University of Wisconsin-Madison, Madison, Wisconsin.

Edil, T., Benson, C., Bin-Shafique, M., Tanyu, B., Kim, W., and Senol, A. (2002), Field Evaluation of Construction Alternatives for Roadway Over Soft Subgrade, *Transportation Research Record*, *Journal of the Transportation Research Board*, 1786. 36-48.

Sawangsuriya, A. (2001). Evaluation of the Soil Stiffness Gauge. MS Thesis, University of Wisconsin-Madison, Madison, Wisconsin.

Sawangsuriya, A., Edil, T., and Bosscher, P. (2003). Relationship Between Soil Stiffness Gauge Modulus and Other Test Moduli for Granular Soils. *Transportation Research Record, Journal of the Transportation Research Board*. in press.

Tanyu, B., Kim, W., Edil, T., and Benson, C. (2003). Comparison of Laboratory Resilient Modulus with Back-Calculated Elastic Modulus from Large-Scale Model Experiments and FWD Tests on Granular Materials. *Resilient Modular Testing for Pavement Components*. STP 1437, ASTM International, West Conshohocken, PA, USA. 191-208.

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