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The Laconia, New Hampshire Bottom Ash Paving Project

Volume 3: Physical Performance Testing Report

*Environmental Research Group
University of New Hampshire
Durham, New Hampshire*

MASTER

National Renewable Energy Laboratory
1617 Cole Boulevard
Golden, Colorado 80401-3393
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NREL Technical Monitor: Philip Shepherd

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EXECUTIVE SUMMARY

Introduction

This project was conducted to demonstrate the feasibility of using municipal solid waste (MSW) grate ash as an aggregate substitute in the construction of a pavement binder course for a portion of Rt. 3 in Laconia, New Hampshire. The research, sponsored by the Concord Regional Solid Waste/Resource Recovery Cooperative (CRSW/RRC), Wheelabrator Environmental Systems Inc. (WES), the U.S. EPA, and the U.S. DOE, was conducted over a two year period completed during 1993 and 1994. This study is the culmination of an earlier two year characterization study between 1990 and 1992 that documented the physical and environmental characteristics of the bottom ash as it is produced at the Concord, N.H. waste-to-energy (WTE) facility. Together, these two studies provide a complete evaluation of the potential for using grate ash in asphalt binder course or as recycled asphalt pavement (RAP) in asphalt stabilized base courses in pavements.

This study included two principal parts: an environmental evaluation and a physical performance evaluation. The environmental component includes an evaluation of the potential short term and long term impacts associated with the use of grate ash in pavement construction, including possible future life cycle scenarios for the pavement. The final Environmental Testing Report is comprised of two volumes (Eighmy et al., 1996a, and b). Volume 1 addresses environmental performance evaluations and Volume 2 provides appendices containing data. The Physical Performance Testing Report addresses physical performance evaluations and is presented as Volume 3.

This Executive Summary includes sections that provide information about the background of the project, mix design, field placement of the binder mixes, pavement evaluation including: climatic and physical traffic effects, automatic road analyzer (ARAN), and falling weight deflectometer (FWD), laboratory testing of pavement samples, and Marshall properties for the grate ash and control mixes.

Background

The use of grate ash or bottom ash in civil engineering construction is widely practiced in western Europe. Applications include its use as granular fill in wind and sound barriers, road embankments, road subbases, parking lots and bike paths. The uses that are anticipated for the U.S. include the use as aggregate substitutes in bituminous asphalt and their recycled products, portland cement concrete, stabilized road subbase, granular fill, or as granular cover material at lined MSW landfills.

The requirements to establish the use of a waste material as a product in the State of New Hampshire involve defining: (i) the physical and chemical variability of the ash process stream, (ii) a suitable market for the product, and (iii) an evaluation of the performance of the product. The earlier two year characterization study addressed the first two requirements by showing the relative invariability of the chemical and physical properties of the grate ash as it is produced at the Concord waste-to-energy facility, the successful substitution of grate ash for aggregate in the establishment of a pavement job mix formula, the successful manufacture of hot mix asphalt (HMA) at an asphalt batch plant, and identification of the role that asphalt cement plays in controlling diffusional

leaching behavior. This project addresses the third requirement, an evaluation of the performance of the product.

The Laconia paving project effort consisted of the reconstruction of Rt. 3 in Laconia, New Hampshire. A 2,000 linear foot (610 m) section was reconstructed. Approximately 1,150 feet (350 m) was constructed as a control section using natural aggregate both in the binder and wearing courses. The 850 foot (260 m) test section was constructed using 50% grate ash with 50% natural aggregate in the binder course and natural aggregate in the wearing course. Both binder courses were placed on a base stabilized with RAP. This asphalt stabilized base was constructed by blending RAP millings from this and other Laconia projects with crushed gravel brought in for elevation control.

Mix Design

A superior aggregate which has been used in thousands of tons of HMA was selected as a control to compare the performance of the grate ash binder. Two viable mixes meeting the specifications of the New Hampshire Department of Transportation for a Type B binder were developed using a gyratory testing machine (GTM). In the process of developing the mixes, it was determined that conventional mix design methods are not reliable for light weight aggregates like grate ash and using them to design a mix would result in a pavement failing from severe rutting.

The GTM was found to be a reliable mix design method for the grate ash mix (test) as well as the conventional aggregate mix (control). The test mix required an

asphalt content of 7.0 % which was 1.9 % higher than the control at 5.1 % but, 2.1 % less asphalt cement than the 9.1 % predicted by the commonly used Marshall mix design method.

The GTM method relates well to the predicted field performance of HMA mixes and was used to predict the performance of both the test and control mixes. Although both mixes were found to undergo degradation during the laboratory mixing and compaction process, neither is expected to continue under the effects of traffic. The effect of degradation can be negated by using less manufactured fines from the conventional aggregates. The test was found to absorb more asphalt cement than the control due to its higher porosity.

Asphalt Plant Moisture Control

The grate ash was stored in a large pile after it was processed through the ¾ inch sieve and the over-sized material was removed. The pile had a considerable amount of snow on it throughout the winter which slowly melted and helped maintain a relatively high moisture content. The moisture content was approximately 20-25 % upon arrival at the asphalt batch plant. The effect of the high moisture content was compensated for by slowing down the feed rate of the aggregates into the dryer such that the aggregates had more time in the dryer to allow for the removal the extra moisture. Even though the moisture was very high and 50 % of the job mix was grate ash, controlling moisture, mix temperature and plant production was not problematic.

Asphalt Plant Gradation Control

The gradation of the mix was found to be shifted towards the fine side due to the tumbling of the aggregate particles in the dryer drum. To compensate for the expected increase in fines, the unwashed manufactured fines were substituted with an equal volume of grate ash. Although the increase in fines exceeded the amount reduced by eliminating the manufactured fines, this was not felt to be problematic. Under full production the amount of conventional fines could be further reduced to the point the amount of reduced manufactured fines normally added to a mix would equal the amount of fines produced in the drying and mixing process.

The batch plant used to produce the mixes used an auger to transfer the fines created in the drying process and those collected in the air pollution system to the sieves located above the aggregate bins. The gap between the auger and its casing compounded with the lightweight grate ash particles made the transfer very difficult and the small lightweight fines accumulated in the lower part of the auger. The accumulated fines were removed with the assistance of a vacuum truck. A total volume of approximately one cubic meter of fines was collected and disposed of at the landfill during the production of the grate ash mix.

Production of fines while drying and mixing can easily be controlled by adjustment of the natural fines in the job mix. An equal amount of natural fines could be removed so the gradation could be achieved after drying and mixing rather than before the aggregates are dried.

Transfer of the fines within the plant can be accomplished by redesigning the mechanical transfer system or by completely eliminating the need for transferring the fines by the use of a drum mixing plant. By either method, the generation of fines in the production of grate ash HMA is not problematic.

Field Placement and Testing of Binder Mixes

The Route 3 project was successfully rehabilitated with the paving of the test and control mixes. Both mixes were mixed, trucked to the site, spread with a paver and compacted without difficulty. Other than confirming that ash fines were increased during mixing and that the test required several additional passes of the roller to achieve compaction, there was no apparent difference between the two mixes. Compaction quality control was successfully completed using conventional nuclear density equipment.

Testing of the batch plant mixtures showed both mixes to be well within the NHDOT specifications for extracted asphalt contents and comparable to the design requirements of the job mix formula. The aggregate gradation of both the control and test were slightly out of specification. However, neither was considered to be significant as to affect the performance of the mixes. The placement of both the test and control mixes was judged to be typical of any routine HMA paving project.

Pavement Evaluation

Climatic and Physical Effects

The pavement sections were subjected to a very severe winter resulting in a depth of frost in excess of 52 inches (1.32 m). Although the Spring thaw effects were very apparent, both test and control responded equally and neither showed any loss of performance during the critical period.

In addition to the severe environmental conditions the pavements were simultaneously subjected to the physical effect of a significant amount of traffic in excess of 5.5 million vehicles including an estimated 400,000 trucks. Pavement evaluation of the test and control pavements showed the two mixes to be physically indistinguishable from each other.

ARAN

The pavements were evaluated for serviceability by the Data Collection Department of the Bureau of Transportation and Planning of the New Hampshire Department of Transportation. The pavements were found to have higher than normal International Roughness Index (IRI) and therefore the present serviceability index (PSI) was also lower than what is expected for a new pavement. The IRI values may have been rendered suspect due to the waviness of pavement caused by the contractor's selection of a short based rubber tire paver and the difficulty was encountered with the compaction of the Northbound test lane surface mix. This mix was noted to have pushed in front of the roller and was difficult to compact. Such properties are referred to as a

"tender mix". Although visually the pavements appear to be performing well and show no difference between them, the ARAN is sensitive enough to detect waviness and other minor surface roughness. The only section which appeared to be significantly different than the others was the Southbound lane of the test section.

One potential reason for this is due to variable subsurface conditions. A typical section of the north (test) end of the project consists of higher elevations with exposed rock out cropping on the east side with ground water close to the surface. On the west side of the pavement the elevation decreases until it intercepts Paugus Bay. Ground water flows from the higher elevation on the west side to the east towards Paugus Bay. In general the ground water table is very shallow on the test section and the groundwater elevation fluctuates significantly during the spring.

The thickness of the stabilized base was less on the southbound lane compared to the northbound lane and both were less than the control due to additional RAP being placed mostly on the control section. The north end has less development and therefore had less elevation control problems and did not require additional RAP.

The subsurface tar macadam was not present on the north (test) end most probably due to the shallow depth of bed rock.

These differences in the north (test) end and especially the southbound lane are very possibly responsible for the ARAN data differences. The seasonal fluctuation of the ground water into the subgrade, subbase and stabilized base would be expected to lower the strength of the pavement. The thinner stabilized base, and absence of the tar macadam would both be expected to lower strength and increase deflections. It is very

likely the test southbound lane would not be as strong as the other sections and would be expected to have increased FWD deflections and show lower ARAN IRI and PSI data. There was no difference between both directions of the control and the Northbound lane of the test.

FWD

Deflections

Extensive FWD testing was conducted on the pavement sections to evaluate the deflections caused by a standard 18,000 pound axle load. The FWD data showed the deflection profiles to vary from high deflection values in areas of lower binder support (subgrade and or base materials) to low values when over areas of higher binder support. The deflections were approximately equal for the test and control sections throughout the study. The profiles were very much affected by temperature and softness in the subsurface layers during the Spring thaw period.

Binder Elastic Modulus

The elastic moduli of the two binders were evaluated by back calculating the deflection profile data. The values for the test and control were found to be statistically equal for various combinations of the 1993 and 1994 Northbound and Southbound data. The moduli decreased between 1993 and 1994 due to an increase in the stiffness of the asphalt stabilized base which was common to both the test and control.

Layer Coefficients

The AASHTO pavement design layer coefficients for the test and control were shown to be statistically equal for the 1993 data (0.41) and the 1994 data (0.40). The reduction of the coefficients as with the elastic moduli was due to the increased stiffness of the asphalt stabilized base. The layer coefficients for the test and control were less than 0.44, the maximum allowed by AASHTO but, higher than the 0.38 the maximum allowed by the NHDOT. In that the test and control were equal, the recommended layer coefficient for use in New Hampshire is 0.38, the maximum allowed by the NHDOT.

Strain Gauges

Both the test and control binders responded similarly to load and temperature thus showing the elastic properties of the two materials to be approximately equal. Binder course horizontal strain was found to increase with temperature, load, and reduced subgrade support during Spring thaw.

Laboratory Testing of Pavement Samples

Laboratory testing of cores and beams taken from the slabs removed from the test and control sections showed the two binder mixes to be approximately equivalent relative to resilient modulus, indirect tension, thermal stress and Marshall properties.

Resilient Modulus and Indirect Tension

The resilient modulus and indirect tension materials properties of the test and control were found to be approximately equal. The values obtained at a given temperature for both the test and control were reasonable and as would be expected for any high quality HMA.

Thermal Stress

The thermal stress was approximately equal for both the test and the control and for equal aging both had cracking temperatures of approximately -4°F (-20°C). This testing result is more a function of the asphalt cement used than due to the effect of the aggregate.

Marshall Properties

The Marshall Properties of both the test and control batch plant mixes had higher densities and lower air voids than were achieved in the field and laboratory. The field core Marshall properties were consistent with the laboratory GTM and Marshall designs. This suggests the batch plant samples were compacted at very high temperatures and the asphalt cement did not have adequate time to age as was allowed during the laboratory mix design and in the field due to transportation time from batch plant to paving site. The test and control binders have not achieved their final GTM air void and density properties after two years of traffic.

Summary

The results show that grate ash can be successfully used as an aggregate substitute in HMA when the GTM method is employed as the mix design method. The Marshall and GTM procedures developed similar mix designs for the control aggregate. However, a significant difference was found between the two design methods for the grate ash mix. The two mixes were successfully installed in a demonstration project located at Laconia, NH during May of 1993.

Preliminary results show the grate ash section performs as well as the control section in all aspects.

In summary, the use of grate ash for conventional aggregate up to 50% substitution ratio in binder course applications is a suitable and viable waste utilization concept. The use of grate ash for aggregate substitution in HMA is recommended for use.

CHAPTER 1. INTRODUCTION

Introduction

This is the third volume of a report titled "The Laconia, N.H. Bottom Ash Paving Project". This volume addresses physical performance and evaluation regarding the project. It contains information relative to pavement design and testing procedures, laboratory mix evaluation, instrumentation of test and control pavements, the paving of the test road, data collection, laboratory testing of field samples and pavement performance evaluation. Companion volumes 1 and 2 (Eighmy et al., 1996a,b) contain information relative to grate ash chemical characterization, evaluation of micro-scale and macro-scale diffusion in pavement materials, evaluation of monolithic diffusional leaching behavior of laboratory and road specimens, modeling of contaminant release behavior, and evaluation of ground water and surface water quality data from the study site. *

This research was sponsored by the Concord Regional Solid Waste/Resource Recovery Cooperative (CRSW/RRC), Wheelabrator Environmental Systems (WES), and the City of Laconia, N.H. Numerous state, federal, and foreign governmental agencies have participated in this project as observers who received progress reports for comment and review. The research was conducted by the Environmental Research Group (ERG) of the University of New Hampshire and the Cold Regions Research and Engineering Laboratory (CRREL) of the U.S. Army Corps of Engineers (U.S. ACOE).

Background

The CRSW/RRC is a consortium of 27 New Hampshire municipalities, including the City of Laconia. The cooperative presently sends its municipal solid waste (MSW) to the Concord, N.H. waste-to-energy (WTE) facility located in the village of Penacook. The facility is owned and operated by WES. Combined bottom ash and scrubber residues are presently disposed in a double-lined ash monofill in Franklin, N.H. The CRSW/RRC owns the landfill and is responsible for its operation.

The CRSW/RRC was interested in securing longer term disposal capacity for the residue that it must manage. The extension of the operational life of the landfill via bottom ash separation and utilization was seen to be an interesting opportunity, hence the CRSW/RRC has actively evaluated bottom ash utilization. WES, as owners or operators of over 16 WTE facilities nation-wide, was also interested in evaluating ash utilization.

In 1990, CRSW/RRC and WES embarked on a two year characterization study to carefully evaluate: (i) the time-dependent chemical and physical properties of the bottom ash waste stream generated at the Concord waste-to-energy facility and (ii) the proper asphalt mix design needed to make a high quality pavement. This study, conducted by ERG, resulted in a report (Eighmy et al., 1992), numerous graduate student theses and dissertations (Whitehead, 1992; Tarr, 1993; Zhang, 1993), and numerous scientific papers (Gress et al., 1991; Gress et al., 1992; Whitehead et al., 1993; Tarr et al., 1994; Zhang et al., 1994) addressing the suitability of municipal waste combustor bottom ash as an aggregate substitute in the manufacture of asphaltic pavement material.

Near the end of the two year characterization project, plans were made to identify a suitable location within the 27 communities of the CRSW/RRC to pave a test road with grate ash used as an aggregate substitute in a binder course. A number of criteria were established for site selection. These included the desire (i) to pave a road which was otherwise scheduled for reconstruction and which carried a high volume of traffic, (ii) to construct a pavement with excellent physical performance capabilities, (iii) to construct a pavement with environmentally protective characteristics, and (iv) to design a project addressing all New Hampshire Department of Environmental Services (N.H. DES) regulatory requirements for ash utilization.

The selection of a site in Laconia, New Hampshire consisted of a process of numerous public meetings, publicity in the press, and meetings with abutters. A request was submitted to the New Hampshire Department of Environmental Services, using an R&D permitting process, to conduct the demonstration. Approval was obtained on November 6, 1992.

In May, 1993, a 2,000 linear foot (610 m) section of US Route 3 was reconstructed in Laconia, N.H. Approximately 1,150 feet (350 m) was constructed as a control section using natural aggregate in the binder course. The 850 foot (260 m) test section was constructed using a 50 percent grate ash and 50 percent natural aggregate blend in the binder course. A wearing course, using 100 percent natural aggregate, was applied over both sections. Therefore, in this demonstration, the test binder course was not directly exposed to traffic or the environment. Route 3 was originally the main highway between Boston and Montreal and was designed and constructed in the 1930's to primary road standards. The original road was consisted of a variable thickness portland cement concrete pavement. It was resurfaced with an asphalt wearing course

sometime in the past. During the 1993 reconstruction, the existing asphalt wearing course was removed from the road by milling and stockpiled off-site for later use as a stabilized asphalt base course. The underlying concrete road surface was broken up, removed from the site and stockpiled for future use on Laconia construction projects.

