INCORPORATING A FLY ASH STABILIZED LAYER INTO PAVEMENT DESIGN – CASE STUDY

by

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ABSTRACT

Pavements were designed and constructed at two sites in southern Wisconsin employing a layer stabilized in situ with fly ash. One pavement is for a residential subdivision. The other is a test section located in a secondary highway that was recently reconstructed. A control test section employing a conventional cut-and-fill approach was also constructed in the secondary highway. Fly ash was used to increase the strength and stiffness of the fine-grained subgrade at both sites, which was soft prior to stabilization. Pavements at both sites were designed using the 1993 AASHTO method for flexible pavements so that their structural number would be equivalent to that of the conventional pavement originally called for in the design. Measurements of California bearing ratio (CBR) and resilient modulus (M_r) were used with the correlation charts for granular subbase materials in the AASHTO manual to define layer coefficients for the stabilized layers. Tests were also conducted on specimens collected during construction to verify that the in situ mixture had similar properties as anticipated during design. The pavement at one of the sites is being monitored seasonally using a falling weight deflectometer and pavement distress surveys. The monitoring program has indicated that the pavements constructed with fly ash stabilized layers provide comparable stiffness to the conventional pavements employing a cut-and-fill approach. No signs of distress have been observed in the pavements constructed with a stabilized layer.

Keywords: Fly ash, soil stabilization, poor subgrade, soft soil, industrial by-products, layer coefficient, pavement design.

INTRODUCTION

Fly ash is an industrial byproduct of coal combustion at electric power plants that is generated in large quantities (63 Mg/yr in the US alone) each year (ACAA 2000). Combustion of sub-bituminous coal produces a fly ash (Class C) that has self-cementing characteristics that can be used for soil stabilization without activators to improve the mechanical properties of soil (Ferguson 1993). In most subgrade applications, fly ash is used to stabilize a soft soil so that a stable working platform is provided for highway construction equipment. Reducing plasticity and shrink-swell potential of fine-grained soils is also a common objective (Nicholson and Kashyap 1993).

The stabilized material typically is strong and stiff, but neither of these properties normally is considered in pavement design. Recent field data show, however, that fly ash stabilized layers provide appreciable structural support to a pavement system (Edil et al. 2002). Accounting for the structural support provided by the stabilized layer during design can result in a less costly pavement through reduction in the thickness of the base and asphalt layers. However, currently there is no standard or accepted method for designing pavements using fly ash stabilized soil (Turner 1997).

This report describes a case history where the structural support afforded by a fly ash stabilized layer was accounted for explicitly during the design of two flexible pavements. Procedures described in the 1993 AASHTO method for flexible pavements (AASHTO 1993) were followed. Layer coefficients used in the design were estimated from data collected from California bearing ratio (CBR) and resilient modulus (M_r) tests conducted on fly ash stabilized soil and charts in the AASHTO manual for granular subbase materials. Laboratory tests were also conducted on specimens prepared from the in situ mixture of soil and fly ash to determine if the properties anticipated during design were achieved in the field. In addition, the dynamic penetration index (DPI) and stiffness of the stabilized layer were measured at both sites during construction. Falling weight deflectometer (FWD) tests and pavement distress surveys were also conducted after construction to assess the performance of the pavement.

SITES

The pavements were constructed at two sites in southern Wisconsin in August 2000. Layouts of the field sites are shown in Fig. 1. One pavement is a 0.3-km-long test section in a segment of Wisconsin State Trunk Highway (STH) 60 (Fig. 1a) that is located between Lodi and Prairie du Sac, WI (Stn. 261+50 to 271+50). The other is a 0.7 km city street in the recently constructed Scenic Edge residential sub-division in Cross Plains, WI (Fig. 1b).

At the Scenic Edge site, the pavement design originally called for a conventional cut-and-fill approach where the upper 0.75 m of soft subgrade would be replaced with coarse sand prior to placement of base course (175 mm) and asphalt concrete (125 mm). Residents in the area opposed this design because it required trucking large amounts of earthen materials through local neighborhoods. To alleviate this problem, in situ stabilization of the soft subgrade was selected. However, in contrast to conventional stabilization projects, the pavement was designed so that it would provide equivalent structural number (SN) as the original design by explicitly accounting for the support provided by the stabilized layer.

The test section at the STH 60 site was constructed as part of a larger project evaluating various methods to stabilize poor subgrades in Wisconsin (Edil et al. 2002). The design for STH 60 also included a conventional cut-and-fill approach to address soft subgrade at the site. The soft subgrade was to be cut and replaced with at least 0.45 m of crushed rock referred to as “breaker run.” The remainder of the conventional profile consisted of base course (255 mm) and asphalt concrete layer (125 mm). As at the Cross Plains site, the test section employing a fly ash stabilized layer at STH 60 was designed to have a structural number equal to that of the conventional pavement profile.

MATERIALS

Soils

Samples of the subgrade were collected along the centerline of the proposed roadway at Scenic Edge and near the shoulder of the roadway in STH 60. Undisturbed samples were also collected from both sites along the centerline of the proposed roadway using thin-wall sampling tubes. Tests were conducted to determine index properties, soil classification, compaction characteristics, unconfined compressive strength, and CBR of the subgrade. Compaction curves for the subgrade were determined using the Harvard miniature method using standard effort (ASTM D 4609). Unconfined compression tests were conducted following ASTM D 2166 on undisturbed specimens (50 mm-diameter and 100 mm-height) trimmed from the tube samples. CBR tests were conducted on laboratory-compacted specimens prepared at the in situ water content and unit weight of the subgrade. A summary of the properties is tabulated in Table 1. Particle size distribution curves for the soils are shown in Fig. 2.

The subgrade classifies as A-7-6 or CL at the Scenic Edge site and A-6 or CL-ML at the STH 60 sites. Both subgrade soils contain more than 90% fines (particles finer than 75 μm) and have a 2-μm clay fraction between 15 and 20%. The in situ water content of the subgrades is 6-7% wet of optimum water content based on standard effort (ASTM D 698). At Scenic Edge, the CBR is 1 and the unconfined compressive strength ranges between 40 and 180 kPa (average = 95 kPa). At STH 60, the CBR is 3 and the unconfined compressive strength ranges between 105 and 136 kPa (average = 124 kPa). Thus, both subgrades are soft to medium stiff soils, which is largely due to their high water content.

The DPI and stiffness of the subgrade were measured at intervals of 30 m to 60 m along the centerline of the roadway after the roadbed was prepared for construction (Sawangsuriya 2001, Albright 2002). DPI was measured with a dynamic cone penetrometer (DCP) that measures the depth of penetration of a cone having a 60^0 apex and diameter of 20 mm that is driven with an 8 kg hammer dropped from a height of 522 mm. The DPI is calculated as the average penetration per blow of the hammer over a depth of 250 mm. Stiffness was measured with a Humboldt soil stiffness gauge (SSG), which measures stiffness of the soil near the surface. The DPI ranged between 20-120 (mm/blow) (average = 64 mm/blow) and the stiffness ranged between 4-9 MN/m (average = 5 MN/m) for the Scenic Edge site. At the STH 60 site, the DPI ranged between 38-112 (mm/blow) (average = 65 mm/blow) and the stiffness ranged between 4-8 MN/m (average = 5 MN/m).

Fly Ash

Fly ash from Unit 2 of Alliant Energy's Columbia Power Station in Portage, Wisconsin was used for stabilization at both field sites. Chemical composition of the fly ash is summarized in Table 2, and its particle size distribution curve is shown in Fig. 2. The specific gravity of the fly ash is 2.68 and the loss on ignition is 0.7%.

Columbia fly ash contains 23% lime (CaO), and is a Class C fly ash per ASTM C 618. The fly ash is comprised primarily (89%) of silt size particles.

Stabilized Soil

Mixtures of soil and fly ash were prepared with both soils using fly ash contents of 10, 14, and 18% (STH 60) or 12, 16, and 20% (Scenic Edge). Air-dry soil ground to pass a US No. 10 sieve was first mixed with the fly ash until the mixture appeared uniform. Tap water was then sprayed on the soil-fly ash mixture to achieve a target water content based on total solids (soil mineral and fly ash solids). Mixing continued during moistening to promote uniform water content and hydration.