From October, 1992 through December, 1994, data have been gathered on the environmental and physical performance of the road. This has involved extensive field and laboratory testing. This report and the two volume environmental testing reports summarize the project information and provide documentation required to obtain an approval for direct reuse pursuant to EnV-Wm 2602.06 from N.H. DES. The approval process will involve the modification of the Concord facility's residuals management plan.

Bottom Ash Utilization World-Wide

A modern, mass burn waste-to-energy (WTE) facility is comprised of a tipping floor (or pit), a crane to load MSW to the combustor, MSW feed hoppers, a furnace with grates, a boiler, heat exchanger, air pollution control system, and stack. Figure 1.1 depicts a typical layout of a WTE facility.

Figure 1.2 depicts a materials balance in a combustion facility. In a modern mass-burn municipal WTE facility, about 80 to 95 percent of the total weight of the residues that are produced are bottom ashes (IAWG, 1994). Bottom ash is the grate ashes and grate siftings that remain on the grate (or sift through the grate) during the combustion process. Typically, bottom ash is comprised of 95 to 97 percent by weight of grate ash and 3 to 5 percent grate siftings. The two materials contain slags, unmelted or partially melted metallic materials, and uncombusted MSW. Frequently, economizer ash from the boiler section of the furnace is added to the bottom ash; though this represents a small

1. Receiving Hall/Tipping Floor
2. MSW Bunker/Pit
3. Grapple Crane
4. Charging Hopper
5. Reciprocating Grates With Underfire Air (Grate Ash and Grate Siftings)
6. Combustion Zone With Overfire Air
7. Superheater
8. Boiler and Economizer (Economizer Ash)
9. Ram Discharge With Quench (Grate Ash, Grate Siftings, Economizer Ash)
10. Dry Lime Scrubber Venturi
11. Fabric Filters (Dry Scrubber Residue)
12. Induced Draft Fan
13. Stack .
14. Combined Ash Roll-Off

Figure 1.1: Schematic of a Typical WTE Facility

Figure 1.2: Mass Balance of Residuals from a WTE Facility (from IAWG, 1994)

fraction of the bottom ash (<1 percent). In the earlier two year characterization study, bottom ash was characterized. In this study, only grate ash was used in the demonstration. These distinctions are maintained throughout the report.

The bottom ash is about 60-80 percent by weight of aggregate-size slag particles. The intended use of the particles and the required size [e.g. < 3/4 inches (1.9 cm)] influences the weight fraction that can be utilized. Traditionally, in western Europe, these residues are utilized as aggregate substitutes in granular fill, road bases, wind : barriers, sound barriers, and portland cement and bituminous asphalt pavements (IAWG, 1994).

A significant amount of research has been conducted on the process of slag formation and the petrology of the grate ash slags (Eusden et al., 1994; Eusden and Eighmy, 1995). Grate ash slag particles consist of non-melted or partially melted waste products (30-45 percent) and melt products (55 to 70 percent). The non-melted waste products include bottle glass, metallic material, soil particles, and uncombusted waste. The melt products consist of glasses and crystalline minerals. The glasses are divided into three types: isotropic silicate, schlieren, and opaque metallic glasses. The crystalline minerals formed in the combustor consist of two major groups: complex silicate and complex oxide minerals. The complex silicate minerals include melilite-like ($\text{Ca}_2\text{Al}_2\text{SiO}_7$) and akermanite-like ($\text{Ca}_2(\text{Mg},\text{Fe}^{2+})\text{Si}_2\text{O}_7$) minerals. The complex oxides include iron oxides (Fe_3O_4) and lime (CaO). The texture and mineralogy of the isotropic and schlieren glasses and silicate minerals is similar to alkaline volcanic rock materials subjected to fractional crystallization at a melt temperature of about 2,200°F (1,200°C). Trapped gas vesicles, which are sometimes expelled from the melt as it cools, give the slag particles a high degree of porosity; making the grate ash a light weight aggregate.

As shown in Table 1.1, grate ash and bottom ash are widely used as a granular fill or road base material in the Netherlands, Denmark, Switzerland, and the western portion of Germany. To a lesser extent, bottom ash is utilized in Japan after it is vitrified and converted into a fine-grain frit (Sakai and Hiraoka, 1994). The frit is used in various applications, largely as a fine aggregate in road construction applications.

In many of the European jurisdictions, grate ash is used after the material has been processed and aged. Aging reactions are conducted by maintaining a moist residue usually after trommeling or sieving to remove oversized material. The presence of moisture allows for a variety of aging reactions to occur (IAWG, 1994). These include (i) CO_2 uptake into the alkaline residue coupled with metal carbonate formation, (ii) elemental metal oxidation and water reduction resulting in H_2 evolution, (iii) metal oxide hydrolysis reactions, (iv) slag weathering to clays, (v) simple salt dissolution and solid phase respeciation and (vi) organic carbon biodegradation. In some jurisdictions, grate siftings are removed from the bottom ash primarily to minimize the potential for H_2 generation resulting from the oxidation of elemental metals found in the siftings. Proper aging of the ash eliminates or controls all of the above potentially detrimental characteristics.

A number of recent studies have examined the environmental impact of MSW bottom ash use as road subbase or fill material. These include test roads in Sweden (Hartlén and Fallman, 1991), structural fill embankments in the Netherlands (DHV Milieu en Infrastructure BV, 1992a,b,c), large scale road bases constructed in lysimeters in the Netherlands (Mulder and Gerritsen, 1990; Mulder, 1991), and historical evaluation of a ten year old road subbase constructed with bottom ash in the Netherlands (van der Sloot and Hoede, 1994a,b). The Swedish study (Hartlén and Fallman, 1990) involved

Table 1.1: Grate Ash/Bottom Ash Utilization in Europe and Japan

Country	MSW Generated (10 ⁶ tonnes/yr) ^a	MSW Combusted (10 ⁶ tonnes/yr)	Ash Produced (10 ⁶ tonnes/yr)	Percent Used	Application	Reference
Netherlands	8.57	3.00	0.75	60	Subbase, embankments	Van der Sloot, 1991
Denmark ^b	5.74	1.80	0.43	72	Subbase, embankments, land reclamation	Hjelmar, 1991
France		8.40	2.52	64	Road base	
Western Germany	22.30	7.36	3.00	50	Subbase, embankments	Vehlow, 1991
Switzerland	3.70	2.30	0.60	>50 ^c	Subbase, embankments	Stämpfli, 1991
Sweden	2.50	1.25	0.40	75	Subbase, embankments	Hartlen, 1990
Japan	42.96	30.86	-	5	Civil construction ^d	Sakai and Hiraoka, 1994

^a Some caution should be used in interpreting these numbers as definitions of MSW vary between countries

^b The data for Denmark include the use of oversize ferrous in the listed percentage, the other countries do not specify if ferrous recycling is included in the percentage.

^c Reflects anticipated usage, not presently used.

^d Ash used after vitrification.

using bottom ash as a granular base course material. The study has shown that after perforation of the asphaltic wearing course to allow infiltration, leachates collected from rain water contacted with the bottom ash were similar to bottom ash leachates from column leaching studies at low liquid to solid (L/S) ratios. The leachates contained dissolved salts (Na, Cl, SO_4^{2-} , K) and some low concentrations of metals (Ca, Cu, Mg, Mn, Pb, Zn). Similar data were seen by Mulder and coworkers in Dutch lysimeter studies where bottom ash was placed below standard pavement materials and road runoff was allowed to infiltrate the ash base courses (Mulder and Gerritsen, 1990; Mulder, 1991). The Highway 15 study in the Netherlands and the lysimeter study in the Netherlands (DHV Milieu en Infrastructure BV, 1992a,b,c) have shown that bottom ash use as granular fill in bridge approaches also produces typical bottom ash leachates. The historical analysis of the use of compacted bottom ash as a base course for the service road in Rotterdam, the Netherlands (van der Sloot and Hoede, 1994a,b) has shown that accumulation of metals in the underlying soils over a ten year period takes place in the first few centimeters of soil below the pile, demonstrating significant attenuation when ashes directly overlie soils and leachates migrate downwards into the soil.

The use of bottom or grate ash as an aggregate substitute in bituminous asphaltic materials has been more prevalent in North America. Fly ash from electrostatic precipitators (ESP) has also been used as a mineral filler in asphaltic materials in the Netherlands (IAWG, 1994) and some fundamental leaching studies on the behavior of metals from ESP ash in asphalt monoliths have been conducted (de Groot et al., 1990). As shown in Table 1.2, a number of demonstrations on the use of bottom ash in pavement materials were conducted in the 1970's and early 1980's. Generally, these

Table 1.2: Earlier Bottom Ash Use Studies in Bituminous Applications in the United States

Project	Date	Residue %	Asphalt %	Lime %	Length m	Thickness cm	Performance	(Reference)
Houston, TX	1974	100 ^a	9.0	2.0	60	15 BC ^c	Excellent	(e,f,g)
Philadelphia, PA	1975	50 ^a	7.4	2.5	30	WC ^d	Acceptable	(h)
Delaware Co., PA	1975	50 ^a	7.0	2.5	20	3.8 WC	Acceptable	(h)
Harrisburg, PA	1975	50 ^a	7.0	2.5	80	3.8 WC	Poor	(h)
Washington, DC	1977	70 ^a	9.0	2.0	130	11.3 BC	Good	(i)
Lynn, MA	1979	50 ^a	6.5	2.0	1500	3.8 WC	Excellent	(j)
Harrisburg, PA	1976	100 ^a	6.7	0.0	60	3.8 WC	Excellent	(k)
Tampa, FL	1987	15 ^b	7.0	-	400	2.5 WC	Excellent	(l)

^a Largely bottom ash; possibly some economizer ashes.

^b Bottom ash and fly ash.

^c BC = base course.

^d WC = wearing course.

^e Haynes and Ledbetter, 1976.

^f Teague and Ledbetter, 1978.

^g Ederly and Ledbetter, 1981.

^h Collins et al., 1977.

ⁱ Pavlovich et al., 1979.

^j Bastanza, 1982.

^k Snyder, 1980.

^l Mehan and Hooper, 1991.

studies utilized residues from facilities that are now considered to be low technology. They focused on the use of bottom ash as an aggregate substitute in asphaltic mixtures with emphasis particularly on the physical properties of the materials and not on the environmental performance of the materials. These earlier studies, do, however, differ from European directions in that the ash was used in an encapsulated form. Such encapsulated materials are considered more likely for utilization scenarios in the U.S. The use of ash as granular material in landfill cover within the footprint of lined landfills or as a subbase below paved roads is also being considered in the U.S.

Several studies have looked at using fused (Snyder, 1980) or treated (Mehan and Hooper, 1991) bottom ashes as aggregate substitutes. Work by Chesner et al. (1988) has shown that bottom ash from the Southwest Brooklyn, New York combustor can be used as an aggregate substitute. They found a 30 percent bottom ash substitution to physically perform as well as controls in batch Marshall testing for pavement design. Recent evaluations of bottom ash use by Chesner (1991) and by the Long Island Regional Planning Board (1994) have examined economic, regulatory, and environmental issues associated with the use of bottom ash. It appears that institutional issues may be the largest impediment to bottom ash use in the United States despite the fact that its use is technically feasible.

The Laconia, N.H. Bottom Ash Paving Project

The Laconia, N.H. Bottom Ash Paving Project has involved over five years of effort. This effort has consisted of two major parts. The first part was an earlier two year characterization study involving the careful evaluation of the time-dependent physical and chemical properties of the bottom ash process stream as it is produced at

the Concord WTE facility, the design of a suitable pavement, and the fundamental evaluation of environmental leaching behavior of the bottom ash and bottom ash/asphalt materials. The second part is this demonstration project.

Earlier Two Year Characterization Study

One of the principal objectives of the earlier research was to design a systematic and comprehensive sampling program to characterize the bottom ash process stream as it was generated at the Concord plant. This effort was a quality assurance (QA) program to confirm that the bottom ash has known properties with characterized bounds of variability. Such assurance is needed if the process stream is to replace a natural material in the views of regulators, end-users, and transportation engineers.

As shown in Table 1.3, 18 different sampling events were conducted from October, 1990 through April, 1992. The sampling dates were distributed on the days of the week as follows: Monday, 3; Tuesday, 4; Wednesday, 4; Thursday, 3; Friday, 4. Sampling was not conducted on Saturday or Sunday because of staffing requirements at the plant. The sampling data had the following monthly distribution: January, 1; March 2; April, 2; June, 1; August, 1; October, 5; November, 4; December 2. This was done to provide a balanced distribution between months, seasons, and times when incoming MSW to the plant was intermittently wet and more difficult to burn. All sampling was conducted over a four to five hour consecutive period usually beginning on the morning of the sampling day.

Figure 1.3 depicts the sampling protocol that was used for each of the sampling visits. The four hour period was separated into four consecutive hourly sampling events.

Table 1.3: Sampling Events for the Earlier Two Year Characterization Study

Sampling Event Number	Date	Day of Week	Time of Sampling	Combustor Unit Sampled	Total Sample Collected lbs (kg)
1	10/3/90	Wed.	1045-1450	1	1,510 (685)
2	10/11/90	Thur.	0935-1345	1&2	1,051 (477)
3	10/19/90	Fri.	1150-1550	1	1,080 (490)
4	10/29/90	Mon.	0945-1415	2	1,058 (480)
5	11/5/90	Mon.	0920-1320	1	705 (321)
6	11/29/90	Thur.	1000-1400	2	813 (369)
7	12/7/90	Fri.	1000-1400	2	887 (403)
8	1/29/91	Tues.	0930-1330	2	1,045 (474)
9	3/26/91	Tues.	1130-1600	2	937 (425)
10	4/24/91	Wed.	0900-1300	2	939 (426)
11	6/17/91	Mon.	1045-1445	2	956 (434)
12	8/2/91	Fri.	1430-1730	2	1,280 (581)
13	10/1/01	Tues.	0930-1410	1	1,006 (456)
14	11/5/91	Tues.	0930-1340	2	1,039 (472)
15	11/20/91	Wed.	1100-1400	2	1,047 (475)
16	12/18/91	Wed.	1120-1520	2	1,050 (477)
17	3/20/91	Fri.	0840-1240	2	1,750 (794)
18	4/2/92	Thur.	0915-1305	2	810 (367)

Figure 1.3: Sampling Protocol for the Earlier Two Year Characterization Study

For each hourly sampling event, plant personnel randomly collected 40 to 60 lbs (18 to 27 kg) of bottom ash off the drag chain conveyor during the first 3 to 4 minutes of a ten minute period. This was repeated for the second, third, fourth, fifth, and sixth ten minute periods of the hour. A spade was used to randomly collect material off the flights as they moved past the sampling station. Objects larger than the spade were not collected.

For each of the six increments during the hour, the material was sieved as received through a 3/4 inch (1.9 cm) sieve over a large barrel, producing a stream passing the sieve and a stream being rejected from the sieve. At the end of the hour, 240 to 360 lbs (109 to 163 kg) were sieved, producing approximately 200 lbs (91 kg) of wet bottom ash smaller than 3/4 inches (1.9 cm) which were made into an hourly composite and approximately 150 lbs (68 kg) of rejects. Scales were used to weigh both the passed and rejected material. In effect, approximately four percent of the bottom ash process stream from the combustor was collected each hour for sieving.

This process was repeated again for the second, third, and fourth hours. Table 1.3 describes the total amount of wet <3/4 inches (<1.9 cm) bottom ash that was collected for each of the eighteen days. For each sampling event, combustor operation was monitored. This included evaluation of combustion temperature, air preheat temperature, over and underfire airflow; economizer outlet temperature, and economizer outlet opacity, CO, and O₂.

As indicated in Figure 1.3, the four hourly composites were combined to generate a daily composite. Approximately 30 lbs of wet bottom ash (13.6 kg) were collected from the four to five sample buckets from each hourly composite, added to a concrete mixer, and blended to make a daily composite containing approximately 120 lbs (54.5 kg)

of bottom ash. Large subsamples of the hourly and daily composites were collected and dried in a large drying oven for subsequent analysis. The remaining hourly and daily composites were stored wet as archived samples that were susceptible to aging processes.

Table 1.4 describes the physical tests that were conducted on the bottom ash collected during the 18 sampling events or on the bottom ash used in the test paving work. The methods were selected to: (i) conform to testing conducted in previous studies (e.g. Chesner et al., 1988; VEABRIN, 1988), (ii) develop a data base to justify the substitution of bottom ash for natural aggregate in asphalt paving from a physical performance perspective, and (iii) help develop a data base to support the use of bottom ash as an aggregate substitute in other applications (e.g. granular fill, portland cement concrete, stabilized bases, and granular road subbases). Table 1.4 briefly explains the purpose of each method.