Two sets of specimens were prepared with the soil-fly ash mixtures. One set was compacted in a mold immediately after mixing with water (referred to herein as “no delay”). The other set was compacted 2 hours after mixing with water (referred to herein as “2-hr delay”) to simulate the typical duration between mixing and compaction that occurs in the field (Ferguson 1993). All specimens were compacted in a Harvard miniature mold (35 mm-diameter and 70 mm-height) using standard effort (ASTM D 4609).

Compaction characteristics of the un-stabilized and stabilized soils are summarized in Table 3. For both soils, maximum dry unit weight and optimum water content for the mixture prepared with “no delay” are comparable to those for the soil alone. However, for the “2-hr delay," the maximum dry unit weight is lower and optimum water content is slightly (1%) higher than those for the soil alone. Additionally, the maximum dry unit weight decreases and the optimum water content increases as the fly ash content increases. The changes in compaction characteristics of the 2-hr delay mixtures reflect the cementing that occurs as the fly ash hydrates during the 2-hr delay. Cementing causes the clods of clay to become stronger, and more difficult to remold. As a result, less solid material can be compacted into a unit volume.

DESIGN PHASE

Structural Requirement

The pavements were designed using the 1993 AASHTO method for flexible pavements. Structural support afforded by the stabilized layer was directly accounted for in the design by treating the stabilized layer as a subbase and assigning it a layer coefficient. In particular, the pavement was designed so that it would have the same structural number (SN) as the conventional cut-and-fill pavement. The SN was computed as:

$$SN = t_1a_1 + t_2a_2 + t_3a_3 \quad (1)$$

where t_1, t_2, and t_3 are the thickness of the surface course, base course, and the stabilized layer, and a_1, a_2, and a_3 are the layer coefficients for surface, base, and stabilized layer. A key difference in this approach is that a_3 and t_3 characterize the stabilized layer rather than a conventional subbase soil. The stabilized layer performs the same functions, and effectively is a subbase layer because it is directly below the base course and directly above the natural subgrade.

At both sites, the same surface and base layers were used as called for in the conventional design. That is, the only change was replacement of the third layer (i.e., 0.45 m breaker run at STH 60 and 0.75 m coarse sand at Scenic Edge) with a fly ash stabilized layer that would provide equivalent contribution to the structural number.

The thickness of the stabilized layer was also constrained to be 0.3 m (i.e., the depth to which fly ash could be mixed into the in situ subgrade). This required that the stabilized soil have an a_3 of at least 0.12 for Scenic Edge and 0.11 for STH 60.

CBR and M_r are the critical parameters affecting a_3. The AASHTO Guide provides charts relating a_3 to CBR and M_r, but these charts apply to granular materials typically used as subbase, not fly ash stabilized soils that rely on cementing to increase strength and stiffness. No such charts exist for fly ash stabilized soils. Thus, in lieu of charts specifically for fly ash stabilized layer, the charts for granular subbase were assumed to apply to fly ash stabilized soil. The field testing, described subsequently, was conducted in part to determine if this assumption is reasonably valid.

Laboratory Testing for Design

A series of specimens were prepared with different fly ash contents and compacted at different molding water contents (based on total solids) to define a mix-design that would provide equivalent structural support as the conventional subbase layers used at the Scenic Edge and STH 60 sites. The testing program for design consisted of two phases. In the first phase, a series of unconfined compression tests were conducted to evaluate the general effects of mix-design variables (i.e., fly ash content, molding water content, and compaction delay) on strength of the fly ash mixture. In the second phase, CBR and M_r tests were conducted under target conditions identified from the unconfined compression tests. Results of the CBR and M_r tests were then used to determine the fly ash content needed to achieve the required a_3.

This two-phase approach was used for pragmatic reasons. Unconfined compression tests were conducted first because specimens prepared for compaction tests could also be used for unconfined compression testing. Thus, no additional effort was required to prepare the specimens. Moreover, unconfined compression tests could be conducted rapidly, which allowed for a quick assessment of the influence of mix-design variables on mechanical properties of the mixtures. By following this approach, only a limited number of the more difficult and time-consuming CBR and M_r tests needed to be conducted.

Unconfined Compression Tests

Specimens prepared for compaction tests were used for unconfined compression testing. Compaction test specimens were wrapped with plastic wrap, allowed to cure for 7 d in a wet room (100% relative humidity), and then tested for unconfined compressive strength following ASTM D 2166.

Unconfined compressive strength is shown as a function of fly ash content and molding water content in Fig. 3 (Scenic Edge) and Fig. 4 (STH 60). Adding fly ash to both soils increased the compressive strength appreciably (by at least a factor of two, and as much as a factor of seven). Slightly higher compressive strengths were obtained at higher fly ash contents, due to greater cementing in the stabilized soil. For both soils, the maximum unconfined compressive strength was obtained approximately at optimum water content for the “no delay” condition and at a water content 1% wet of optimum for the “2-hr delay” condition. Lack of sufficient water for hydration at lower water content, and reduction of contact areas (for bonding) among soil particles at higher water content is responsible for the reduction in compressive strength as the water content deviates from optimum (ACAA 1999). Compaction after a 2-hr delay caused the strength to decrease by as much as 25%, primarily because compaction breaks down some of the bonds that form during the first two hours of hydration.

California Bearing Ratio (CBR) and Resilient Modulus Tests

Based on the outcome of the unconfined compression tests, CBR and M_r tests were conducted at three different fly ash contents (12, 16, and 20% for Plano silt loam and 10, 14, and 18% for Joy silt loam). These fly ash contents were selected because the unconfined compression tests indicated that fly ash contents higher than 20% would provide little improvement, and that fly ash contents as low as 10-12% might yield acceptable properties at lower cost. An intermediate fly ash content was tested as well in case the CBR or M_r achieved at the lowest fly ash content was too small. The tests were conducted at water content corresponding to the water content at which the unconfined compressive strength was maximum.

Specimens for CBR testing were mixed and moistened using the same procedure used for the compaction tests. The mixture was compacted into a CBR mold using standard effort immediately after mixing, or after a 2-hr delay. Specimens for resilient modulus testing were prepared in a split mold using a similar method, except that only 2-hr delay specimens were prepared. After compaction, the specimens were sealed in plastic and cured for seven days in the wet room. The CBR specimens were cured in the mold, whereas the M_r specimens were extruded prior to curing. After curing, the specimens were tested following ASTM D 1883

(CBR) and AASHTO Standard T 294 (M_r). The M_r test was conducted following the protocol for Type 2 (cohesive) materials.

CBR of the fly ash stabilized soils is shown in Fig. 5. As with unconfined compression, adding fly ash increases the CBR significantly. Also, the effect of fly ash addition on CBR is reduced by a 2-hr delay in compaction. For Scenic Edge, the CBR is sensitive to fly ash content, whereas for STH 60 the CBR increases only slightly as the fly ash content increases. For both soils, CBRs in excess of 37 (Scenic Edge) and 32 (STH 60) were achieved at the lowest fly ash content, even with a 2-hr delay. These CBRs are higher than CBRs generally associated with good subbase materials, and are more typical of CBRs associated with base materials (Bowles 1992).

Resilient modulus of fly ash stabilized soil at different fly ash contents is shown in Fig. 6. No M_r are shown for soil without fly ash because the soil was so soft that the specimens failed during the first loading cycle of the M_r test. As with the CBR tests, addition of fly ash resulted in relatively large M_r (at least 90 kPa), and somewhat larger M_r were obtained as the fly ash content increased.

Mixture Selection

Results of the CBR and M_r tests were used to select the fly ash content to be used at each site. For both sites, the lowest fly ash content that was tested was selected for field application because it provided an a_3 exceeding that required for the subbase layer in the conventional design. Since the strength and stiffness decreases significantly due to compaction delay, CBR and M_r obtained from “2-hr delay” specimens were used when estimating a_3. A summary of the a_3 that were estimated for the fly ash stabilized layers are shown in Table 4, along with the a_3 assigned to the conventional subbase materials by designers of the original pavement profiles (the conventional pavement profiles were not designed by the authors).

FIELD CONSTRUCTION

Based on the laboratory mix-design, the subgrade was stabilized using a fly ash content of 12% for the Scenic Edge site and 10% for the STH 60 site. The intended mixture water content was 1% wet of optimum (based on total solids) to achieve maximum strength, assuming that a 2-hr delay was realistic. Prior to placing the fly ash, the existing water content of the subgrade was measured with a nuclear density gage. Water was added as needed so the soil-fly ash mixture would be at the target water content.

The fly ash was spread onto the moist subgrade in a relatively uniform layer using truck-mounted lay-down equipment designed specifically for fly ash application with minimal dust generation (Edil et al. 2002). After placing the fly ash over a road segment approximately 200 m long, a road reclaimer was used to mix the fly ash with the subgrade soil to a depth of 0.3 m. Immediately after mixing, the mixture was compacted using three different compactors (tamping foot, steel drum, and rubber tire) in sequence. Compaction was completed within approximately one hour after mixing the fly ash and moist subgrade soil at both sites.

A nuclear density gage was used to measure the dry unit weight and the water content that was achieved. For the Scenic Edge site, the dry unit weight varied between 93% and 106% of the target dry unit weight (16.2 kN/m^3) and averaged 98% (15.9 kN/m^3). For the STH 60 site, the dry unit weight varied between 96% and

107% of the target dry unit weight (16.5 kN/m^3) and averaged 103% (17 kN/m^3). For both sites, the water content varied within 2% of the target mixture water content.

POST-CONSTRUCTION TESTING

Laboratory Tests

Grab samples were collected from the field sites just prior to compaction. Samples were also collected after compaction using thin-wall (71 mm diameter) sampling tubes. The samples collected in tubes were extruded within 24 hr, wrapped with plastic, and cured for 7 d in a wet room. The grab samples were compacted into CBR molds immediately after sampling to the dry unit weight achieved in the field at the sampling location. After compaction, the CBR specimens were wrapped in plastic, and cured for 7 d in a wet room.

After curing, unconfined compression tests were conducted on specimens trimmed from the tube samples, and CBR tests were conducted on the specimens compacted in molds. The specimens for unconfined compression testing were trimmed to a diameter of 50 mm and height of 100 mm. At some locations, the tube samples were brittle, and broke into pieces during sampling and extrusion. A pocket penetrometer was used to estimate the unconfined compressive strength at these locations. Resilient modulus tests were not conducted after construction because intact undisturbed samples of sufficient size for M_r tests could not be retrieved.

Compressive strength and CBR of the stabilized layers are shown in Figs. 7 and 8, along with those of the original subgrades. At the Scenic Edge site, the mean unconfined compressive strength increased from 85 kPa to 370 kPa (Fig. 7a) and the CBR increased from 1 to approximately 25 (Fig. 8a). At the STH 60 site, the mean unconfined compressive strength increased from 112 kPa to 300 kPa (Fig. 7b), and the CBR increased from 3 to approximately 20 (Fig. 8b).

These increases in unconfined compressive strength and CBR are smaller than anticipated during design. The unconfined compressive strength in the field is approximately one-half of that measured during design for the Scenic Edge site, and two-thirds of that measured during design for the STH 60 site. Similarly, the CBR of the field mixture at both sites is approximately two-thirds of that anticipated during design. The reason for these differences has not been determined, but a likely factor is differences in the mixing process that occur in the laboratory and field. In the laboratory, the air-dried soil is ground into small particles and carefully mixed with fly ash, and then water is added to create a uniform material. In the field, the particle sizes are much larger (i.e., moist clods or clumps of clay are mixed with fly ash rather than finely ground soil particles). The larger clod size in the field probably reduces blending of the fly ash and soil particles, and reduces the uniformity of cementing that occurs in the soil.

Field Tests

Field testing of the stabilized layer was conducted with the DCP and SSG at both sites. Falling weight deflectometer (FWD) tests and distress surveys are being conducted semi-annually (fall and spring) at the STH 60 site. FWD tests are conducted with a KUAB Model 2m-33 FWD at 24 stations (6 in control section, 18 in fly ash section) along the STH 60 alignment using a 90 kN load. More FWD tests are conducted in the fly ash section because it is four times longer than the control section. The distress survey is conducted by the Wisconsin Department of

Transportation (WisDOT). WisDOT uses the survey to calculate the pavement distress index (PDI):

$$PDI = \left[1 - \frac{A}{B}\ L\ T\ P\ F\ E\ S\ R\ D_L D_T\right] \tag{2}$$

The parameters on the right-hand side of Eq. 1 are distress factors for alligator cracking (A), block cracking (B), longitudinal cracking (L), transverse cracking (T), patching (P), flushing (F), edge raveling (E), surface raveling (S), rutting (R), longitudinal distortion (D_L), and transverse distortion (D_T). The PDI can vary between 0 and 100, with 0 corresponding to no distress. WisDOT considers pavements having a PDI between 60-75 to be at the end of their service life. To date, WisDOT has reported PDI = 0 for all of the pavements, indicating that there is no evidence of distress.

DPI and stiffness measured with the DCP and SSG are shown in Fig. 9 and Fig. 10 for conditions before and after stabilization. DPI of the subgrade decreased on average from 64 to 20 mm/blow at the Scenic Edge and from 65 to 16 mm/blow at the STH 60 site as a result of fly ash stabilization. The stiffness increased from 5 to 12 MN/m, on average, at both sites. Additionally, at STH 60, the stiffness of the fly ash stabilized layer (12 MN/m) is higher than that of the breaker run subbase (10 MN/m) in the adjacent control test section.

Means and standard deviations of the centerline deflections measured at STH 60 with the FWD are shown in Table 5. The centerline deflection is measured at the center of the loading plate, and is an indicator of pavement stiffness. Deflections in both sections are small (< 2mm). The mean deflection in the fly ash section is slightly larger than that in the control section during the first year of monitoring (October 2000 and May 2001), but is smaller or similar to that in the control section during the second year of monitoring (October 2001 and May 2002).

A two-tailed unpaired t-test with unequal variance was conducted to determine if the apparent differences in the mean centerline deflection of the fly ash and control sections were statistically significant. The test was conducted assuming unequal variances using a conventional significance level of 5%. Results of the t-test are summarized in Table 5. The p-value is slightly larger than 0.05 for the tests conducted in May and October 2001, and is appreciably larger than 0.05 for the tests conducted in October 2000 and May 2002. Thus, in all cases the means are not different statistically, although in May and October 2001 the differences are only marginally insignificant. The apparent gain in strength and stiffness that is occurring over time in the fly ash section (as shown by the decrease in centerline deflection, Table 5) suggests that the marginal differences between the two sections should diminish over time. Monitoring of the test sections with the FWD and using distress surveys, which will continue semi-annually for five years after construction, will be used to evaluate this supposition.

SUMMARY AND PRACTICAL IMPLICATIONS

This paper has described a case study where pavements at two sites were designed and constructed using fly ash to stabilize a soft subgrade. A control test section was also constructed using a conventional cut-and-fill approach with crushed rock instead of the fly ash stabilization at one of the sites. The unique aspect of these pavements is that the structural support afforded by the fly ash stabilized soil was incorporated into the pavement design. The design followed the 1993 AASHTO method for flexible pavements, and was based primarily on laboratory measurements of CBR and resilient modulus of the fly ash stabilized soil. The relationship between CBR and layer coefficient for granular subbase in the 1993 AASHTO Guide was assumed to apply to the fly ash stabilized soil. Field tests were conducted after construction to evaluate the effectiveness of the design methodology.

At both field sites, stabilization with fly ash improved the strength and stiffness of the subgrade significantly. However, the CBR of the field mixture at both sites was approximately two-thirds of the CBR measured during design. Nevertheless, the increase in strength was more than adequate to provide a strong working platform for construction equipment, and the increase in stiffness resulted in small pavement deflections during testing with a falling weight deflectometer (< 2 mm using a 90 kN load). FWD testing at the STH 60 site showed that similar centerline deflections and stiffness (as measured with a SSG) were achieved in the fly ash and control sections, indicating that the two pavements are comparable structurally. In addition, no distress has been observed in either section since construction. Thus, assigning layer coefficients for fly ash stabilized soils based on correlations for granular subbase materials appear reasonable until layer coefficients specific to fly ash stabilized soils become available.

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TABLE 1. Properties of Subgrade Soils.