Table 1.5 provides the ASTM reference method for each analysis and also describes the frequency of analysis. The majority of the methods were conducted on each hourly composite (72 analyses) and each daily composite (18 analyses). Some of the more intensive analyses were conducted on selected daily composites. Some level of replication was employed with each method to verify precision and accuracy.

Table 1.6 summarizes the mean values and ranges for all the physical testing parameters that were conducted in the earlier study. The data show (Eighmy et al., 1992; Tarr, 1993; Zhang et al., 1993) the time dependent physical property variability of the bottom ash over a two year period. While these variabilities are slightly larger than what is encountered with natural aggregates that are mined or produced from crushing operations, their general characteristics are quite good as a lightweight aggregate substitute (Eighmy et al., 1994).

**Table 1.4: Physical Testing Requirements for the Earlier
Two Year Characterization Study**

Parameter	Purpose
1. Mass <3/4" (1.9 cm)	- Measure of Material Suitable for Base or Binder Course
2. Moisture Content	- Measure of Water Content for Handling, Drying
3. Bulk Specific Gravity, Fine	- Mix Design, Weight - Volume Conversions
4. Bulk Specific Gravity, Coarse	- Mix Design, Weight - Volume Conversions
5. Bulk SSD ^a Specific Gravity, Fine	- Mix Design, Weight - Volume Conversions
6. Bulk SSD Specific Gravity, Coarse	- Mix Design, Weight - Volume Conversions
7. Apparent Specific Gravity, Fine	- Mix Design, Unit Weight
8. Apparent Specific Gravity, Coarse	- Mix Design, Unit Weight
9. Absorption, Fine	- Tendency to Absorb Asphalt
10. Absorption, Coarse	- Tendency to Absorb Asphalt
11. Loss on Ignition	- Tendency to Absorb Asphalt
12. Ferrous Content	- Measure of Metallic Content, Corrodible Material
13. Passing #4 Sieve	- Grain Size Distribution
14. Passing #200 Sieve	- Grain Size Distribution
15. Uniformity Coefficient	- Cohesive Soil Property
16. Effective Size	- Cohesive Soil Property
17. Los Angeles Abrasion (Grading B)	- Material Strength, Friability
18. Los Angeles Abrasion (Grading C)	- Material Strength, Friability
19. Soundness, Fine	- Material Durability
20. Soundness, Coarse	- Material Durability
21. California Bearing Ratio @ 0.1 Inch	- Material Strength
22. California Bearing Ratio @ 0.2 Inch	- Material Strength
23. Optimum Proctor Moisture	- Material Compactability
24. Maximum Proctor Density	- Material Compactability
25. Unit Weight	- Compacted Volume-Density Relationships
26. Marshall Stability	- Asphalt Paving

^aSaturated, surface dry.

**Table 1.5: Physical Testing Methods and Frequencies for
the Earlier Two Year Characterization Study**

	Parameter	Method	Frequency ^a
1.	Mass < 3/4"	3/4" Wet Sieve	H
2.	Moisture Content	ASTM D2216	H
3.	Bulk SG, Fine	ASTM C128	H,D
4.	Bulk SG, Coarse	ASTM C128	H,D
5.	Bulk SSD ^b SG, Fine	ASTM C128	H,D
6.	Bulk SSD SG, Coarse	ASTM C127	H,D
7.	Apparent SG, Fine	ASTM C128	H,D
8.	Apparent SG, Coarse	ASTM C127	H,D
9.	Absorption, Coarse	ASTM C128	H,D
10.	Absorption, Coarse	ASTM C127	H,D
11.	Loss on Ignition	ASTM C114	H,D
12.	Ferrous Content	Hand Magnet	H,D
13.	Passing #4 Sieve	ASTM C136	H,D
14.	Passing #200 Sieve	ASTM C136	H,D
15.	Uniformity Coefficient	ASTM C136	H,D
16.	Effective Size	ASTM C136	H,D
17.	Los Angeles Abrasion Grading B	ASTM C131	S
18.	Los Angeles Abrasion Grading C	ASTM C131	S
19.	Soundness, Fine	ASTM C88	S
20.	Soundness, Coarse	ASTM C88	S
21.	California Bearing Ratio @ 0.1 inch	ASTM D1883	D
22.	California Bearing Ratio @ 0.2 inch	ASTM D1883	D
23.	Optimum Proctor Moisture	ASTM D1557	D
24.	Maximum Proctor Density	ASTM D1557	D
25.	Unit Weight	ASTM C29	D
26.	Marshall Stability	ASTM D1559	D

^aH - every hourly composite, D = every daily composite, S = selected daily composites.

^bSaturated, surface dry.

Table 1.6: Upper and Lower Limits of Physical Properties of Bottom Ash
From the Earlier Two Year Characterization Study

Property	Mean	Range
Mass Rejects, %	32.9	20.8 - 50.1
Mass < 3/4", %	67.01	49.8 - 79.1
Water Content, %	37.8	22.3 - 60.5
Uniform Coefficient	21.7	8.3 - 34.0
Effective Size	0.29	0.11 - 0.52
BSG ^a , Fine	1.86	1.55 - 2.22
BSG, Coarse	2.19	1.92 - 2.43
BSG SSD ^b , Fine	2.13	1.893 - 2.40
BSG SSD, Coarse	2.32	2.11 - 2.58
Apparent SG ^c , Fine	2.56	2.20 - 2.98
Apparent SG, Coarse	2.51	2.31 - 2.76
Absorption, Fine	14.7	6.81 - 24.7
Absorption, Coarse	5.69	1.79 - 12.2
Loss on Ignition, %	6.36	3.16 - 9.99
Iron Content, %	25.8	11.8 - 39.6

^aBulk specific gravity

^bSaturated, surface dry

^cSpecific gravity

Pavement mix designs involving the Marshall Test Procedure and the Gyratory Test Method (GTM) were used to evaluate suitable mix designs. The GTM provided for an excellent mix design when compared to the Marshall method. At a 50 percent substitution of bottom ash for natural aggregate, a mix with excellent strength and resistance to flow was produced. The GTM was used in the actual design of the binder course for the paving demonstration because it more closely approximates the compactive effort of rollers and traffic.

Table 1.7 describes the environmental tests that were conducted on the bottom ash produced during the 18 sampling events or on the bottom ash used for conducting comprehensive environmental leaching studies. As with the physical testing, the environmental testing was conducted to: (i) conform to data from other studies (e.g. de Groot et al., 1990), (ii) develop a data base to document the effects of the substitution of bottom ash for natural aggregate in asphalt paving from an environmental perspective, and (iii) develop a comprehensive leaching study on the leaching of metals and anions from bottom ash and bottom ash/asphalt materials. Table 1.7 briefly describes the purpose of each analysis.

Table 1.7 also provides the appropriate reference for each of the tested environmental parameters. It also identifies the frequency with which these parameters were applied to bottom ash samples. The majority of the parameters were applied to all of the ash samples that were collected.

Table 1.8 summarizes the mean values for all the elemental total concentrations in the ash samples that were collected during the earlier characterization study. The values that are shown, and their ranges, can be compared to values for typical soils. As can be seen, all of the major constituents in the bottom ash (Si, Fe, Al, Ca, etc.) are

Table 1.7: Environmental Testing Requirements and Frequencies for the Earlier Two Year Characterization Study

Parameter	Purpose	Method	Frequency ^a
1. % Ground to < 300 μ m	- Sample Grinding Effectiveness	UNH Method	SH,D
2. Acid Neutralizing Capacity	- Bottom Ash Buffer Capacity	Environment Canada, 1990	SH,D
3. Total Elemental Composition	- Elemental Composition	NAA	SH,D
4. Total Available Composition	- Elemental Composition Available for Leaching	Dutch NEN 7341	SH,D
5. pH-Dependent Leaching	- Influence of pH on Leaching	UNH Method	SD
6. TCLP Extraction	- Regulatory Leaching Test	EPA	SD
7. Method 3050 Digestion	- Standard Digestion Method	EPA 3050	SD
8. Batch Leach Test (Hot Mix)	- Granular Leaching After Size-Reduction	UNH Method	SD
9. Field Lysimeter (Bottom Ash)	- Field Leaching	UNH Method	SD
10. Field Lysimeter (Ash/ Aggregate/Asphalt Pavement)	- Field Leaching	UNH Method	SD
11. Monolith Leaching Test	- Laboratory Monolith Leaching	Dutch NEN 7345	SD

^aD = daily composite, SH = selected hourly composite, SD = selected daily composite.

Table 1.8: Upper and Lower Limits for Bottom Ash from the Earlier Characterization Study and Soil Elemental Composition (mg/kg)

	Bottom Ash ^a		Soils ^b	
	Range	Average	Range	Average
Al	34,400-64,800	52,530	10,000-300,000	71,000
Ca	51,200-102,900	78,275	7,000-500,000	13,700
Fe	56,400-114,950	82,940	7,000-550,000	38,000
Si	182,500-274,000	233,000	230,000-350,000	320,000
Na	20,200-48,000	37,200	750-7,500	6,300
K	7,200-11,600	9,330	400-30,000	8,300
Ba	785-1,480	978	100-3,000	430
Mg	1,900-10,700	7,555	600-6,000	5,000
Cl	1,105-4,190	2,289	20-900	100
Br	16.65-126	45.5	1-10	5
Mn	605-1,560	1,033	20-3,000	600
Ti	3,490-9,500	7,085	1,000-10,000	4,000
Sr	0.0-1,000	581	50-1,000	200
As	17.9-189	42.0	1-50	5
Sb	60-283	118.0	-	-
V	31.6-123	57.5	20-500	100
Mo	20-161	45.4	0.2-5	2
Co	21-43	29	1-40	8
Cr	867-2,005	1,419	2-100	100
Cu	1,140-6,980	2,414	2-100	30
Ni	249-855	610	5-500	40
Zn	2,198-7,640	3,500	10-300	50
Pb	1,200-6,600	2,855	2-200	10
Cd	2.62-70.6	18	0.01-0.70	0.06
Hg	0.0-2.25	0.95	0.01-0.30	0.03

^aData based on 18 daily composites collected to date.

^bData based on compilation by Lindsay (1979) for a wide variety of soils.

similar in composition to soil minerals. Trace elements like Cr, Cu, Zn, Pb, Cd and Hg are enriched.

The reader may refer to Eighmy et al. (1992), Whitehead et al. (1993), Whitehead (1992), Tarr (1992), and Zhang (1993) for a more thorough evaluation of the data from the earlier two year characterization study.

Present Study

The overall goal of the present study was to document that processed grate ash can be used in paving applications as an acceptable substitute for natural aggregates from both a physical and an environmental performance standpoint. Specific demonstration goals included: (i) successful utilization of 50 percent aggregate substitution of grate ash in an asphalt binder course; (ii) development of American Association of State Highway and Transportation Officials (AASHTO) pavement design layer coefficients for the asphalt binder course; (iii) evaluation of the environmental performance of the pavement; and (iv) the development of a data set for field performance to allow for a request for a modification of the Concord WTE facility's residual management plan for bottom ash utilization in binder course and subsequently in RAP base course.

Project Chronology

An abbreviated project chronology, starting in 1990 and ending in 1995 is provided in Table 1.9. Major chronological events are shown. The chronology shows the earlier two year characterization study, the site selection process, the public and regulatory approval process, and salient dates for the present study.

Table 1.9: The Chronology of the Laconia, N.H. Bottom Ash Paving Project
(1990-1995)

Date	Event
April, 1990	- Discussions Amongst CRSW/RRC, WES and UNH-ERG About Ash Utilization
August, 1990	- Earlier Two Year Study Initiated
October, 1990	- First Test Batch at Franklin, N.H. Monofill
April, 1991	- Discussion with N.H. DES About Demonstration Project
June, 1991	- Letter to CRSW/RRC Members Requesting Candidate Roads
August, 1991	- Meeting in Laconia, N.H. to Visit U.S. Route 3
September, 1992	- Meeting in Laconia, N.H. with N.H. DES About Route 3
March, 1992	- Meeting with N.H. DES Regarding Site Hydrogeology, Permit Issues
April, 1992	- Letter to Route 3 Abutters - First Neighborhood Abutters Meeting - Second Letter to Route 3 Abutters - Second Neighborhood Abutters Meeting - Laconia, N.H. City Council Meeting - Articles in <u>Laconia Citizen</u> , <u>Manchester Union Leader</u>
May, 1992	- Interim Report (Eighmy et al., 1992) Issued
June, 1992	- Public Meeting with Laconia, N.H. City Council Public Works Subcommittee
September, 1992	- Notification Letter Sent to Abutters - R&D Application Sent to N.H. DES
October, 1992	- Second Test Batch at Franklin, N.H. Monofill
October, 1992	- Monitoring Wells Installed; Surface Soils Collected
November, 1992	- N.H. DES Request for Additional Information - Monitoring Wells Installed, Surface Soils Collected
November 20, 1992	- Sampling of Ground Waters, Surface Waters
December, 1992	- N.H. DES Air Resources Division Approval Request Submitted - Laconia, N.H. City Council Approval
January, 1993	- <u>Concord Monitor</u> Article - Present Research Project Starts

Table 1.9: The Chronology of the Laconia, N.H. Bottom Ash Paving Project - Continued

Date	Event
January 29, 1993	- Sampling of Ground Waters, Surface Waters
February, 1993	- N.H. DES Air Resources Division Approval
February 25, 1993	- Sampling of Ground Waters, Surface Waters
March, 1993	- N.H. DES Waste Management Division Approval - Public Hearing on Bond Issue
April 1, 1993	- Sampling of Ground Waters, Surface Waters
April 10, 1993	- Route 3 Reconstruction Initiated
April 29, 1993	- Lysimeter Trenches Constructed, RAP, Shallow Soils, Deep Soils, MW 5 Surface Soils Collected
May 6, 1993	- FWD Testing on Base Course
May 7, 1993	- Binder Courses Paved
May 8, 1993	- FWD Testing of Entire Road on Binder Course
May 15, 1993	- Wearing Courses Paved, t=0 Year Pavement Slabs Removed, Road Run-Off Systems Constructed
May 26, 1993	- ARAN Testing of Road
June 6, 1993	- Strain Gauge Testing
July 29, 1994	- t=1.25 Year Pavement Slabs Removed
November 5, 1993	- FWD Testing of Entire Road and Strain Gauges
March 6, 1994	- FWD Testing of Entire Road and Strain Gauges
April 5, 1994	- FWD Testing of Entire Road
June 26, 1994	- ARAN Testing of Road
October 13, 1994	- FWD Testing of Entire Road
December, 1994	- Present Research Project Completed

Site Plan and Pavement Construction

A plan view of the site is shown in Figure 1.4. The site consists of a control section and a test section. In May, 1993, 2,000 linear feet (610 m) of US Route 3 in the City of Laconia, New Hampshire was reconstructed using, in part, grate ash aggregate in a binder course pavement. US Route 3 in this area is a two lane secondary roadway having an average daily traffic, which varies seasonally, from about 5,000 to 20,000 vehicles per day.

Of the 2,000 linear feet (610 m) of reconstruction, 1,150 linear feet (350 m) was reconstructed in the control section using natural aggregate both in the two inch (5.0 cm) binder course and the one inch (2.5 cm) wearing course. The 850 linear foot (260 m) test section was constructed using a 50 percent bottom ash and 50 percent natural aggregate blend in the two inch (5.0 cm) binder course pavement. A one inch (2.5 cm) wearing surface of 100 percent natural aggregate was applied over the bottom ash asphalt binder course such that, in this demonstration, the grate ash asphalt binder is not exposed to traffic or the environment. The transition point from the control to the test section occurs near the intersection of White Oaks Road with Rt. 3 at station 158 + 92. Figure 1.5 shows a road cross-section schematic for the transition and standard sections. The transition point was located so that, jurisdictionally, the test section remained solely within the City of Laconia.

The original concrete pavement consisted of a variable thickness portland cement concrete pavement which had been overlaid with approximately three inches of asphaltic concrete. The pavement was in reasonably good condition and did not show signs of pumping or frost heaving. The original concrete pavement was six inches thick with an edge thickness of eleven inches. The three inches of asphaltic concrete was milled off

- 1) ELEVATIONS TAKEN FROM PLAN TITLED, "TOPOGRAPHIC SURVEY, CITY OF LACONIA, BELKNAP CO., N.H. PAUGUS BAY AREA". CONTOUR INTERVAL 5'. SCALE 1"=100' BY GORDON E. AINSWORTH & ASSOC. BY PHOTOGRAMMETRIC METHODS (FROM PHOTOGRAPHY DATED APRIL, 1957). HORIZONTAL CONTROL BY POLARIS. VERTICAL CONTROL BY U.S.C. & G.S.
- 2) UTILITIES SHOWN ARE VERY APPROXIMATE LOCATIONS SCALED FROM PLAN ENTITLED, "CITY OF LACONIA, N.H. PAUGUS INTERCEPTOR AND FORCE MAIN" BY METCALF & EDDY, BOSTON, MA. DATED AUGUST, 1985. LABELED RECORD DRAWING.
- 3) EXISTING SEWER IS 18" REINFORCED CONCRETE PIPE
- 4) EXISTING WATER MAIN NOT SHOWN ON THIS PLAN.