Soil (1)	Liquid limit (2)	Plasticity index (3)	Specific gravity (4)	LOI (%) (5)	Classification		Avg. q_u (kPa) (8)	CBR (9)	Avg. SSG Stiffness (MN/m) (10)	w_N (%) (11)	$\gamma_{d(CBR)}$ (kN/m^3) (12)	w_{OPT} (%) (13)	γ_{dmax} (kN/m^3) (14)
					USCS (6)	AASHTO (7)							
Scenic Edge	44	20	2.71	2	CL	A-7-6 (20)	95	1	5	27	14.6	20	16.2
STH 60	39	15	2.70	1	CL-ML	A-6 (16)	124	3	5	25	15.1	19	16.5

Notes: LOI = loss on ignition, UCSC = Unified Soil Classification System, AASHTO = Association of American State Highway and Transportation Officials, q_u = unconfined compressive strength, CBR = California bearing ratio, SSG = soil stiffness gauge, W_N = natural water content, $\gamma_{d(CBR)}$ = dry unit weight for CBR tests, w_{OPT} = optimum water content, and γ_{dmax} = maximum dry unit weight.

TABLE 2. Chemical Composition of Columbia Fly Ash and Typical Class C and F Fly Ashes.

Chemical species (1)	Percent of Composition			
	Columbia [1] fly ash (2)	Typical [2] Class C (3)	Typical [2] Class F (4)	Typical [2] Portland cement (5)
CaO (lime)	23.1	24	9	64
SiO_2	31.1	40	55	23
Al_2O_3	18.3	17	26	4
Fe_2O_3	6.1	6	7	2
MgO	3.7	5	2	2
SO_3	3.7	3	1	2

Notes: [1]Chemical analysis provided by Alliant Energy, [2]From FHWA (1995)

TABLE 3. Compaction Characteristics of Soils and Fly Ash-Stabilized Soils.

Soil type (1)	**Fly ash content (%) (2)**	**Stabilized Soil (No Delay)**		**Stabilized Soil (2-hr Delay)**		**Soil Alone**	
		γ_{dmax} (kN/M^3) (3)	**w_{opt} (%) (4)**	**γ_{dmax} (kN/M^3) (5)**	**w_{opt} (%) (6)**	**γ_{dmax} (kN/M^3) (7)**	**w_{opt} (%) (9)**
Scenic Edge	12	16.2	21	15.6	21	16.2	20
	16	16.2	21	15.5	21		
	20	16	22	15.5	22		
STH 60	10	16.6	20	16.1	20	16.5	19
	14	16.5	20	15.9	20		
	18	16.4	20	15.8	20		

Notes: γ_{dmax} = maximum dry unit weight, and w_{OPT} = optimum water content

TABLE 4. Layer Coefficients Estimated from CBR and Resilient Modulus.

Material	Specimen type	Strength Based a_3		Modulus Based a_3	
		CBR	a_3	M_r (MPa)	a_3
Fly ash stabilized soil (12% FA) Scenic Edge	Laboratory	37	0.12	115	0.12
	Field	28	0.11	NM	NM
Fly ash stabilized soil (10% FA) STH 60	Laboratory	32	0.11	99	0.11
	Field	23	0.10	NM	NM
Coarse sand (Scenic Edge)	NA	NA	0.05	NA	0.05
Breaker run (STH 60)	NA	NA	0.07- 0.08	NA	0.07- 0.08

Note: a_3 = subbase layer coefficient, FA = fly ash, M_r = resilient modulus, NA = not applicable, and NM = not measured.

TABLE 5 Statistical Analysis of Maximum Deflection Obtained from FWD at STH 60.

Date (1)	Mean Deflection (mm) (Standard Deviation)		t-Statistic (4)	p-Value (5)	Significant Difference? (6)
	Control section (2)	Fly ash section (3)			
Oct. 21, 2000	0.95 (0.17)	1.00 (0.31)	-1.02	0.32	No
May 16, 2001	1.04 (0.27)	1.23 (0.28)	-1.88	0.07	No
Oct. 12, 2001	0.72 (0.07)	0.61 (0.11)	2.33	0.06	No
May 15, 2002	0.73 (0.12)	0.74 (0.06)	-0.41	0.70	No

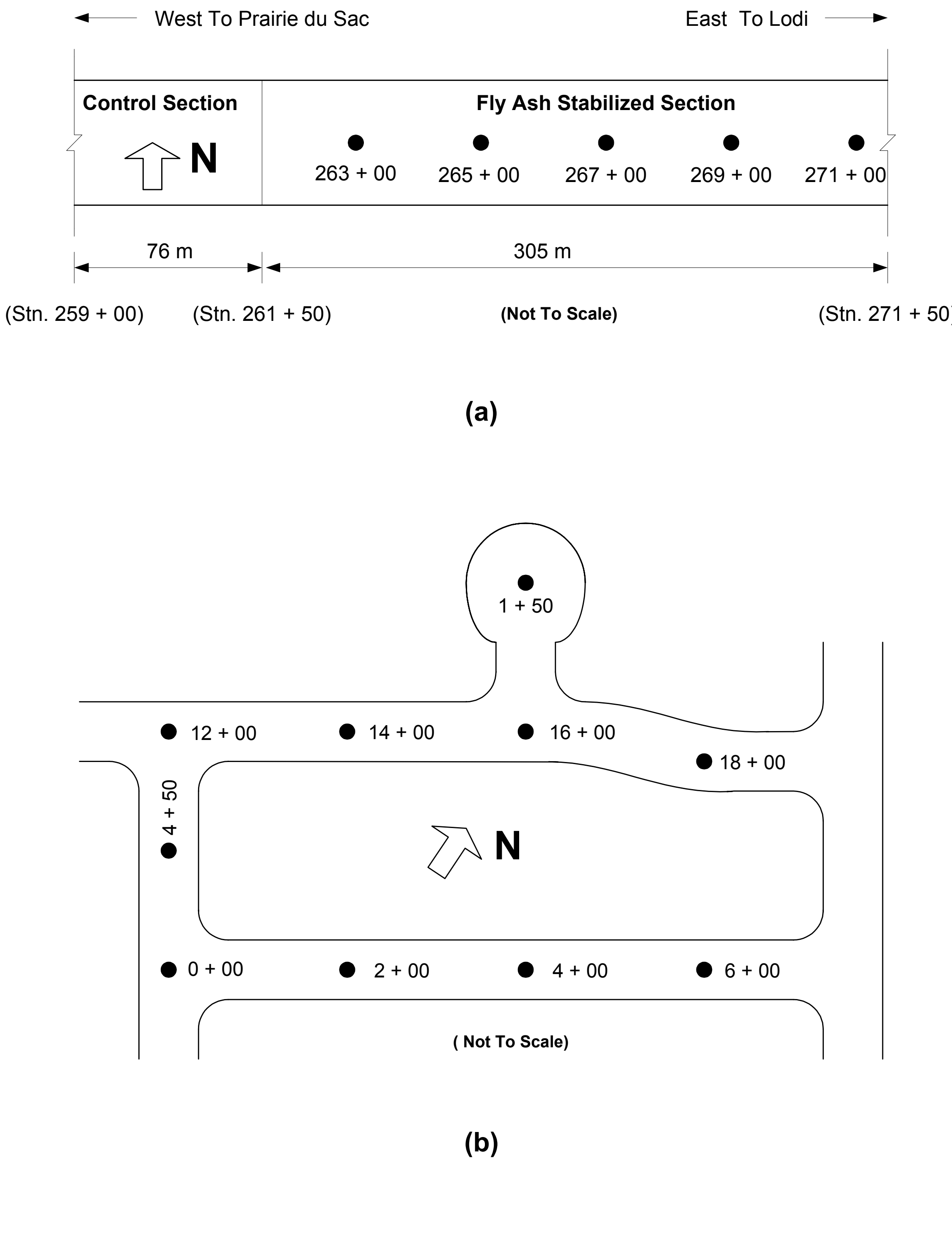


FIG. 1. Layout of Test Sections at STH 60 (a) and Layout of the Scenic Edge Site (b).

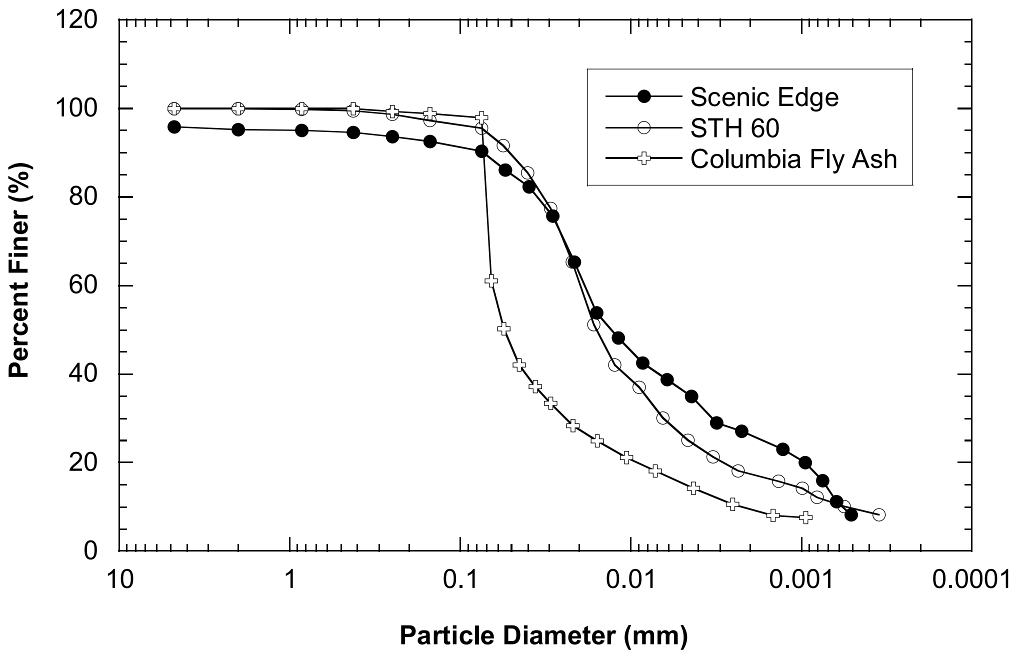


FIG. 2. Particle Size Distribution Curves for the Soils and Fly Ash.

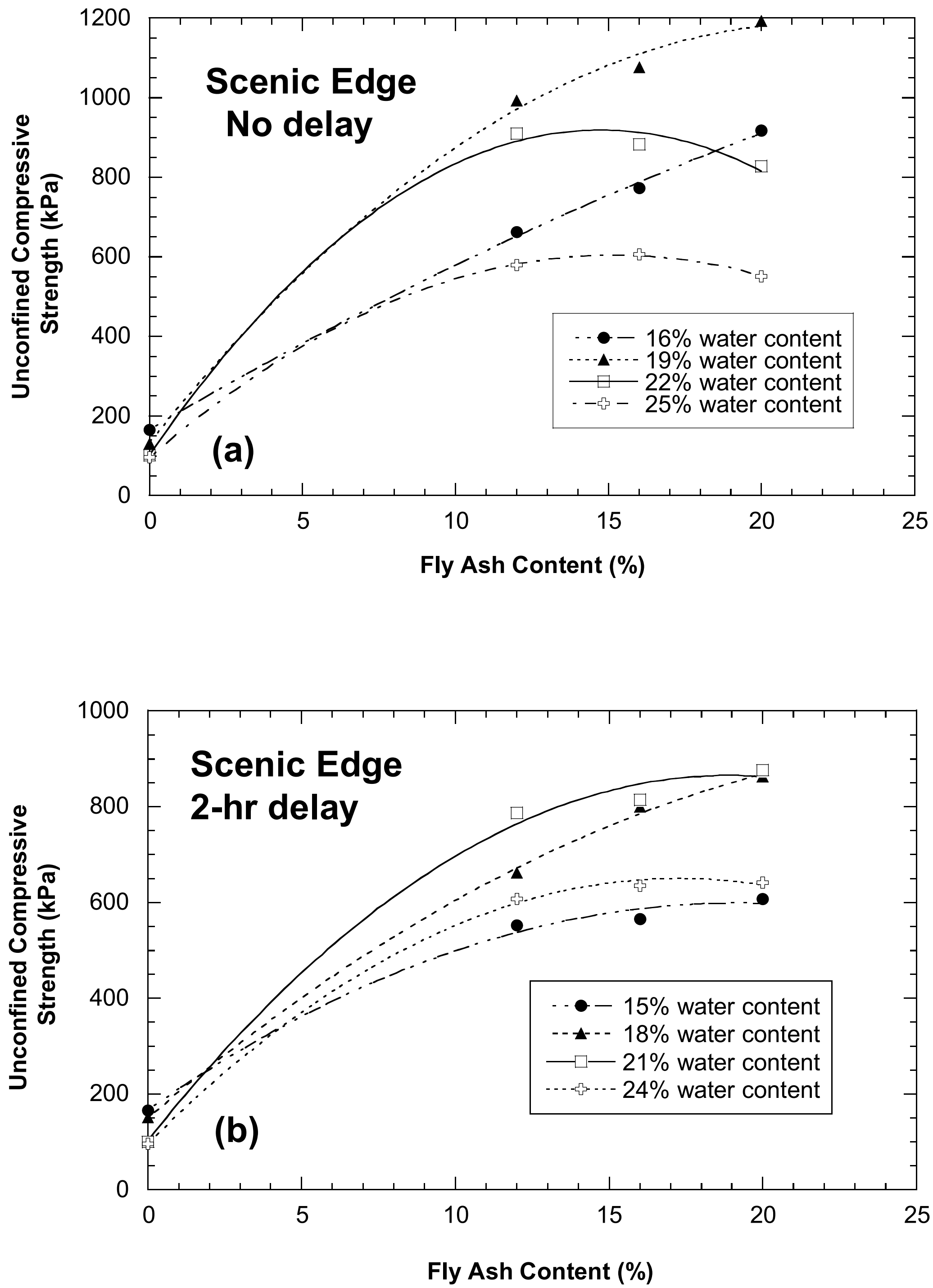


FIG. 3. Unconfined Compressive Strength of Fly Ash Stabilized Soil from Scenic Edge at Various Fly Ash and Molding Water Contents: (a) No Compaction Delay and (b) 2-hr Delay.