Figure 14
**The Laconia, NH Bottom Ash
 Paving Demonstration**

- SITE PLAN -

CMA ENGINEERS, INC.
CIVIL/ENVIRONMENTAL ENGINEERS

Northbound Lane Construction Details

Southbound Lane Construction Details

Figure 1.5: Pavement Construction Details for Northbound and Southbound Lanes

the concrete pavement and stored off-site for later use. These millings are referred to as recycled asphalt pavement (RAP). After the old concrete was broken up and removed from the site as waste, new base material was brought in to make up the difference in the loss of elevation caused by the removal of the old concrete pavement. The original base material was a high quality well graded gravel with less than two percent passing the No 200 sieve.

The RAP was then brought back and spread uniformly over the new base material and blended together by making several passes with a Caterpillar reclaimer. This new base (RAP and aggregate base material) is referred to as an asphalt stabilized base (ASB). The new ASB was not thick enough to match elevation of existing adjacent pavements (drive ways, parking lots, etc.) without adding additional asphaltic concrete. To make up for this elevation difference, the control on the Southern portion of the project had additional RAP trucked in and spread. Approximately 28 truck loads of RAP from other city projects were placed on the Northbound lane approximately from stations 172 through 158. These millings were then dispersed across both lanes by making four passes with the reclaimer. The end result is that the control section's ASB is thicker and has a higher asphalt content than the test section. Within the control section the Northbound lane would be expected to have a thicker ASB and an higher asphalt content than the Southbound lane at a given station. This would be expected to create a stronger base (and therefore pavement) for the control section than the test section.

The hot asphalt concrete control and test binders were compacted by using a rubber tired paver and rollers typically used on NHDOT projects, except vibratory rollers were not allowed due to concerns noted in the literature that they may have the

potential to fracture friable particles. First, several passes were made with a rubber tire breakdown roller, then a heavy three steel wheel roller was used to compact the layers to constant density as measured by nuclear density.

The binder course pavement for the first 100 feet on either side of the transition point between the test and control pavements was constructed to a four inch depth by placing two, two inch layers. This section was placed thicker to facilitate special testing with a Falling Weight Deflectometer (FWD), and strain gauges. It also allowed for the laboratory testing of specimens which required more than two inches of thickness. Moisture, resistance and temperature probes as well as traffic induction loops were also installed in the subgrade within this section. The wearing course remained at a one inch thickness through this 200 foot transition section.

The land use in the control section is commercial, including a resort/conference center, several smaller motels, a restaurant, and retail businesses. In the test section, abutting properties were largely undeveloped. Paugus Bay, a part of the Lake Winnepesaukee system, is in close proximity to the test section.

Scope of Work

Table 1.10 briefly identifies the major aspects of the physical testing for this study. A scope of the work for the environmental testing can be found in the companion volume (Eighmy et al., 1996a) to this report.

The scope addresses pavement design, laboratory mix design, paving procedures, field data collection, pavement performance evaluation, laboratory evaluation of field specimens, and evaluation of the American Association of State Highway and

Table 1.10: Physical Testing Scope of Work

Physical Testing	
1.	Laboratory Gyratory Testing Machine Mix Designs Unit Weight Gyratory Compactibility Index Air Voids Voids in Mineral Aggregate and Voids Filled With Asphalt Absorbed and Effective Asphalt Water Absorption of Bituminous Specimens Gyratory Shear Gyratory Shear Factor Gyratory Stability Index Grate Ash Degradation in Asphaltic Mixes
2.	Paving of Test and Control Pavements
3.	Field Instrumentation of Test and Control Pavements TDR Moisture Gauges Temperature and Resistance Gauges Traffic Induction Loops Strain Gauges
4.	Field Testing Moisture Temperature and Resistance Traffic Serviceability Falling Weight Deflectometer (FWD) Strain Serviceability
5.	Laboratory Testing Resilient Modulus Indirect Tension Marshall Stability Thermal Stress

Transportation Officials (AASHTO) design layer coefficient parameters for the test and control mixes.

A number of papers have been recently published regarding this present project. They include papers by Musselman et al. (1994a,b) and Eighmy et al. (1995). Readers can refer to these papers for general overviews of various aspects of the project.

Utilization Approval Process

According to the New Hampshire Solid Waste Rules, under Env-WM 318.03, Criteria for Direct Re-Use, a certificate for direct reuse can be obtained on the basis of demonstration that (i) the ash is produced by a process which is consistent and uniform, (ii) the ash is characteristically and analytically consistent, and (iii) there is a use or a market for the waste product where its use will not pose a risk to human health or the environment that is greater than the risk of secure disposal. The information contained in Eighmy et al. (1992) for the earlier two year study addresses criteria (i) and (ii). This report is designed to address criterion (iii). The formal approval of the direct reuse of the residue in the U.S. Route 3 reconstruction is intended to be sought under the provisions of Env-Wm 2602.06 as part of the Concord facility's residuals management plan.

CHAPTER 2. LABORATORY MIX DESIGN

Introduction

The concept of using municipal solid waste (MSW) combustion residues as construction materials is not new. In the 1940's, in an effort to overcome war-related shortages, the ash residue from coal power plants was utilized for embankment and subbase construction at various sites in Pennsylvania and New York (Pindzola and Collins, 1975). In the 1960's and 1970's, with the push to construct large, centrally located MSW facilities, interest in the technology of ash residue utilization was revived primarily due to the rising cost of ash disposal. Several studies were performed and demonstration projects were built to evaluate the technical feasibility of using ash residue as a total or partial aggregate replacement in paving mixtures.

Additionally, a process was developed for fusing ash residue to produce a higher quality aggregate material (Pindzola and Chou, 1974, Pindzola, 1976). Material produced using this process performed well in a bituminous surface course in Harrisburg, PA (Snyder, 1980). The most comprehensive effort addressing the potential for using ash residue in various highway applications was completed by FHWA in December 1976 (Collins et al., 1976). In this study, a nationwide survey of locations and types of MSW facilities was made, samples of residues were collected and characterized physically and chemically, bituminous mixture designs were developed for base and surface course applications, and field tests were constructed. The performance of these test sections varied from "Excellent" to "Poor" during the periods of evaluation. A few studies have looked at using fused or treated residues as aggregate substitutes (Snyder, 1980, Mehan and Hooper, 1991). Work by Chesner et al. (Chesner, 1989) has shown that bottom ash

from the Southwest Brooklyn, New York combustor can be used as an aggregate substitute.

All past ash studies, as previously presented in Table 1.2, employed either the Marshall or Hveem traditional mix design methods for mix evaluation. The performance of these demonstration projects have been inconsistent, varying from "Excellent" to "Poor". These traditional mix design methods are empirical, and are not directly related to pavement performance. Premature distress in many flexible pavements suggests that these empirical mix design procedures are inadequate.

To achieve a breakthrough in the performance of asphalt pavements, the Strategic Highway Research Program (SHRP) Asphalt Program was initiated in 1987 and completed in 1993. SHRP has developed the SUPERPAVE (Superior Performing Asphalt Pavements) Mix Design System. SUPERPAVE is a comprehensive system for the design of paving mixes tailored to the unique performance requirements dictated by the traffic, environment (climate), and structural section at a pavement site. It facilitates the selection and combination of asphalt binder, aggregate and, where necessary, a modifier to enhance pavement performance (Leahy, 1992). SHRP chose the gyratory compactor as the standard laboratory compaction equipment in the performance-based mix specification for several reasons (Leahy, 1992): testing showed the gyratory compactor simulates field compaction reasonably well, the gyratory kneading action closely simulates the shear loading associated with field scale roller compactors during paving and with traffic throughout the pavement's life, equipment size and cost make it suitable for both central lab and field control operations, and its use permits determination of specific gravity and air voids content during compaction.

The Corps of Engineers Model 4C Gyratory Testing Machine (GTM) with oil roller owned by the Maine Department of Transportation (MEDOT) was used in this study. When this research was initiated the MEDOT was the only New England state which had experience with the GTM so emphasis was placed on their criteria. The previous studies (Hughes, 1964, Ruth and Schaub, 1966, Potts, 1967) indicated the GTM can be used to simulate the initial field compaction and traffic densification. Maine DOT (Rand, 1992, MEDOT, 1992) has used the GTM in conjunction with the Hveem procedure. Their experience indicates an asphaltic hot mixture could be designed using the GTM exclusively rather than the Hveem procedure. More information can be obtained with the GTM: it can be used to predict initial density, density after several years, density under very severe traffic conditions, and density over a long period of exposure to traffic. It also will detect problems in mixes that are not detected by the Hveem or Marshall testing procedures.

Both the Marshall and GTM design methods were used to develop the mix designs for a 50 percent grate ash and control Type B binder mix. The Marshall procedure yielded an optimum asphalt content of 9.1 percent for the 50 percent grate ash and 5.5 percent for the control mix. Details of the Marshall mix design can be found elsewhere (Gress et al. 1992).

Materials

The upper limit of aggregate size is 3/4 inches (19 mm) for a NHDOT Type B binder base course mix. Figure 2.1 presents an average gradation of 72 samples taken over the 2 year earlier study along with the New Hampshire specifications for Type B base or binder mix. The gradation of grate ash meets NH specifications. This suggests

Figure 2.1: Grate Ash Gradation and Type B Binder Specifications

that, based on gradation alone, grate ash passing the 3/4 inches (19 mm) sieve could provide 100 percent of the aggregate in a job mix formulation. The grate ash evaluated in this study was a composite which was created by blending 72 samples taken in the earlier two year study.

The control aggregates used in this study were obtained from two sources, Pike Industries Inc., a materials supplier, asphaltic concrete producer and contractor, and Concord Sand and Gravel, a materials supplier. The 3/4 inch, 1/2 inch, 3/8 inch, dust and washed dust aggregates were manufactured crushed stone supplied by Pike Industries Inc. The sand supplied by Concord Sand and Gravel was glacial in origin.

The control mix consisted of a unique combination of Pike aggregates, which corresponded very closely to the mid point of the NHDOT gradation requirements. Grate ash was substituted for the control aggregates on a dry weight basis. The gradation of the 50 percent grate ash and the control mix met the requirements of a NH Type B binder mix as shown in Figure 2.2. The Pike aggregates and grate ash were separated individually on each sieve and recombined to produce the formula under study. To minimize gradation variation a separate batch was prepared for each specimen.

An AC-20 asphalt supplied by Pike Industries Inc. was employed in this study.

Mix Preparation and Compaction

The GTM laboratory test specimens were fabricated in standard four inch (101.6 mm) diameter molds. Aggregates and asphalt were heated separately at 295°F (146°C). This temperature was selected to simulate the plant mixing temperature required to provide an asphalt viscosity of 170 ± 20 centistokes. The mixing time was between 90

Figure 2.2: Gradation of Control and Test Type B Binder Blends

and 120 seconds. A mechanical mixer was employed. Three samples were prepared for each batch.

After initial mixing, the mixture was placed in a tray in a forced air oven set at 275°F (135°C) for three hours. The purpose of this heating condition was to simulate plant hardening (Von Quintus et al., 1991). After the initial heat conditioning, a representative sample of the mixture was taken and allowed to cool. Then, the maximum theoretical specific gravity was determined in accordance with AASHTO T209.

A Corps of Engineers Model 4C GTM with oil roller (as specified by ASTM D3387) and a 4 inch (101.6 mm) diameter mold was employed for sample compaction. The test apparatus has the capability of varying the angle through which the loading head rotates during each revolution. This angle is referred to as the angle of gyration and is selected based on the anticipated pavement deflection. An angle of one degree was selected for this research. Vertical ram pressure, which corresponds to the maximum anticipated tire contact pressure, was set at 120 psi (827.4 kPa). The ability to select a ram pressure equal to the anticipated tire contact pressure makes the GTM technique unique because the theoretical stress for compaction and maximum induced shear is based on the concept of employing realistic loads for the test. The compaction temperature was 230°F (110°C). The compaction efforts produced after 60 and 300 revolutions of the GTM were chosen to simulate the initial compaction and ultimate traffic densification respectively (Rand, 1992, MEDOT, 1992).

Asphalt contents investigated for the 50 percent test mix ranged from 6.0 to 9.5 percent. The asphalt contents for the control mix ranged from 4.0 to 6.0 percent. Asphalt content was based on weight of total mix.

The sample weight was measured prior to compaction. The sample height and the oil roller pressure at three roller locations as per ASTM D 3387 were obtained at 30, 60, 90, 120, 150, 200, and 300 gyrations. The shear strain experienced by the mixture during compaction was recorded with a gyrograph. After compaction, all specimens were cooled to room temperature, and the bulk specific gravity and water absorption were determined in accordance with AASHTO T166. Using the information obtained, calculations were made to obtain the physical and engineering properties for each specimen (Rand, 1992, MEDOT, 1992, ASTM, 1983). The degradation of weak friable particles caused by GTM compaction was also investigated.

Results and Discussion

The data discussed herein represents an average of three samples unless otherwise specified. In all figures presented in this section, C stands for the control mix, and T for the 50 percent grate test mix.

Unit Weight

Unit weight was determined in accordance with AASHTO T 166. Figure 2.3 presents unit weight as a function of GTM revolutions for the control and 50 percent test mixes. Unit weight increases with increasing GTM revolutions. Varying the asphalt content tends to shift the curves but does not change the general shape.

Unit weight as a function of asphalt content is shown in Figure 2.4. Sixty and 300 GTM revolutions simulate initial compaction and traffic densification, respectively. The unit weight of the control mix at 300 revolutions is 3 to 4 lb/ft³ (0.048 to 0.064 g/cm³) higher than at 60 revolutions, and that of the test mix is 2 to 7 lb/ft³ (0.032 to 0.112

Figure 2.3: Unit Weight Versus GTM Revolutions

Figure 2.4: Initial and Final Unit Weight Versus Asphalt Content

g/cm³) higher. Unit weight increases with revolutions and slightly with increasing asphalt content with the exception that the test mix unit weight at 300 revolutions varies inconsistently with asphalt content. In concept, unit weight should increase initially as asphalt content increases, and reach a peak, and then begin to decrease. One explanation of this is that the aggregate particles for both the control and the test are angular and therefore, have large surface areas. The asphalt contents used were not sufficient to push the aggregate particles further apart. Hence, the trend of lower unit weight at higher asphalt contents was not observed.

Gyratory Compatibility Index (GCI)

GCI is the ratio of the unit weight of asphaltic concrete at 30 revolutions to that at 60 revolutions (ASTM, 1983). GCI is an indicator of the compatibility of the mix. The closer this index approaches unity, the easier the mix is to compact. Figure 2.5 presents GCI as a function of asphalt contents. The GCI of the control mix ranges from 0.981 to 0.984, with an average of 0.982, and that of the test mix ranges from 0.977 to 0.979, with an average of 0.978. Although the differences between these indexes is small, it is expected the test mix may require a slightly higher compaction effort, due to the angularity of the grate ash.

Air Voids

Air voids in a compacted paving mixture consist of small air spaces between the coated aggregate particles. This is the most important parameter in mix design. It is well known that an asphaltic concrete mixture that is compacted to a low air void content will have increased fatigue life, reduced permanent deformation, reduced

Figure 2.5: Gyroscopic Compatibility Index Versus Asphalt Content

distortion, reduced aging of the asphalt cement, and reduced moisture damage than will the same mixture that is compacted to a higher air void content (Von Quintus et al., 1991). It is also well known that air voids at a given time are dependent on tire pressure, wheel load magnitudes, traffic, and environmental conditions. Hence, the air void level at which the engineering properties are measured is critically important. An air void level of five to eight percent after construction and that of three to five percent after traffic densification has been found to be acceptable in most environments for both surface and binder mixes (Von Quintus et al., 1991).

Figure 2.6 presents air voids as a function of GTM revolutions. Air voids of the control and test mixes decrease significantly at the beginning, then tend to be less affected with increased revolutions. Use of different asphalt contents tends to shift the entire curve but does not change its general shape.

The effect of asphalt content upon air voids at 60 revolutions is shown in Figure 2.7. Both mixes exhibit the same trend. As a result of air voids being filled with asphalt, air voids decrease with increasing asphalt contents. To achieve the same air voids, the test mix requires more asphalt cement (approximately 2.3 to 2.5 percent more) than the control mix because of the absorptive nature of grate ash. The asphalt content meeting the air voids specification of five to eight percent ranges from 4.4 to 5.7 percent for the control mix and seven to eight percent for the test mix.

Figure 2.8 presents the air voids at 300 revolutions as a function of asphalt content. Air voids decrease with increasing asphalt content, with the exception of a one point anomaly for the test mix. The reason for the anomaly is not known although it seems reasonable that it was experimental error. At the same air voids content, the test mix requires approximately two to 2.7 percent more asphalt than the control mix. The

Figure 2.6: Air Voids Versus GTM Revolutions

Figure 2.7: Initial Air Voids Versus Asphalt Content

Figure 2.8: Final Air Voids Versus Asphalt Content

asphalt content meeting the air voids specification of three to five percent ranges from 4.8 to 5.3 percent for the control and 6.8 to 7.9 percent for the test mix.

Voids in Mineral Aggregate (VMA) and Voids Filled with Asphalt (VFA)

Both VMA and VFA have been considered as mix design parameter specifications. The Asphalt Institute and many state highway agencies have adopted minimum VMA requirements for mixture design. The Corps of Engineers and Federal Aviation Administration have adopted limits on final air voids of three to five percent and a VFA requirement of 75 to 85 percent to ensure the mixture's durability. On the other hand, other's such as the National Asphalt Pavement Association (NAPA, 1986) have argued that there are insufficient performance data to justify these minimum and maximum requirements. The National Cooperative Highway Research Program (NCHRP) (Von Quintus et al., 1991) investigated the effect of VMA on indirect tensile strength at a test temperature of 41°F (5°C). The data showed a trend, but no distinguishable relationship was found to exist. It is commonly agreed that the VMA specification should be supported by field performance data. Table 2.1 presents the guidelines for VMA and VFA (Von Quintus et al., 1991).

VMA as a function of asphalt content is shown in Figure 2.9. The VMA of the control mix at 300 revolutions does not meet the 14 percent minimum requirement, with the exception that the VMA at four percent asphalt does. The VMA of the test mix at 300 revolutions does not meet the specification at any asphalt content (4.3 to six percent lower than the 14 percent minimum specification). The control only meets the specifications at less than 4.5 percent asphalt cement. VMA was not used as a design criteria in this study.

Table 2.1: VMA and VFA Guidelines (Von Quintus et al., 1991)

VMA and VFA Criteria			
Property	Aggregate Size, in.	Minimum %	Maximum %
VMA	1.5	12	20
	1	13	20
	0.75	14	21
	0.5	15	21
VFA		75	85

Figure 2.9: Absorbed Asphalt

VFA is inversely related to the air voids. As the percentage of the air voids approaches zero, VFA approaches 100. The asphalt mix is initially compacted to some percentage of VFA (usually 50 to 70 percent). This percentage increases as the asphalt mix continues to densify under traffic. When VFA exceeds approximately 80 to 85 percent, the asphalt mix typically becomes unstable and rutting is likely to occur (Robert et al., 1991). Figure 2.10 presents VFA as a function of asphalt content. As would be expected, VFA increases with increased asphalt. The asphalt content meeting the 75 to 85 percent VFA requirement for the control mix ranges from 5.3 to 5.8 percent, and that of the test mix ranges from 8.4 to 8.7 percent.

Absorbed and Effective Asphalt

Asphalt absorption is a time and temperature dependent variable (Kandhal and Khatri, 1991). The laboratory investigation of absorption must consider plant and field conditions. In this study, a heating condition of 3 hours at 275°F (135°C) was selected to simulate plant hardening. Figure 2.11 presents the absorbed asphalt by weight of total mix as a function of asphalt content. Each data point was based on one measurement. The absorptions of the control range from 0.94 to 1.06 percent, and those of the test range from 4.64 to 5.48 percent. The test mix absorbs asphalt cement approximately five times as much as the control mix. The permeable pores of the grate ash make it absorptive. The variation of asphalt absorption for the test mix was observed.

Figure 2.12 presents the effective asphalt as a function of asphalt content. Each data point was based on one measurement. The effective asphalt increases with increased asphalt content. As a result of high asphalt absorption, the test mix demands

Figure 2.10: Voids in the Mineral Aggregate Versus Asphalt Content

Figure 2.11: Voids Filled With Asphalt Versus Asphalt Content

Figure 2.12: Effective Asphalt Versus Asphalt Content

more asphalt cement to obtain the same degree of coating (effective asphalt) as the control mix.

Water Absorption of Bituminous Specimens

Figure 2.13 presents the water absorption as a function of asphalt content. Absorption decreases with increased asphalt content, indicating all samples become less permeable with increasing asphalt content. The absorption at 300 revolutions was less than at 60 revolutions at any asphalt content, showing that increased compaction effort has made the samples less permeable. At higher asphalt contents, the voids were almost completely filled with asphalt such that water absorption was low regardless of the type of mixes. The absorption of the test mix was comparable to that of the control mix because the high asphalt absorption by the test mix makes it less permeable.

Gyratory Shear (S_g)

Gyratory shear, S_g , is the shear resistance of the specimen which is, among other things, a function of the imposed vertical pressure and degree of strain. Gyratory shear can be used to evaluate the stability of a mix. The formula for S_g is as follows (Rand, 1992, MEDOT, 1992):

$$S_g = 4.00 \times \text{Oil Roller Pressure} / \text{height of specimen in inches.}$$

A reduction in oil-filled roller pressure or S_g during the compaction process indicates loss of stability. Currently, there is no official specification for gyratory shear values. In general, stability was assured by specifying a gyratory shear (S_g) value between 30 and 40 psi (206.84 to 275.79 kPa) after traffic densification (Rand, 1992, MEDOT,

Figure 2.13: Water Absorption Versus Asphalt Content

1992). Maine DOT (Rand, 1992) recommends that a minimum gyratory shear value of 35 psi (241.32 kPa) after 300 revolutions be adopted as the minimum design value for most traffic conditions. If very heavy traffic conditions are expected such as at bus-stops, or city street intersections with traffic lights, then a minimum S_g value of 30 psi (206.84 psi) after 500 revolutions should be used.

S_g as a function of revolutions is shown in Figure 2.14. As would be expected, S_g decreases with increased revolutions for the control and test mixes. Figure 2.15 presents gyratory shear at 300 revolutions as a function of asphalt content. The minimum S_g value of 35 psi (241.32 kPa) after 300 revolutions, as recommended by Maine DOT, was used to evaluate the test results. The control mix with < 5.7 percent asphalt content and the test mix with < 7.5 percent asphalt content met the Maine DOT gyratory shear requirement of a minimum value of 35 psi (241.32 kPa). The S_g for the control mix remains stable until asphalt content reaches 5.5 percent, and the S_g for the test mix decreases more significantly at high asphalt contents than at low asphalt contents.

Gyratory Shear Factor (GSF)

GSF is the ratio of the measured gyratory shear stress to the approximate theoretical maximum induced shear stress. This is a factor of safety type index with regard to failure in simple shear for the defined loading conditions. GSF must be interpreted with recognition of the assumption of plane maximum shear (the unit contact pressure divided by p) for a strip load on a homogeneous elastic isotropic mass. The approximate theoretical maximum induced shear stress (Rand, 1992, MEDOT, 1992) is:

$$t_{\max} = \text{Vertical Pressure} / p = 120/3.14 = 38.2 \text{ psi (263.39 kPa)}.$$

Figure 2.14: Gyrotory Shear Versus GTM Revolutions

Figure 2.15: Gyratory Shear Versus Asphalt Content

The gyratory shear factor (Rand, 1992, MEDOT, 1992) is determined as follows:

$$\text{GSF} = S_g / t_{\max}$$

Figure 2.16 presents GSF as a function of asphalt content. As asphalt content increases, GSF decreases. The control and test mixes at 60 revolutions remain stable at any asphalt content. At 300 revolutions, the control mix becomes unstable with the asphalt content greater than 5.5 percent, and the test mix becomes unstable with asphalt content greater than 7.5 percent. "Unstable" means a mix does not have sufficient shear strength to resist shear failure.

Gyratory Stability Index (GSI)

GSI is the ratio of the maximum gyratory angle to the minimum gyratory angle,

$$\text{GSI} = q_{\max} / q_{\min}$$

where $q_{\max}$ and $q_{\min}$ are the maximum and minimum angles respectively the sample deforms to during the rotation of the gyratory compactor for a given number of revolutions (usually 300).

The gyratory angle is a measure of the magnitude of the gyratory strain. The GTM is equipped with a gyrograph recorder. The shear strain (or pavement rutting potential) experienced by the bituminous mixture during the compaction test can be investigated by looking at the gyrograph obtained during compaction. No widening of the gyrograph indicates an ideally stable condition at this asphalt content, also reflected as a GSI value of unity. When the gyrograph widens excessively, instability of the mix at this asphalt content occurs and rutting is anticipated. This is reflected in GSI values greater than unity.

Figure 2.16: Gyratory Shear Factor Versus Asphalt Content

Illinois studies (MEDOT, 1992) showed the gyratory stability index (GSI) should not exceed 1.25 after 300 revolutions. If the GSI value exceeds 1.30 then rutting is very probable. Canadian literature indicates a much more stringent specification to prevent rutting (MEDOT, 1992). This literature indicates that the GSI value should not exceed 1.06 after 500 revolutions using a ram pressure and compaction angle similar to those used by Maine DOT. The research conducted by Maine DOT showed that GSI should be less than 1.15 after 300 revolutions to prevent pavement rutting. The criteria of GSI recommended by Illinois and Maine are used in this study for evaluating test results.

Figure 2.17 illustrates GSI as a function of asphalt content. At 60 revolutions, all mixes remain stable with GSI ranging from 1.011 to 1.024 for the control mix and 1.005 to 1.07 for the test mix. At 300 revolutions, GSI increases appreciably with increased asphalt content, indicating the mixes become unstable with increased asphalt content. The control mix with ≤ 5.75 percent asphalt and the test mix with ≤ 7.5 percent asphalt met the Maine DOT GSI requirement of a maximum of 1.15. The test mix requires approximately 1.75 percent more asphalt cement than the control mix.

Grate Ash Degradation in Asphaltic Mixes

Extraction tests were conducted in accordance with AASHTO T164 - Method A to investigate the grate ash degradation caused by laboratory compaction. The samples chosen for the extraction test included the test mix at 60 and 300 revolutions with 7.70 percent asphalt and the control mix at 60 and 300 revolutions with 5.11 percent asphalt. Figure 2.18 presents the difference in gradation before and after compaction. Degradation occurred in all mixes and was associated with particle size. The test mix degraded more than the control mix for particles in the 3/8 inches (1 cm) to No. 20

Figure 2.17: Gyratory Stability Index Versus Asphalt Content

Figure 2.18: Grate Ash Degradation Versus Particle Size

Figure 2.20: Optimum Asphalt Content Versus Asphalt Content

3. The unit weight was found to follow an asymptotic relationship with revolutions.
4. The test and control mixes have low water absorption.
5. Gyratory shear at 300 revolutions decreases with increasing asphalt content.
6. All mixes at 60 revolutions remain stable with GSI ranging from 1.005 to 1.07 for the test mix and 1.011 to 1.024 for the control, and the GSI at 300 revolutions increases appreciably with increasing asphalt content.
7. Degradation occurred in all mixes. The test mix degraded more than the control mix for particles in the 3/8 inches to No. 20 size range. The difference between gradations at 60 and 300 revolutions for the test and control mixes were not significant, indicating the degradation under traffic is not problematic.
8. The test mix absorbs more asphalt cement than the control mix.

(1 mm) size range. It appears that for both the test and control mixes the difference between gradations at 60 and 300 revolutions are not significant, indicating the degradation under traffic is not problematic.

Selection of Design Asphalt Content

The selection of the design asphalt contents is based on the four most important requirements among all factors considered. They included stability index, gyratory shear stress, air void after initial compaction and air void after final compaction. Figure 2.19 presents the allowable ranges of asphalt contents that are defined as those values being within the minimum and maximum limits of the requirements. The design asphalt contents selected for the demonstration project were 5.1 and 7.0 percent for the control and test mixes respectively.

Figure 2.20 compares the optimum asphalt contents developed by the GTM and Marshall design methods. The GTM methods predicted lower optimum asphalt contents than the Marshall method.

Summary and Conclusions

Based on the results and discussion as presented the following conclusions are drawn:

1. The design asphalt contents meeting all design criteria were 7.0 percent for the test and 5.1 percent for the control mixes.
2. The air voids decrease with increasing revolutions and asphalt contents.
The test mix meeting the initial and final air void requirements requires more asphalt cement than the control mix.

Figure 2.19: Allowable Range of Design Requirements Versus Asphalt Content

CHAPTER 3. PAVEMENT DESIGN AND TESTING PROCEDURES

Pavement Design

Pavement design strategies must consider the factors of subgrade and pavement layer strength and composition, load and traffic characteristics, environmental conditions and economics. An asphaltic concrete pavement, often referred to as a flexible pavement, must be designed to withstand stress repetitions based on the expected volume of traffic over its design life. The number of stress repetitions depends upon the type of road. A heavily traveled road may carry over 50,000 vehicles per day, whereas a low volume road may only carry 100 vehicles per day. The greatest percentage of vehicles using US roads are passenger cars and light trucks, which do not affect the performance of a pavement or its life appreciably. Pavements are designed for truck traffic specifically, and commonly are designed for well over 10^6 applications of equivalent 18000 pound (18-kip) single axle loads. Almost all state highway agencies use the number of equivalent 18-kip single axle loads, referred to as ESAL's, as an indicator of traffic and load intensity. Although there are several accepted pavement design methods, emphasis will be placed on the most highly accepted and utilized AASHTO Method.

The "AASHTO Guide for Design of Pavement Structures", published by the American Association of State Highway and Transportation Officials, 1993 is the most highly regarded and frequently used method of pavement design (AASHTO 1993). A brief description is presented herein of the design principles presented in the guide. Further details may be found in the reference. The pavement design guide discusses the following parameters for consideration in a pavement design:

- pavement performance
- traffic
- roadbed soil
- materials of construction
- environment
- drainage
- reliability
- life-cycle costs
- shoulder design.

Although the guide describes specific methods and procedures for the design of pavements it makes an effort to be flexible such that it may be used by all state agencies. For example, provisions are available for each user to input material properties consistent with their experience.

The fundamental AASHTO design equation for designing the layers of a pavement is:

$$\log_{10}(W_{18}) = Z_R S_o + 9.36 \times \log_{10}(SN + 1) - 0.20 + (\log_{10} \Delta PSI) / 2.7 \times (0.4 + (1,094)/(SN + 1)^{5.19}) + 2.32 \times \log_{10}(M_R) - 8.07$$

where

- W_{18} = predicted number of 18-kip equivalent single axle load applications
- Z_R = standard normal deviate
- S_o = combined standard error of the traffic prediction and performance prediction
- ΔPSI = difference between the initial design serviceability index, p_o , and p_t , the terminal serviceability
- MR = resilient modulus in psi.

SN is equal to the structural number indicative of the total pavement thickness required

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 + \dots + a_i D_i m_i$$

where

a_1 = the asphaltic concrete layer coefficient

D_1 = the asphaltic concrete layer thickness

a_i = the i^{th} layer coefficient

D_i = the i^{th} layer thickness in inches

m_i = the i^{th} drainage coefficient.

The structural number is an abstract number that represents the strength of the pavement system required to withstand the total traffic, expressed in equivalent 18-kip single axle loads, terminal serviceability, and the environment. The structural number is converted to actual thickness of asphaltic concrete, base material and subbase material by means of appropriate values of the layer coefficients and thickness representing the relative strength of the construction materials being used. The AASHTO guide and NHDOT use the layer coefficients presented in Table 3.1.

Table 3.1: AASHTO and NHDOT Layer Coefficients

Layer Coefficients		
Material	AASHTO	NHDOT
hot bituminous base	0.44	0.34
hot bituminous binder and wearing surface	0.44	0.38
crushed gravel base	0.14	0.07
sandy gravel subbase	0.11	0.05

These coefficients and others were developed using elastic multilayer theory and are presented in NCHRP Report 128, "Evaluation of AASHTO Guide for Design of Pavement Structures," 1992. Using multilayer theory or other means, these coefficients may be adjusted by increasing or decreasing their magnitude as required in order to

maintain constant strain or stress to provide comparable performance. Most state agencies have adjusted these coefficients to correlate to their local materials and environment. New Hampshire has elected to lower the coefficients as shown in the second column above. This increases the actual thickness required for the layer being designed.

Knowledge of the layer coefficients of a spectrum of different materials allows the pavement designer great flexibility in developing a design strategy for a given pavement. As discussed above the layer coefficients must be known for all the materials to be used in the basic AASHTO pavement design equation. Knowing the layer coefficient for the grate ash amended binder mix would allow the pavement designer to compare pavement designs using conventional binders with designs using grate ash amended binders.

Pavement Layer Coefficient Determination

The layer design coefficient of a given material can be determined by several methods; (i) it may be assumed to follow AASHTO criteria applicable to all HMA mixes, (ii) by back calculation of FWD deflection data, and (iii) by substituting FWD deflection data into empirically derived equations.

Using FWD to determine the layer coefficient requires the determination of Young's modulus of all the layer materials. Several problems arise in the process of attempting to establish a good statistical average of an asphaltic concrete modulus; the underlying pavement materials are not homogeneous, uniform or elastic. The asphaltic concrete materials are viscoelastic, and temperature and moisture are recordable but not

controllable under field conditions. The end result is a constantly changing pavement system that is extremely sensitive to temperature and moisture.

This research project determined the layer coefficients for the control and grate ash binder mixes. Given these layer coefficients, any DOT agency could make appropriate local and regional adjustments and then design a pavement with grate ash using the AASHTO design equation.