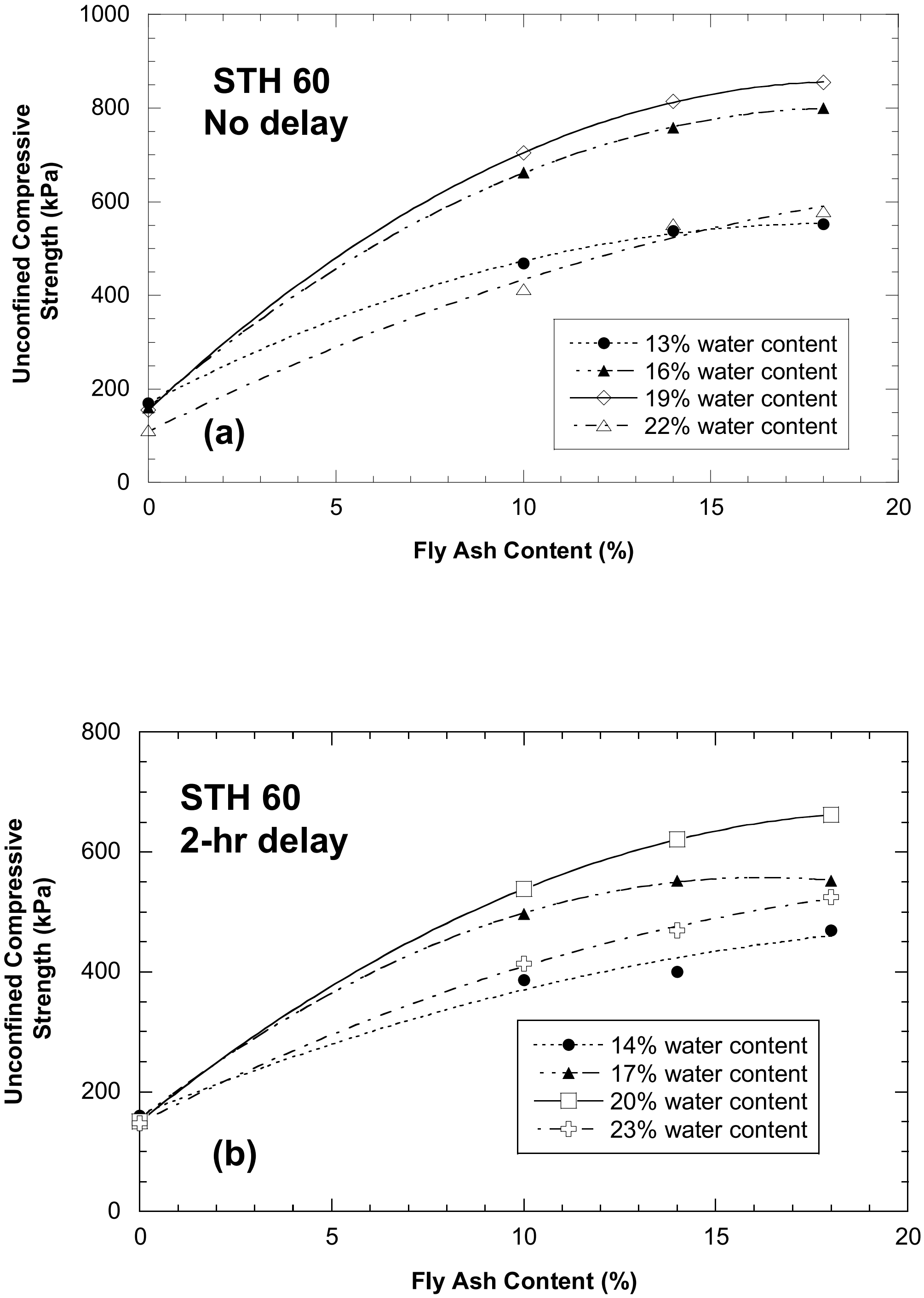


FIG. 4. Unconfined Compressive Strength of Fly Ash Stabilized Soil from STH 60 at Various Fly Ash and Molding Water Contents: (a) No Compaction Delay and (b) 2-hr Delay.

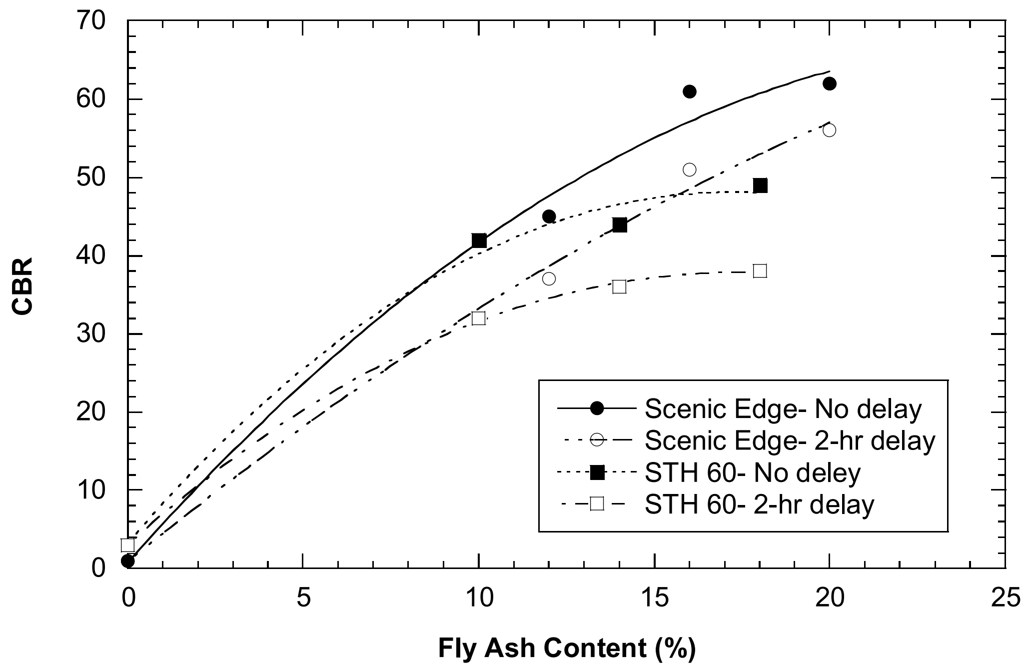


FIG. 5. California Bearing Ratio (CBR) of Fly Ash Stabilized Soils at Various Fly Ash Contents.

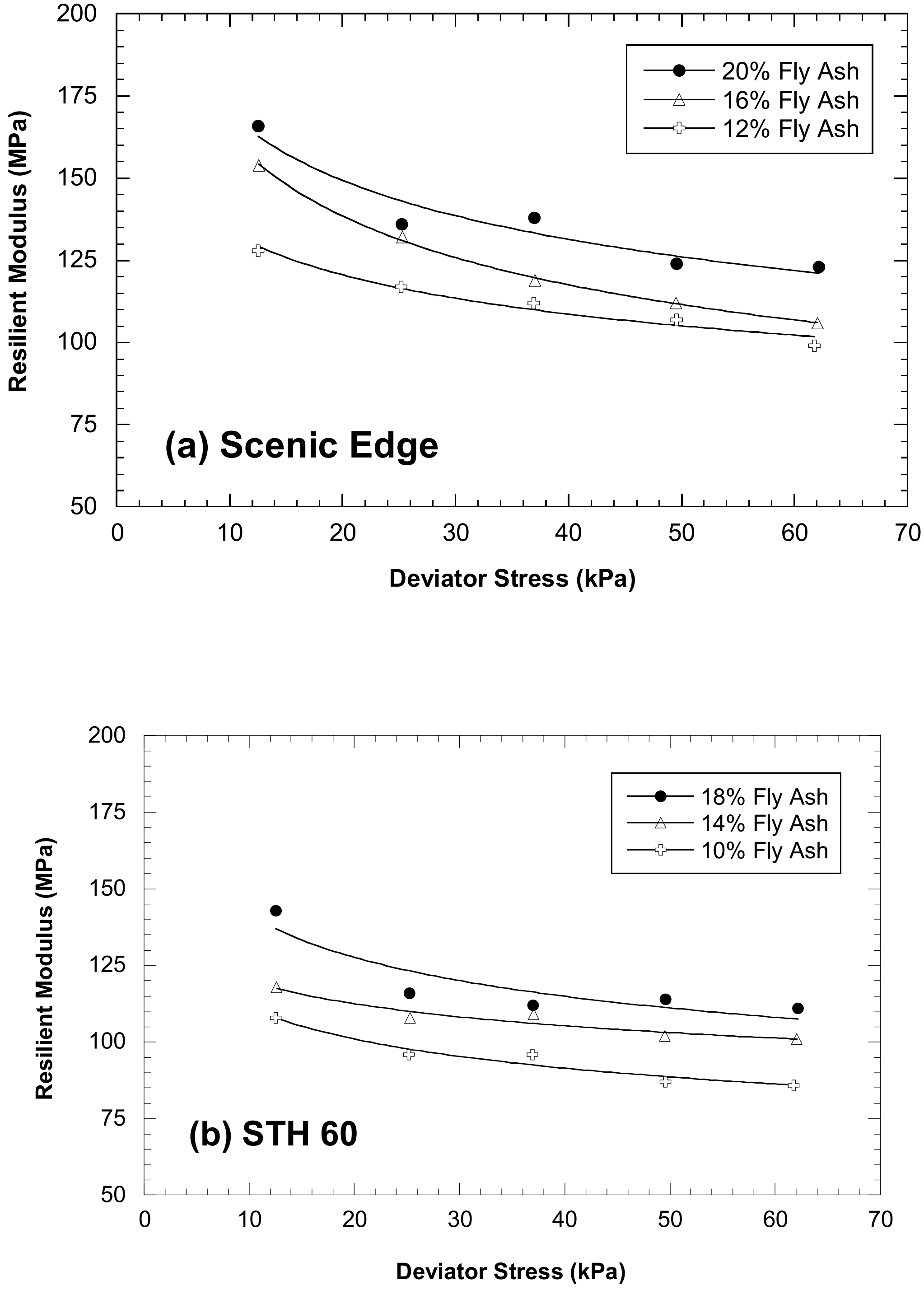


FIG. 6. Resilient Modulus of Fly Ash Stabilized Soils at Various Fly Ash Contents: (a) Scenic Edge and (b) STH 60.