Pavement Evaluation

Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) is an impulse loading device which is used universally to measure surface deflections of pavements. The device is capable of applying a variable impulse load to a pavement which simulates a moving truck wheel load. The FWD creates the impulse load within a wide range of approximately 5000 to 23000 pounds (2270 to 10450 Kg) by hydraulically lifting weights up to about 15 inches (38.1 cm) and dropping them on a spring buffer (rubber pad) of radius 5.91 inches (15 cm). The impulse load is transferred from the spring buffer to the pavement over a period of about 25 to 30 milliseconds duration. The load transferred to the pavement can be calculated from:

$$F = [2Mghk]^{1/2}$$

where

F = peak force generated by FWD

M = mass of the falling weight

g = gravitational acceleration

h = height of drop

k = spring constant

Pavement surface deflections are measured at 0, 8, 12, 18, 24, 36, and 60 inches (0, 20.3, 30.5, 45.7, 61.0, 91.4, and 152.4 cm) from the centerline of the load. These deflections are measured by velocity transducers placed on the surface of the pavement.

As the load is transferred through the pavement it spreads out and down through the underlying layers at an angle somewhere between 30 and 45 degrees depending on the properties of the layer materials. The incline of the line as it goes through each layer is related to the relative stiffness of the layer material. As the modulus increases the stress is displaced over a larger area. Likewise a low modulus spreads the stress over a smaller area. The distance from the center of the load to where the stress zone intercepts the lower layers is unique. Deflection measurements obtained at a given distance are a function of the thickness and stiffness of the material located approximately at the depth equal to the horizontal distance from the load. This allows the subgrade stiffness to be estimated independent of the overlying layers.

Objectives

The objectives of evaluating the pavement with the FWD device were to determine the AASHTO layer coefficients for the control and test Type B binder courses, and to evaluate the response of the pavements during the destructive Spring thaw period.

AASHTO layer coefficients can be determined indirectly through FWD analysis by calculating the elastic modulus of the layer, converting the elastic modulus to a

reference temperature of 20°C, and then determining the layer coefficient by using an AASHTO empirical relationship.

Utilizing FWD surface deflections to determine pavement layer moduli involves using multilayer elastic theory to calculate a theoretical deflection profile. The theoretical deflection profile is then compared to the actual deflection profile obtained by FWD analysis. If the results are not within a predetermined amount of error, the process is repeated until the deformations are equivalent to those measured in situ with the FWD. The hypothesis is that a unique set of layer moduli and their thickness exist such that the theoretically predicted deflection profile is equivalent to the field measured deflection profile. The process of determining the layer moduli and their thickness is referred to as “back calculating” (Lytton, 1988).

Using FWD to monitor a pavement's response to climatic effects is easily accomplished by observing the surface deflections of a pavement during the critical period in question. In this case, surface deflections were observed at the beginning of the winter and at the time the subsurface materials were in an unstable state during spring thawing. This is a critical period for the HMA due to loss of support by the subsurface materials.

Testing Procedure

The FWD testing procedure consisted of placing the apparatus over a given station and then applying a preprogrammed load dropping sequence. A series of three seating drops of approximately 16 kips (7,258 kg) were used before the data collection began. After the seating drops were completed, four drops were made at each test load of approximately 9 kips (4,082 kg), 12 kips (5,443 kg), 16 kips (7,258 kg) and 22 kips

(9,979 kg). On test dates March 26, 1994 and November 5, 1995 an HWD was used to obtain pavement deflections. The procedure was the same as with the FWD except the test loads were 7 kips (3,175 kg), 9 kips (4,082 kg) , 11 kips (4,990 kg), and 15 kips (6,804 kg).

Protection for deflection overflow during the spring thaw period was set so the test data would be rejected if any deflection exceeded 80 mils (2.0 mm).

Performance Criteria

The performance of any pavement system is a function of its internal properties as well as its external properties. Specifically, the pavement must be strong enough to withstand the effects of traffic and the environment and the surface must be reasonably smooth yet have a coefficient of friction which assures the safety of its users.

Serviceability is a parameter used by all highway departments and agencies responsible for the design and maintenance of transportation facilities. The serviceability of a pavement is typically defined as its ability to adequately serve the type of traffic that uses the facility. Traffic includes the quantity of vehicles (automobiles and trucks) which use the pavement as well as their wheel loads and axle configuration.

The primary measure of serviceability is the Present Serviceability Index (PSI), which ranges from 5 for a perfect road to 0 for a road which is impassable. The serviceability-performance concept is utilized by DOT agencies as a means to design and rehabilitate pavements by assigning a minimum level of serviceability desired at the end of the performance period for a given total volume of traffic. Selection of the lowest allowable PSI, identified as the terminal serviceability, is done at the design stage of a pavement and is based on the lowest condition of the pavement that will be tolerated by

the users prior to resurfacing, rehabilitation or reconstruction of the pavement. A terminal serviceability of 2.5 or higher is suggested by AASHTO for major highways and 2.0 for highways of lesser traffic volume. Guidelines suggested by AASHTO to identify a minimum level of service established on the basis of public acceptance are presented in Table 3.2.

The time it takes a given pavement to reach its terminal serviceability depends on the traffic and environmental effects (frost heaving, soil swelling, aging, drying, and overall disintegration caused by weathering) as well as the original (initial) PSI. Higher original serviceability increases the life of a pavement. Initial serviceabilities obtained in the original AASHO Test Road were 4.2 and 4.5 for flexible pavements and rigid pavements respectively.

Table 3.2: AASHTO Terminal Serviceability Public Acceptance Guidelines

Terminal Serviceability	People Stating Unacceptable
3.0	12 %
2.5	55 %
2.0	85 %

The PSI is determined by measuring roughness and distress (cracking, patching and rut depth) at a given time during the life of the pavement. Roughness is the dominant factor in estimating the PSI of a pavement. The emphasis placed on roughness makes the method applied for measuring it very important in monitoring the performance history of pavements.

The original research findings of the AASHO Test Road that were used to develop the equations utilized in calculating PSI have been modified by most users of the

present AASHTO Guide for the Design of Pavement Structures. These modifications are the result of applying local experiences in determining PSI. It is presumed that these modifications do not affect the original concept that PSI values represent the ride quality of a pavement. Many agencies evaluate only roughness due to the small effect that physical distress has on PSI.

The importance of measuring roughness for use in the design of pavements, through PSI, can not be overstated. The change in roughness with time controls the life cycle of pavements. The quality of construction therefore can negatively affect pavement performance by not creating a pavement with high smoothness. It is possible to construct a high quality pavement and through poor craftsmanship apply the last surface such that the initial serviceability is lower than what AASHTO considers to be standard for initial construction. Such a pavement would however be expected to have a service life similar to a pavement made with the same materials but having excellent surface smoothness. The difference between these two pavements is only original surface smoothness as defined by a given method of measurement. The pavement with the smooth surface would be considered to have a higher initial serviceability.

Automatic Road Analyzer (ARAN)

Performance was evaluated on the project by the Data Collection Department of the Bureau of Transportation and Planning of the New Hampshire Department of Transportation. Surface roughness was recorded by the NHDOT Automatic Road Analyzer (ARAN) vehicle in the estimated wheel paths of the northbound and southbound lanes at approximately 40 miles/hour (64.4 km/hr). These data were used to predict the PSI of the test and control pavements.

CHAPTER 4. INSTRUMENTATION OF TEST AND CONTROL PAVEMENTS

The test and control four inch thick sections were instrumented in the pavements with various gauges so that moisture, resistance, temperature, traffic and strain could be monitored. Special probes were made to contain and protect the gauges and their cables from construction equipment and other effects which could have decreased service life. All assemblies were made from electrical grade PVC tubing ranging from 1/2 inch to 1 1/4 inches (1.3 to 3.2 cm) in diameter. Figure 4.1 shows the instrumentation plot view of the location of the various gauges. The transition from the test to control occurred at station 159 + 82, with the grate ash test on the left and the control on the right. The approximate transition from the standard two inch binder section to the four inch test section and control section are approximately located at stations 158 + 82 and 160 + 82 respectively.

The moisture and resistance and temperature gauges were installed as probe assemblies to minimize installation time. A hole approximately three by four feet and four feet deep was excavated with a backhoe and the probe assemblies were placed along the relatively undisturbed edge of the hole closest to the centerline of the pavement. Care was taken during backfilling and compaction of the excavated material so as to protect the gauges as much as possible. A hand operated plate vibratory compactor was used to recompact the materials after the probes were installed.

TDR Moisture Gauges

Moisture was measured using a Time Domain Reflectometer (TDR). A Tektronix model 1502B Time Domain Reflectometer was used to measure the moisture contents in the base, subbase and subgrade materials at approximately 12, 24, 36, and 48

INSTRUMENTATION PLOT PLAN

Figure 4.1: Instrumentation Plot View

inches (30.5, 61.0, 91.4, and 121.9 cm) below the finished pavement surface of the test and control sections. Figure 4.2 shows the TDR probe assemblies and their approximate location relative to the test and control sections. The test and control probe assemblies were located approximately at the center of the south bound lane at station 159 + 67 and station 159 + 98 respectively.

A TDR device, although not manufactured specifically for measuring moisture, is in fact a good device for measuring moisture. It is designed to determine the location of a broken wire by sending an electrical pulse down a cable. It detects any reflections made by discontinuities by sensing changes in impedance along the cable. This is known as time domain reflectometry. The device measures the moisture content by measuring the impedance between two parallel stainless steel rods. The length of cable never changes so the output is proportional to volumetric water content of the material between the two rods. The probe assemblies were installed next to the rods which were driven into the undisturbed side wall of the excavated hole. The TDR cables were attached to the rods and fast setting epoxy was applied over the ends. The entrance of the cables into the probe assemblies was coated with silicone sealant to prevent water from entering.

Temperature and Resistance Gauges

The temperature and resistance gauges were preassembled as shown on Figure 4.3. The probes were manufactured with 20 thermocouples and 17 copper rings. The thermocouples were installed at the surface of the wearing course, at the top of the binder, in the middle of the binder, on the bottom of the binder, and then every three inches down to 53 inches (134.6 cm) below the finished surface of the pavement. The

MOISTURE TDR PROBE

Figure 4.2: TDR Moisture Probe

RESISTANCE AND THERMOCOUPLE PROBE

Figure 4.3: Temperature and Resistance Probe

thermocouple gauges were located half way between each resistance ring. Each resistance gauge was wired independently so the resistance between each ring could be determined.

Induction Loops

The Northbound and Southbound 48 x 48 inch (121.9 x 121.9 cm) square induction loops were installed approximately 12 inches (30.5 cm) below the surface of the stabilized base. The induction loop assemblies, as shown by Figure 4.4, were constructed of 1/2 inch (1.3 cm) electrical grade PVC pipe and connections. Three loops of parallel 14 gauge copper wire with a water resistant casing were fed through the 48 x 48 inch (121.9 x 121.9 cm) pipe assembly. The two lead wires were twisted to prevent unwanted induction from occurring in the area between the loop and the control box. The location of the Northbound and Southbound induction loops is shown on Figure 4.4.

Strain Gauges

Six gauges were installed on the bottom of the binder mixes as shown on Figure 4.5. The installation process consisted of pouring hot AC-20 asphalt cement similar to what was utilized in the binder mixes on top of the asphalt stabilized base to form a flat base to support a rich hot sand mix which the gauge was pressed into. The gauge was then surrounded by the hot sand mix such that the top surface of the gauge was flush with the asphalt stabilized base. The gauges were protected from wheel loads by placing plywood over them until the binder mixes were applied. Control gauge #6 was run over

Figure 4.4: Induction Loops

STRAIN GAUGES

Figure 4.5: Strain Gauges

by the front tire of one of the trucks loading the hot mix into the paver. The gauge was not visually damaged, so it was reinstalled as discussed above.

CHAPTER 5. PAVING OF TEST AND CONTROL PAVEMENTS

Procedures

During the period of November to December 1992, approximately 400 wet tons (363 Mg) of grate ash were collected and delivered to the Franklin landfill. The residues were trommeled through a 3/4 inch (1.9 cm) trommel to remove oversize material as the grate ash was delivered to the landfill. The trommeled material was then piled and aged for about six months prior to its delivery to the Pike Asphalt Plant in Pembroke, New Hampshire.

After the first few loads of grate ash were delivered and trommeled, seven grab samples were randomly removed from the pile using a spade and a three dimensional grid coordinate randomizing process. At the end of the six month aging process, the grate ash pile was again sampled using a backhoe and spade to collect fourteen aged grab samples. This was done to facilitate the characterization of fresh grate ash versus aged grate ash. Table 1.4 describes the tests that were applied to the grate ash samples.

The average grain size distribution obtained from the representative sampling of the 400 ton (363 MT) stockpile was utilized to determine the job mix formula for the test pavement. The control job mix formula was similar to the standard mix supplied to NHDOT projects. The hot mix asphalt (HMA) was produced in one of Pike Industries asphalt plants located at Pembroke, New Hampshire. The mix sampling procedure was to obtain representative samples from the trucks after they were loaded at the asphalt mix plant. Three trucks were sampled for the control and test mixes, one at the start of the paving, one in the middle and one near the end of the paving. Five Marshall pucks were prepared from the mixes and their dry weights and submerged weights were determined. Values for densities were calculated and relayed to the paving site for

immediate use in density control. One sample from each truck was extracted and the asphalt content and grain size distribution determined. The Marshall pucks were transported to UNH laboratories for Marshall stability and flow testing.

The control binder mix was placed first, starting on the Northbound lane using a short base rubber tire paver. A butt joint was made at the transition between the control and test sections by paving up to a two by four inch board placed transversally across the pavement lane. The four inch section was done with two lifts, the second being placed as soon as the first was compacted. The paver was then taken back to the South end of the project and the Southbound lane was paved going in a Northerly direction. The four inch section was paved in a similar manner as previously described.

Compaction was monitored with a New Hampshire DOT nuclear density apparatus. A compaction control procedure was developed by measuring the compaction obtained with passes of the rollers. Conventional rollers normally used on NHDOT projects were used to compact the HMA binder courses except vibratory rollers were not allowed due to the effect they could have on fracturing the friable ceramic particles in the grate ash aggregate. The procedure developed for compaction control consisted of making several passes with the breakdown roller, then as many passes as required with the heavy three wheel steel roller to compact the layers to constant pavement density as measured by nuclear density. Density testing is not normally utilized by the NHDOT to control the compaction of binder courses, however due to the importance of the project the entire 2000 feet (.61 km) of the project was tested to ensure that adequate compaction occurred.

Asphalt Plant Moisture Control

The grate ash was stored in a large pile after it was processed through the $\frac{3}{4}$ inch sieve and the over-sized material was removed. The pile had a considerable amount of snow on it throughout the winter which slowly melted and helped maintain a relatively high moisture content. The moisture content was approximately 20-25 % upon arrival at the asphalt batch plant. The effect of the high moisture content was compensated for by slowing down the feed rate of the aggregates into the dryer such that the aggregates had more time in the dryer to allow removal of the extra moisture. Even though the moisture was very high and 50 % of the job mix was grate ash, controlling moisture, mix temperature and plant production was not problematic.

Asphalt Plant Gradation Control

The gradation of the mix was found to be shifted towards the fine side due to the tumbling of the aggregate particles in the dryer drum. To compensate for the expected increase in fines, the unwashed manufactured fines were substituted with an equal volume of grate ash. Although the increase in fines exceeded the amount reduced by eliminating the manufactured fines, this was not felt to be problematic. Under full production because the amount of conventional fines could be further reduced to the point the amount of reduced manufactured fines normally added to a mix would equal the amount of fines produced in the drying and mixing process.

The batch plant used to produce the mixes used an auger to transfer the fines created in the drying process and collected in the air pollution system to the sieves located above the aggregate bins. The gap between the auger and its casing compounded with the lightweight grate ash particles made the transfer very difficult and

the small lightweight fines accumulated in the lower part of the auger. The accumulated fines were removed with the assistance of a vacuum truck. A total volume of approximately one cubic meter of fines was collected and disposed of at the landfill during the production of the grate ash mix.

Production of fines while drying and mixing can easily be controlled by adjustment of the natural fines in the job mix. An equal amount of natural fines could be removed so the gradation could be achieved after drying and mixing rather than before the aggregates are dried.

Transfer of the fines within the plant can be accomplished by redesigning the mechanical transfer system or by completely eliminating the need for transferring the fines by the use of a drum mixing plant. One way or the other the generation of fines in the production of grate ash is not problematic.

Compaction Testing Results

Table 5.1 shows the density and percent compaction of the Northbound and Southbound lanes of the control and test sections. The compaction is based on the densities of the pucks fabricated from the mix samples taken at the plant during paving. The average percent compaction for the Northbound and Southbound lanes of the control and test pavement were 92.5 and 92.2%, and 89.8 and 90.8% respectively.