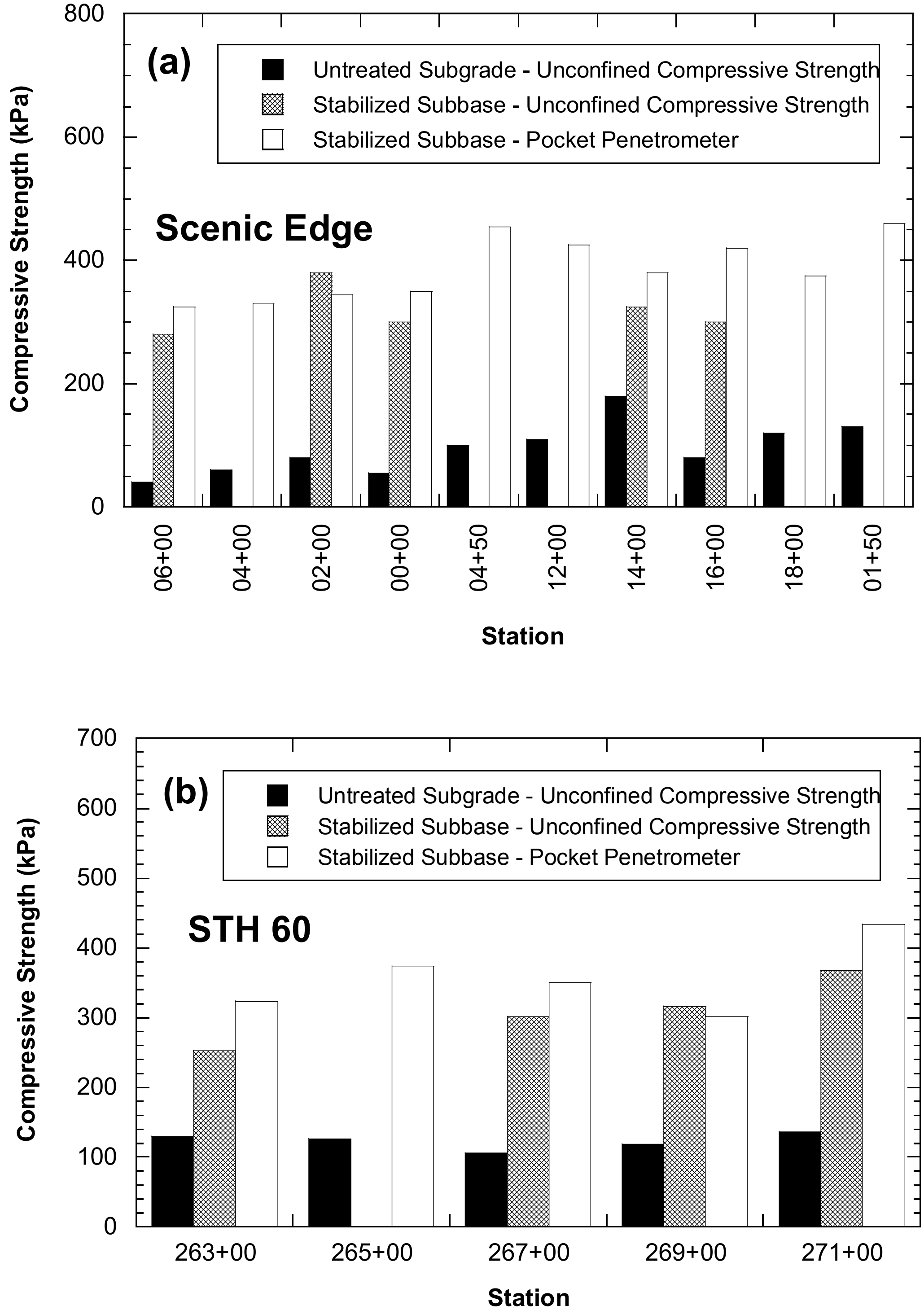


FIG. 7. Compressive Strength of Untreated Subgrade and Fly Ash Stabilized Layer Along Center Line of Pavement: (a) Scenic Edge and (b) STH 60.

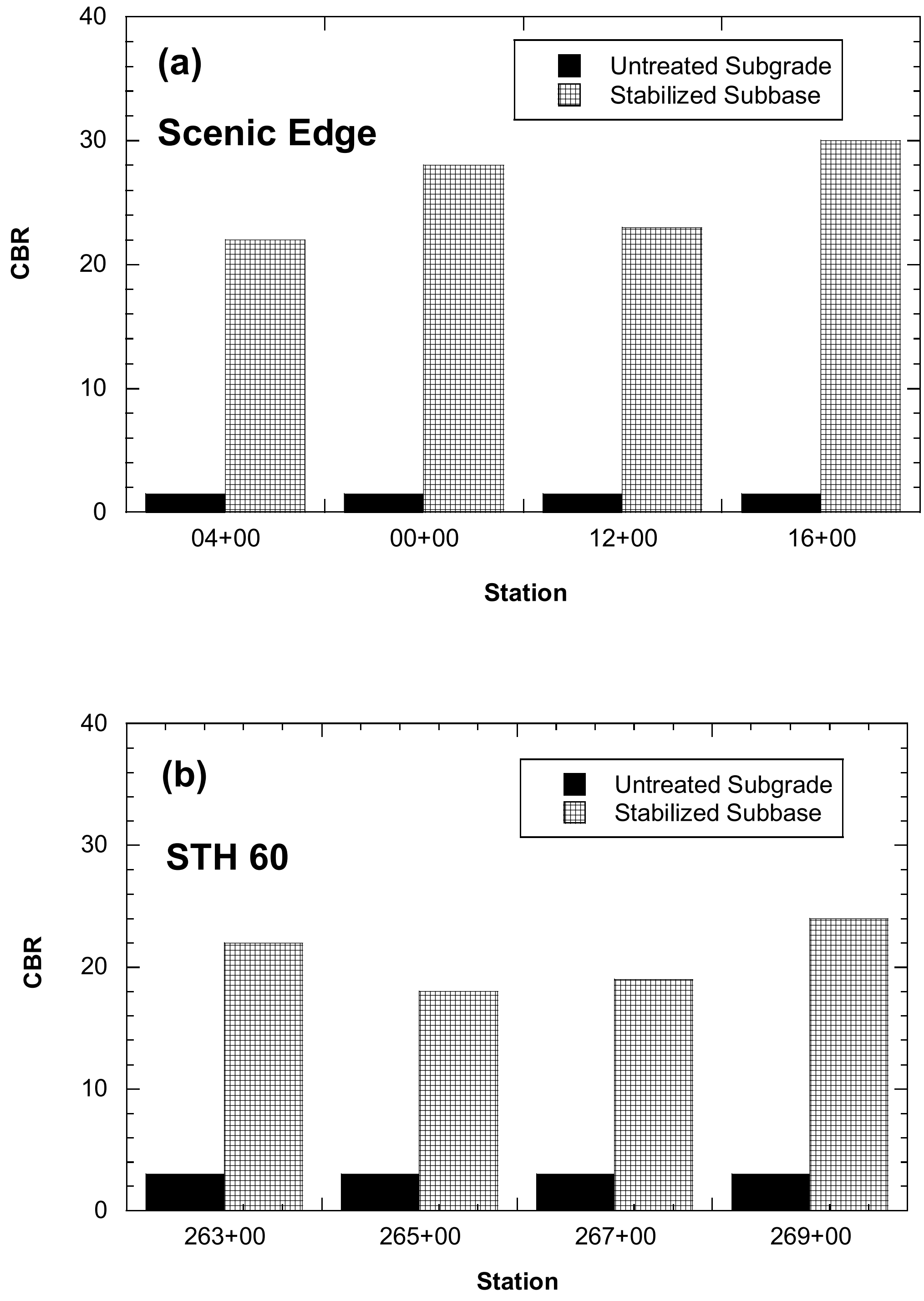
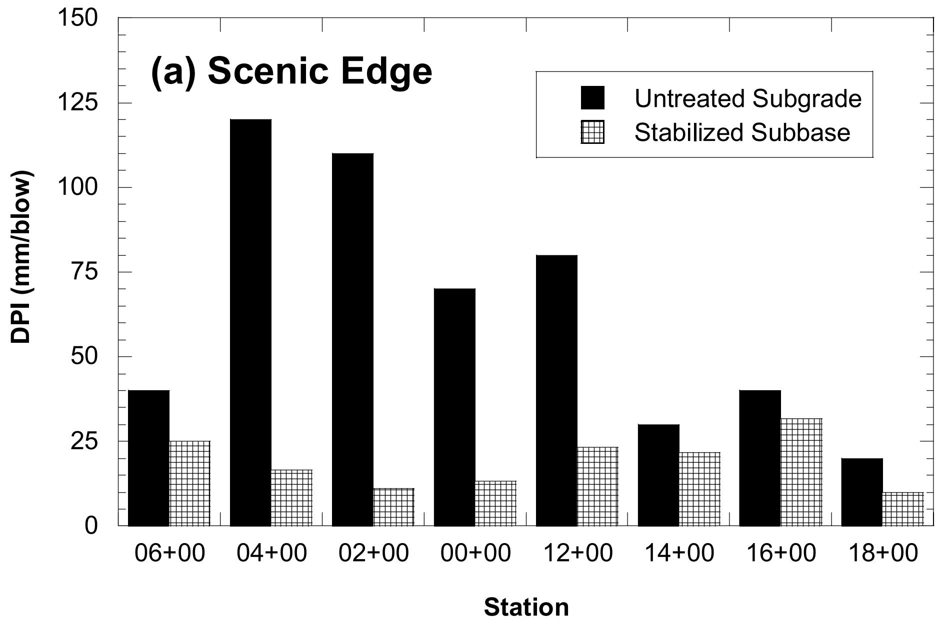


FIG. 8. CBR of Untreated Subgrade and Fly Ash Stabilized Layer Along Center Line of Pavement: (a) Scenic Edge and (b) STH 60.



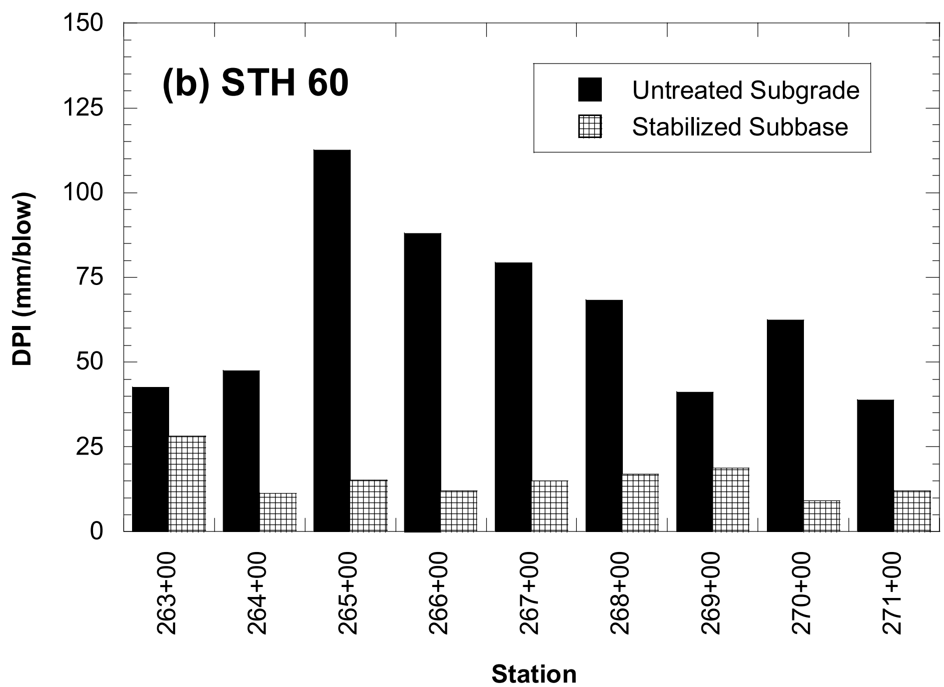
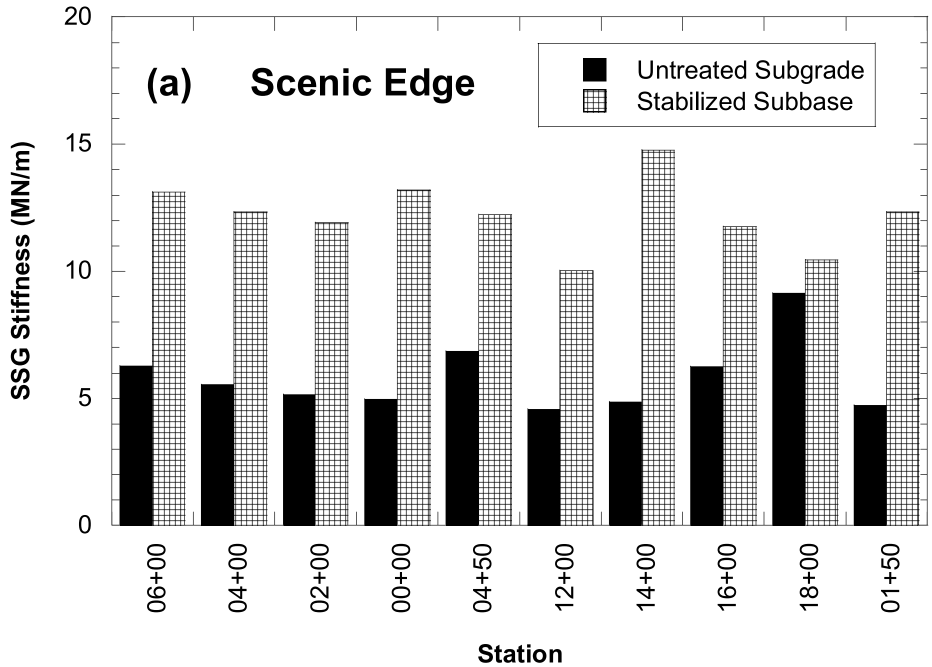


FIG. 9. Dynamic Penetration Index of Untreated Subgrade and Fly Ash Stabilized Layer Measured Along Center Line of Pavement: (a) Scenic Edge and (b) STH 60.



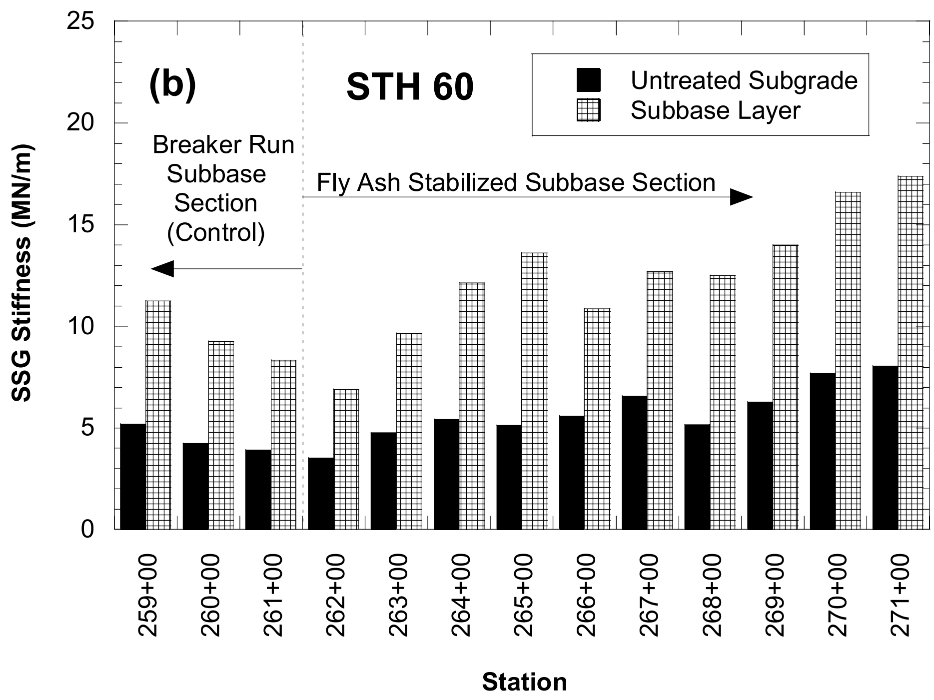


FIG. 10. Stiffness of Untreated Subgrade and Fly Ash Stabilized Layer Measured with SSG Along Center Line of Pavement: (a) Scenic Edge and (b) STH 60.