The test pavement was placed immediately after the completion of the control. The test mix was very similar to the control in terms of placement and compaction properties. Several additional passes of the heavy three wheeled steel roller were required to obtain compaction, presumably due to the increased angularity of the grate ash. Both binder mixes were placed without any problems and obtained compaction with

Table 5.1: Field Density and Compaction Control Data

Control Section					Test Section				
Station	Density ¹		Compaction ²		Station	Density ¹		Compaction ⁵	
	Northbound	Southbound	Northbound	Southbound		Northbound	Southbound	Northbound	Southbound
170+00 ³	137.5		90.9	90.9	159+50 ⁶	132.1		87.4	
169+82 ³	137.0		90.6		159+50 ⁷	139.2	137.9	92.1	91.2
168+00 ³	139.3	141.0	92.1	93.3	157+82		137.3		90.8
166+00	141.3	138.4	93.5	91.5	157+00	136.8	137.0	90.5	90.6
165+82	141.3	139.9	93.5	92.5	156+00	138.0	137.8	91.3	91.1
164+00	142.4	140.3	94.2	92.8	154+00	133.9	138.1	88.6	91.3
162+00	141.2		93.4		153+82	138.0	136.5	91.3	90.3
160+00 ⁴	139.4	139.5	92.2	92.3	152+00	132.1	136.0	87.4	89.9
Average	139.9	139.8	92.5	92.2		135.7	137.2	89.8	90.8
Stdev	2.0	1.0	1.3	0.8		3.0	0.8	2.0	0.5

Note: ¹ Based on at least 3 average readings from 2 of three test locations in the wheel paths and center of lane

² Based on plant average density of 15 pucks, 151.2 lbs/ft³

³ Triaxial roller not used

⁴ Taken in the 4" control section

⁵ Based on plant average density of 15 pucks, 143.6 lbs/ft³

⁶ Placed during heavy rain

⁷ Taken in the 4" test section

averages above 90 percent. The NHDOT does not specify a percent compaction requirement for binder courses.

Of the approximate 2,000 linear feet (0.61 km) of reconstruction, 1,150 linear feet (0.35 km) were reconstructed in the control section using natural aggregate both in the two inch binder course and the one inch wearing course. The 850 linear foot (0.26 km) test section was constructed using a 50 percent grate ash and 50 percent natural aggregate blend in the two inch binder course.

A one inch wearing course (surface course) of 100 percent natural aggregate was applied over the test and control binder courses. The compaction of the surface course was not inspected during mixing or compaction due to a misunderstanding of scheduling by the contractor. Although it most likely had little if any effect, the contractor selected a surface course which was produced in a non NHDOT approved asphalt plant located at Pike Industries Tilton, New Hampshire. The thicker four inch sections of the binder courses were placed such that their top surfaces were flush with the two inch binder courses. This assured a continuous smooth one inch layer of surface mix throughout the project. The surface mix that was placed on the Northbound lane starting approximately at the transition between the control and test sections appeared to be a tender mix and difficulty was encountered during its compaction. Tender mixes cannot be compacted in the normal manner due to their slow development of sufficient stability to withstand the weight of the compaction equipment. They exhibit high plastic distortion when they are rolled immediately after laydown. Such mixes are typically caused by poor aggregate gradation of naturally occurring sands. The effect of this on the service life is not problematic. However tender mixes can result in producing a wavy pavement surface

which, are easily detected by road performance testing devices like ARAN, but depending on the severity, may or may not be noticed by the users.

CHAPTER 6. DATA COLLECTION

Campbell System

A Campbell CR10 Measurement and Control System was utilized to monitor and record the test data. The CR10 system combines a micro-computer, clock, multimeter, calibrator, scanner, timer, frequency counter, and controller in a compact, sealed, stainless steel package. The programmable unit provides sensor measurement, time keeping, communication and data storage. Each unit is capable of storing 29,900 data points before data is lost due to overwriting of the memory. The system was programmed to collect and store data from the sensors on a routine basis. Data were downloaded by a modem attached to a telephone line placed in service at the test site.

Moisture

Moisture measurements were taken every 60 minutes by means of the TDR equipment. An eight channel multiplexer was used to sequentially switch the four control and four test moisture gauges to the TDR equipment which sent the processed data to storage within the CR10 unit.

Temperature and Resistance

Temperature and resistance were measured every 60 minutes as was the moisture. Temperatures from twenty thermocouples from each temperature probe plus the air, control box and CR10 temperatures were sequentially measured through multiplexers and stored within the CR10 memory. AC Voltages were measured between the 17 copper rings on each resistance probe at the same time the temperatures were measured. Each copper ring was sequentially electronically connected to the legs of an ac bridge

and the unbalanced voltage was measured. The next copper ring was then connected to the bridge and the unbalanced voltage was measured. This procedure was continued until the voltage was measured between each copper ring. The voltages were not processed into resistances prior to being sent to storage in the CR10 memory.

Traffic

The induction loops were connected to an electronic device which detects changes in inductance. The equipment, loaned to the project by the NHDOT Traffic Department, consisted of induction detectors normally used as devices to switch traffic lights. In this case the detectors were not used to switch on or off a traffic light, but to count the traffic. The two detectors were wired to close a circuit when a vehicle crossed over the induction loops. The continuity of the connection was sensed by the CR10 as a pulse mode. Every time a vehicle passed over the induction loop a pulse is sensed and counted by the CR10. The detectors worked very well as a traffic measuring device.

ARAN

The pavement control and test sections were evaluated for roughness by the NHDOT ARAN surface testing vehicle. The ARAN vehicle was run in the estimated wheel paths of the Northbound and Southbound lanes at approximately 40 miles/hour (64.4 km/hr) when the data were collected. The initial study was performed on May 26, 1993 and the final on June 14, 1994. Both profile studies were run by Mr. James County of the Data Collection Department of the Bureau of Transportation and Planning of the New Hampshire Department of Transportation

FWD

The testing of the pavement sections using a Falling Weight Deflectometer was done by CRREL under the direction of Dr. Vincent Janoo following SHRP procedures. The testing was done at various locations and dates as previously presented in Table 1.9. The general procedure was to determine the deflection response to the FWD loading for different conditions of the pavement as it was being constructed and for different environmental conditions after construction was completed. The first evaluation was on top of the asphalt stabilized base before the binder and surface course was applied. The second test date was the day after the control and test binder courses were applied. The third test date was June 2, 1993 after the surface course had been placed; testing on this day consisted of only the four inch sections and only over the strain gauges. The fourth test day was on November 5, 1993. The fifth test date was March 23, 1994 when the underlying materials were frozen. The sixth was April 6, 1994. The seventh test date was April 26, 1994 when the frost was gone from the pavement layers. The eighth and last test date was on October 13, 1994.

Strain

Six strain gauges, three in the control and three in the test section were electrically connected to a full bridge circuit which had three 120 ohm precision resistors attached to the non active legs. These resistors approximately balanced the 120 ohm Past II AC strain gauges manufactured in Denmark by Dynatest. The CR10 was used to monitor the bridge voltages from each gauge for evaluating the climatic effect on the two binder mixes.

The strain gauges were also used in conjunction with the Falling Weight Deflectometer to determine the peak strains caused by the impact of the standard loadings. The strain gauges were tested on June 2, 1993, November 5, 1993, March 23, 1994 and April 29, 1994.

The procedure was to place the FWD over the strain gauges and determine their response to the standard series of weight drops used in the FWD testing (9, 12, 16, and 22 kips) (4,082, 5,443, 7,258, and 9,979 kg). The strain gauges were monitored to determine the peak strain that occurred on the bottom of the test and control binders during the FWD loading.

The impulse from a loading occurs over a very short time period of approximately 0.02 to 0.035 seconds and therefore required special equipment and testing procedures to assure the data were collected. A computer data acquisition system designed to accept bursts of data at a frequency of 5000 Hz was utilized to capture the strain gauge outputs.

Even though up to 15,000 data points were taken for each weight drop it was very difficult to synchronize the dropping of the weight and the start of the data collection due to the maximum data collection period of 1.5 seconds. This resulted in lost data for some of the weight drops. Another problem encountered was the positioning of the FWD foot over the exact location of the gauges. If the FWD load was off center relative to the strain gauge, negative strains (compression) develop. Only positive strains were analyzed.

CHAPTER 7. LABORATORY TESTING OF FIELD SAMPLES

Pavement Slab and Road Core Collection

Pavement slabs were removed from the road on May 15, 1993 and July 27, 1994 to facilitate the physical testing of the test and control pavements and to evaluate monolithic leaching behavior. Figures 7.1 and 7.2 schematically depict the location of these slabs. Slabs were removed from the four inch thick binder course section in the region near the transition from the control to test sections. A large pavement saw was used in both instances to cut the pavement. This saw requires the use of water to lubricate the diamond saw blade. Slabs at $t=0$ years were stored at the Laconia Department of Public Works for six months prior to delivery to UNH. Slabs at $t=1.25$ years were immediately transported to UNH. Four inch cores were taken from the slabs using a diamond core drill. This device also requires the use of water to lubricate the core barrel. Beams for the thermal stress test were cut from the slabs with a water cooled concrete saw and transported to CRREL for testing.

Resilient Modulus and Indirect Tension

Resilient modulus of elasticity is a unique material's property which is used by pavement designers to evaluate the relative quality of a given material as well as to generate input for use in pavement evaluation and analysis. The laboratory test to determine the resilient modulus of bituminous mixes is a repeated load indirect tension test. The repeated load is a compressive load applied to the sample with a haversine waveform. The load is applied vertically to the vertical diameter plane of the four inch cylindrical specimen. The resulting lateral horizontal deformation of the specimen is

Figure 7.1: Schematic Showing Location of Test and Control Pavement Slabs Removed on May 15, 1993 ($t=0$ years)

Figure 7.2: Schematic Showing Location of Test and Control Pavement Slabs Removed on July 27, 1994 ($t=1.25$ years)

measured and with an assumed value of Poisson's ratio the resilient modulus is calculated.

Collecting the deformation data as well as its interpretation can be a challenge due to high variability caused by the testing apparatus, the unconfined test sample and variation of the material itself. AASHTO notes in its design guide that laboratory resilient modulus values can be obtained that significantly differ from what exists for an in situ condition. The resilient modulus test used in this study is referred to as the Total Resilient Modulus, which is calculated using the total recoverable deformation. This includes the instantaneous recoverable as well as the time dependent continuing recoverable deformation during the loading and unloading and rest period portion of one cycle of loading.

The test was performed on test and control field samples using a contact load of ten pounds (4.5 kg) and a cyclic load of 90 pounds (40.8 kg) at three temperatures, 14°F (-10°C), 32°F (0°C) and 77°F (25°C).

Upon completion of the resilient modulus testing the specimens were destructively tested to determine their indirect tensile strength. The indirect tension test is similar to the diametral tension test commonly used to determine the tensile strength of portland cement concrete. The stress at failure is defined as the indirect tension of the material.

Thermal Stress

The test and control binder mixes were evaluated for cracking when subjected to low temperatures. Thermal stresses are non-load associated and can cause transverse cracking of pavements if the stress exceeds the tensile strength of the pavement. Field observations have indicated that cracking starts at the top surface and progresses down

with time. Mixes which have high stiffness at low temperatures are more prone to cracking from thermal stresses. The low temperature properties of the asphalt cement are the predominant cause of thermal shrinkage and cracking. The testing procedure used was a non-standard test developed by SHRP to give an estimate of a mix's ability to withstand stress caused by low temperatures.

A specimen was prepared by cutting a two inch x two inch x 15 inch (5.1 cm x 5.1 cm x 38.1 cm) beam from the large samples removed from the pavement and then machining a dog bone shape in the middle of the beam such that the reduced section was one inch x one inch (2.5 cm x 2.5 cm).

The parallel ends of the beam were epoxied to metal plates which were attached to thermally rigid mounts inside a temperature chamber. As the chamber temperature was slowly lowered, the beam attempts to thermally contract but is unable to do so because of the end restraints. As the temperature continues to lower, the stress within the beam increases. The test continues until the beam fractures and the temperature at failure is recorded. The test has very high variation and the precision of the test method has not been established.

Marshall Stability

Marshall stability is defined as the maximum load that is required to fail a four inch diameter x 2.5 inch high (10.2 x 6.4 cm) bituminous specimen at a standard temperature of 140°F (60°C). Flow is defined as the deformation, in units of 0.01 inch (0.025 cm), that occurs when the Marshall stability specimen fails. The sample is loaded by deforming the sample at a standard deformation rate of two inches/minute (5.08

cm/min) while it is held in place with a loading collar head that fits the four inch (10.2 cm) diameter curved surface.

Marshall stability and flow were determined on the field mixes as placed and on cores taken from the large specimens cut from the test sections.

CHAPTER 8. DATA ANALYSIS AND RESULTS

Moisture

The Time Domain Reflectometer (TDR) works by polarizing the free water molecules in the material between the two stainless steel probes which, in terms of the electronic output, is a function of the volumetric water content. It is not possible to obtain correct moisture contents with the TDR when water changes phase (liquid to solid) because the molecules within the ice are not mobile and therefore can not be polarized. The net result is that when the water freezes the TDR moisture readings show an erroneous reduction in water content. The TDR moisture contents are based on volume and can be converted to approximate percent by weight by dividing by two. This assumes the material has a dry density of two times the unit weight of water, 124.8 lbs/ft³ (2.0 gm/cm³).

Moisture data were collected throughout the study except for a period when an internal TDR power source failed. In that the two test and control moisture probe assemblies were approximately 31 feet (9.4 m) apart it would be expected that moisture readings at a given depth would be approximately equal.

The volumetric moisture contents are shown on Figures 8.1 and 8.2. Data from the upper test and control section probe (at 12 inches (30.5 cm)) within the stabilized base are not shown on the Figures because each showed very high variation which was technically not possible. This in part may be responsible to the effect of the potentially polarizable hydrocarbons in the RAP used to stabilize the base. The probe located at 24 inches (61.0 cm) in the control section also showed high variation and is not included in Figure 8.2.

Figure 8.1: Moisture Contents Under the Test Pavement as a Function of Time

Figure 8.2: Moisture Contents Under the Control Pavement as a Function of Time

The volumetric moisture contents revealed several important trends; the water table was never higher than somewhere below the lower TDR probes located at four feet (1.2 m), the slowly declining moisture content shown by the three foot depth of the control prior to freezing suggests moisture migration towards the surface occurred, moisture contents increased with depth as shown on the test section, and the depth of frost exceeded four feet (1.2m) during the winter time.

Traffic

The traffic was recorded in the Northbound and Southbound lanes by the use of induction loops as previously discussed in Chapter 6. Figure 8.3 shows the cumulative traffic in both directions as a function of time. Although the amount of traffic in the Southbound direction is slightly more than the traffic in the Northbound direction the traffic data were very surprisingly constant on a daily basis over the two year study period. On several occasions data were overwritten within the memory of the Cambell Cr10 data collection system. The fact that the traffic data were so repeatable allowed the estimation of lost data by assuming it to be the same as the traffic on the same day of the previous or next week.

Figure 8.4 shows a typical traffic count as a function of day of the week. Approximately 5.5 million vehicles have traveled on the road since it was reconstructed as shown on Figure 8.5. Assuming a truck distribution of seven percent, which is typical of New Hampshire traffic, predicts the truck traffic to date is approximately 400,000 vehicles.

Figure 8.3: Cumulative Vehicles Versus Time for the Northbound and Southbound Directions

Figure 8.4: Vehicles Versus Day of the Week for the Northbound and Southbound Directions

Figure 8.5: Cumulative Vehicles Versus Time for Both Directions

ARAN

The pavement was evaluated for roughness by the NHDOT ARAN surface testing vehicle. The ARAN was run at approximately 40 miles/hour (64.4 km/hour) while collecting the data. The vehicle was run in the estimated wheel paths of the Northbound and Southbound lanes. The initial study was performed on May 26, 1993 and the final on June 14, 1994. Both profile studies were run by James County of the Data Collection Department of the Bureau of Transportation and Planning of the New Hampshire Department of Transportation.

Table 8.1 presents the average profile data for the two test years. These data consist of an average of three runs for the 1993 test date and four runs for the 1994 test dates. These data represent roughness as measured by the International Roughness Index (IRI) with units of inches/mile. The ARAN vehicle has sensors installed such that it is capable of measuring the longitudinal profile of the pavement's wheel paths. The data presented are an average of the two wheel paths as a function of station along the test pavement. Stationing runs from 150 + 25 on the North end of the project to 170 + 15 on the south end. The transition between test and control sections is at 159 + 82.

The data were statistically analyzed to determine if the IRI numbers were increasing with time. A paired observation statistical analysis showed no difference between the two test years at the 95% level of confidence for both directions of the control and the Northbound direction of the test pavement. The grate ash test pavement had higher IRI numbers than the control pavement (165 versus 128 for 1993 and 175 versus 130 for 1994 for the Northbound test and control respectively). The IRI values are in general high for a new pavement facility. This is most likely due to the porpoising

Table 8.1: International Roughness Index for Years 1993 and 1994

Test							Control						
Northbound			Southbound ¹				Northbound			Southbound			
Station	93	94	93-94	93	94	93-94	Station	93	94	93-94	93	94	93-94
15.25				82	120	-38	16.04	201	229	-29	69	107	-38.58
15.31	63	96	-33	67	101	-34	16.10	227	229	-2	164	103	60.23
15.36	139	128	10	87	131	-44	16.15	136	137	-1	86	89	-2.60
15.41	101	99	2	80	116	-36	16.20	102	112	-9	81	88	-7
15.46	192	174	17	113	168	-54	16.26	65	64	1	67	102	-35
15.52	231	258	-27	78	105	-26	16.31	104	112	-9	116	83	33
15.57	152	201	-49	101	108	-7	16.36	93	109	-16	226	89	138
15.62	194	207	-12	82	92	-11	16.41	51	56	-5	167	126	41
15.68	128	151	-23	48	94	-46	16.47	166	132	34	109	121	-12
15.73	119	145	-26	101	103	-3	16.52	127	186	-59	87	121	-35
15.78	122	120	2	93	108	-15	16.57	176	194	-18	238	112	126
15.83	113	148	-35	137	195	-58	16.63	61	88	-27	136	120	16
15.89	241	204	37	88	152	-65	16.68	171	133	39	138	132	6
15.94	278	280	-2	133	158	-25	16.73	96	110	-14	172	129	44
15.99	239	240	-1	130	147	-18	16.78	133	149	-16	116	110	6
							16.84	122	110	13	49	106	-57
							16.89	63	53	11	66	112	-47
							16.94	103	90	13	44	73	-30
							17.00	77	72	6	94	81	131
							17.05	96	91	5		107	
							17.10	163	143	20		111	
							17.15	270	261	9			
Average	165	175	-10	95	127	-32 ¹		128	130	-2	117	106	12
Stdev	64	58	24	25	31	19		57	58	22	56	17	53

Note: ¹ Significantly different at the 95 % confidence level

of the short base paver used to spread the HMA on the project, and on the test Northbound lane, the presence of a tender wearing course mix.

Figure 8.6 shows the Northbound IRI values for the two test years as a function of stationing. Although the values are very variable as a function of the stationing, the two data points (1993 versus 1994) are remarkably the same. There are a few points which show the roughness to be less during 1994 as compared to 1993. This implies the pavement became smoother after one year of traffic which is not likely. Such variation is most probably within the measuring accuracy of the ARAN vehicle.

The Southbound IRI data are similarly shown by Figure 8.7. These data are more consistent and of lower magnitude than the Northbound data. As observed with the Northbound data there are also several points which show the roughness to decrease with time on the control section.

Table 8.2 presents the PSI data calculated from the IRI data. The relationship between the IRI and PSI comes from an equation obtained by normalizing all the profile data collected by the Data Collection Department of the Bureau of Transportation and Planning of the New Hampshire Department of Transportation for a given time period. The normalized equation used to calculate the present serviceability index was

$$\text{PSI} = 4.55 - 1.5 \log (0.221 \times \text{IRI} - 10).$$

The PSI data presented in Table 8.2 are plotted in Figure 8.8 for the Northbound direction and in Figure 8.9 for the Southbound direction. Statistically the PSI data follow the same trends as shown by the IRI values i.e. no statistical difference at the 95 percent level between the two test years at the 95% level of confidence for both directions of the control and the Northbound lane of the test. As with the IRI data

Figure 8.6: Average Northbound IRI Versus Station for 1993 and 1994

Figure 8.7: Average Southbound IRI Versus Station for 1993 and 1994

Table 8.2: Present Serviceability Index for Years 1993 and 1994

Test							Control						
Northbound							Southbound						
Station	93	94	93-94	93	94	93-94 ¹	Station	93	94	93-94	93	94	93-94
15.25				3.18	2.72		16.04	2.25	2.14	0.11	3.48	2.85	0.63
15.31	3.67	2.98	0.69	3.54	2.91	0.63	16.10	2.14	2.14	0.01	2.42	2.89	-0.46
15.36	2.58	2.65	-0.08	3.10	2.63	0.47	16.15	2.60	2.59	0.01	3.11	3.07	0.04
15.41	2.91	2.94	-0.02	3.22	2.76	0.46	16.20	2.90	2.80	0.10	3.20	3.09	0.11
15.46	2.29	2.37	-0.08	2.78	2.40	0.38	16.26	3.58	3.61	-0.03	3.54	2.90	0.63
15.52	2.13	2.04	0.09	3.26	2.87	0.38	16.31	2.88	2.79	0.09	2.78	3.16	-0.41
15.57	2.49	2.25	0.24	2.91	2.84	0.07	16.36	3.02	2.83	0.19	2.15	3.07	-0.93
15.62	2.27	2.22	0.05	3.19	3.02	0.17	16.41	4.39	4.01	0.38	2.41	2.67	-0.26
15.68	2.66	2.50	0.16	4.94	3.01	1.93	16.47	2.41	2.63	-0.22	2.83	2.72	0.11
15.73	2.74	2.53	0.20	2.92	2.89	0.03	16.52	2.66	2.31	0.35	3.11	2.71	0.40
15.78	2.71	2.72	-0.01	3.02	2.84	0.18	16.57	2.36	2.27	0.09	2.11	2.80	-0.69
15.83	2.78	2.52	0.27	2.59	2.27	0.32	16.63	3.74	3.09	0.65	2.60	2.72	-0.12
15.89	2.10	2.23	-0.14	3.09	2.49	0.60	16.68	2.38	2.62	-0.24	2.58	2.63	-0.04
15.94	1.98	1.98	0.00	2.62	2.45	0.16	16.73	2.97	2.82	0.15	2.38	2.65	-0.27
15.99	2.10	2.10	0.00	2.64	2.52	0.12	16.78	2.62	2.51	0.11	2.76	2.82	-0.06
							16.84	2.70	2.82	-0.12	4.69	2.86	1.83
							16.89	3.64	4.24	-0.59	3.57	2.80	0.78
							16.94	2.89	3.05	-0.16		3.36	
							17.00	3.27	3.40	-0.13	3.01	3.21	-0.20
							17.05	2.98	3.05	-0.07		2.85	
							17.10	2.43	2.55	-0.12		2.81	
							17.15	2.01	2.03	-0.03			
Average	2.53	2.43	0.10	3.13	2.71	0.42 ¹		2.86	2.83	0.02	2.93	2.89	0.06
Stdev	0.44	0.32	0.21	0.59	0.24	0.47		0.58	0.57	0.26	0.63	0.20	0.64

Note: ¹ Significantly different at the 95 % confidence level

Figure 8.8: Average Northbound PSI Versus Station for 1993 and 1994

Figure 8.9: Average Southbound PSI Versus Station for 1993 and 1994

those which show increased values with time are most likely within the limits of the variation of the ARAN test equipment.

FWD

The FWD data were analyzed to evaluate the characteristic elastic properties of the test and control pavement components and to evaluate the response of the two binder courses and the other pavement components due to seasonal temperature changes. Although FWD testing was done on the two inch binder sections for general observation of the pavements and their components, only the four inch thick binder sections were selected for detailed analysis of the binder's elastic properties. Only the thicker sections were evaluated because the FWD technique of estimating elastic properties is sensitive to thickness of surface courses. The effects of thin courses are not easily determined by FWD techniques.

Data Processing

There are a number of ways to process FWD data to determine normalized deflections as a function of offset distance. The procedure used in this research is a statistically based approach. The FWD data were processed to reduce random error prior to back calculating by statistically fitting the data using best fit least square linear techniques. A best fit equation was determined for each of the six offset deflections based on 16 data points (four replicates of four loads).

The raw FWD data, the statistical regression coefficients R^2 values, and predicted deflections for the normalized nine kip load (4,082 kg) for the October, 1994 test station 159 + 62 Northbound center of the lane data are presented in Table 8.3. These data

Table 8.3: FWD Data for Test Station 159 + 62 North Bound Center Lane
October 1994

Raw FWD Data							
Load, lbs	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
Deflections, 0.001 inches							
6,607	6.35	5.32	4.61	3.67	2.96	2.01	1.06
6,639	6.30	5.31	4.59	3.67	2.95	2.00	1.07
6,623	6.28	5.32	4.60	3.68	2.97	2.00	1.06
6,650	6.30	5.31	4.59	3.68	2.96	2.01	1.07
8,668	8.54	7.16	6.19	5.00	4.00	2.71	1.48
8,676	8.51	7.15	6.19	4.98	4.02	2.71	1.46
8,684	8.50	7.14	6.15	4.98	4.01	2.70	1.46
8,676	8.52	7.14	6.19	4.99	4.01	2.72	1.47
11,390	11.36	9.50	8.21	6.65	5.34	3.61	1.96
11,255	11.35	9.48	8.27	6.59	5.33	3.61	1.94
11,255	11.31	9.43	8.17	6.60	5.33	3.61	1.96
11,298	11.30	9.44	8.17	6.59	5.34	3.60	1.94
15,331	15.54	13.02	11.30	9.11	7.33	4.93	2.67
15,254	15.41	12.95	11.25	9.04	7.31	4.96	2.70
15,310	15.41	12.93	11.26	9.03	7.38	4.96	2.69
15,283	15.37	12.89	11.20	9.04	7.30	4.95	2.71
Statistical Regression Data $\text{deflection} = a + b \cdot \text{Load}$							
a	-0.6306	-0.5019	-0.4852	-0.4104	-0.3699	-0.2406	-0.1670
b	0.00105	0.00088	0.00077	0.00062	0.00050	0.00034	0.00019
R ²	0.99969	0.99986	0.99977	0.99985	0.99983	0.99977	0.99946
FWD Deflections for Normalized 9,000 Pound Load							
Load	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
Deflections, 0.001 inches							
9000	8.84	7.42	6.43	5.17	4.17	2.82	1.52

are typical of all the FWD data and are presented here to show how the normalized data were determined. The R^2 values for the regression equations typically exceeded 0.999 for all the FWD data. Figure 8.10 shows the FWD deflections as a function of load for the data of Table 8.3. Each offset point consists of four observations as shown on Figure 8.10. Figure 8.11 shows the statistically predicted deflections for a normalized nine kip (4,082 kg) load as a function of offset distances. These data are the predicted deflections from each of the regression equations for the normalized nine kip load. The procedure was repeated for each FWD set of data to obtain the normalized data used in the back calculating of elastic moduli and layer thickness.

Back Calculations to Determine Elastic Properties

The procedure of back calculating starts with the outer deflection information. As previously discussed, the outer deflections are related only to the lower subgrade layer. This allows the estimation of E_s , the stiffness moduli of the subgrade. The deflection at the intersection of the stress zone at the surface/base interface are related to the elastic moduli of the base, E_b , subbase, E_{sb} , and E_s . Using the estimated E_s allows the estimation of the moduli of the base and subbase. The solution always initiates at the outer-most deflection and proceeds inward towards the load using the estimated moduli and layer thickness determined in the process. The process becomes more reliable with increasing knowledge of the actual field conditions which exist at that location.

Deflections

Deflections were determined at offsets 0, 8, 12, 18, 24, 36 and 60 inches (0, 20.3, 30.5, 45.7, 61, 91.4 and 152.4 cm) from the centerline of the 5.91 inch (15 cm) radius

Figure 8.10: FWD Peak Deflections for Test Station 159 + 62 North Bound Center of Lane

Figure 8.11: Peak Deflections for 9000 Pound Normalized Load

FWD loading foot. Typical peak deflections for the normalized nine kip load as a function of offset distances are shown on Figure 8.11.

Asphalt Stabilized Base Deflections

FWD analysis was conducted on top of the stabilized base prior to paving so the deflection properties of the layers below the HMA could be quantified. Figures 8.12 and 8.13 show the Northbound and Southbound offset deflections as a function of station for the asphalt stabilized base. The top line on the Figure represents D_0 , the maximum deflection that occurs under the center of the load. The other lines represent the other offsets down to the last one at 60 inches (152.4 cm) from the center of the load.

The stiffness of the stabilized base and all underlying materials affects the shape of the deflection profiles. If for instance ledge was very close to the surface of the base, the deflections would be greatly reduced. On the other hand the deflections would be expected to increase if the underlying materials were soft as in a clay.

Appendix I and II contain the FWD deflection data used to evaluate the stiffness and layer coefficients of the test and control pavement sections. The data show the deflections vary point by point but in general are somewhat equal except for possible softness in the upper layers of the Northbound lane starting at about station 164 + 00 in the control section. It was also noted that the 60 inch (152.4 cm) offset deflection increases slightly for both directions as the station number increases. This suggests that the subgrade modulus decreases with increasing station number going towards the control section.

Figure 8.12: Northbound Stabilized Base Deflection Profiles

Figure 8.13: Southbound Stabilized Base Deflection Profiles

Binder Course Deflections

FWD analysis was done on top of the binder courses the next day after it was paved on May 7, 1993. The structural benefit of applying the binder courses relative to decreasing the FWD deflections is easily seen by comparing the previous asphalt stabilized base data to the new deflection profiles.

Figures 8.14 through 8.17 show the deflection profiles for the Northbound and Southbound directions for the outer wheel path and the lane centerline. The average temperature of the binder course during the testing was approximately 81°F (27°C). These data, as in the stabilized base data, show the deflections to vary point by point. Several general trends are however apparent; the maximum deflections were reduced by the application of the binder courses by about 0.05 inches (0.13 cm) for the Southbound lane and about 0.07 inches (0.18 cm) for the Northbound lane, and the Northbound lane of the control showed the lowest maximum deflections. These data are consistent with the previously discussed effect of placement of the extra 28 truck loads of RAP on the Northbound control section.

Wearing Surface Deflections

The FWD testing continued after the wearing surface was placed on May 15, 1993 over the binder courses. FWD data were collected over the strain gauges in the four inch test and control sections on June 4, 1993. A complete set of data for the whole project was collected on November 5, 1993. The pavement had an average temperature of about 41°F (5°C) when the November data were collected. The effect of reducing the temperature from summer to fall was estimated by comparing the May 8, 1993 data with the November 5, 1993 data. The seasonal effect of spring thaw on the pavement was

Figure 8.14: Northbound Binder Course Wheel Path Deflection Profiles

Figure 8.15: Northbound Binder Course Centerline Deflection Profiles

Figure 8.16: Southbound Binder Course Wheel Path Deflection Profiles

Figure 8.17: Southbound Binder Course Centerline Deflection Profiles

evaluated by comparing deflection data obtained during March, 1994 with data obtained during November 5, 1993. Normalized deflections for all test data are shown in Appendix II.

Seasonal Temperature Effects

Comparing the May 8, 1993 deflection data with November 5, 1993 data shows the significant effect of lowering the temperature of a viscoelastic material. These data when compared to the previous profiles almost appear to be a different pavement until the effect of temperature and aging is considered. The deflections were significantly reduced because the temperature dropped from approximately 81°F (27°C) to 41°F (5°C). Another well established physical effect which affects the stiffness of HMA is aging of the asphalt cement. Asphalt cement is very soft when first paved and begins to stiffen as the small, more active hydrocarbons, evaporate and the remaining hydrocarbons oxidize. The magnitude of changes in stiffness that occur over several months due to aging is not significant relative to the change caused by lowering of the temperature.

The effect of spring thaw was evaluated the spring of 1994. The temperature of the pavement on March 16, 1994 varied from slightly above 32°F (0°C) in the asphalt to below freezing in part of the underling layers. Depth of frost ranged from a depth of about 24 inches (61 cm) to 48 inches (122 cm). Figures 8.18 through 8.25 show the deflection profiles obtained on top of the final pavement surface for the Northbound and Southbound directions for the wheel path and centerline of the lane for November 4, 1993 and March 25, 1994.

Comparing the deflections of November and March very quickly shows the seasonal effect to be very significant for some sections of the pavement. The "Spring

Figure 8.18: Northbound Surface Wheel Path Deflection Profiles for November 4, 1993

Figure 8.19: Northbound Surface Wheel Path Deflection Profiles for March 16, 1994

Figure 8.20: Northbound Surface Centerline Deflection Profiles for November 4, 1993

Figure 8.21: Northbound Surface Centerline Deflection Profiles for March 16, 1994

Figure 8.22: Southbound Surface Wheel Path Deflection Profiles for November 4, 1993

Figure 8.23: Southbound Surface Wheel Path Deflection Profiles for March 16, 1994

Figure 8.24: Southbound Surface Centerline Deflection Profiles for November 4, 1993

Figure 8.25: Southbound Surface Centerline Deflection Profiles for March 16, 1994

Thaw” in general is a very challenging time period for all flexible pavements due to increased stress. These stresses are caused by the tremendous reduction of the physical properties in the underlying layers due to increased moisture contents upon melting of ice in the subsurface layers. The deflections in general increased for all stations during the March testing. The North portion of both lanes of the test section showed in some cases a doubling of the deflections. The four inch sections showed very little increase in deflections, thus showing the direct benefit of increased binder thickness. Although the frost was not totally out of the pavement on the March test date, the weakening effect of the “Spring Thaw” was very apparent.

Data Analysis

A statistical technique was utilized to select FWD stations which were to be analyzed for layer coefficients. The pavement stations selected were in the four inch test and control sections. These four inch sections were designed for the FWD analysis as thin pavements are not easily evaluated by FWD techniques. Stations in the instrumentation areas and those close to the transition between the two inch and four inch sections were not analyzed.

The station selection technique consisted of selecting FWD stations which yielded deflection parameters which fell within one standard deviation of the parameters average for the May 6, 1993 FWD data. These FWD data were obtained on top of the stabilized base. The rationale of selecting stations to analyze for the layer coefficient was based on the assumption of uniformity. It was assumed that if the deflections on the base prior to paving were reasonably uniform, then the difference in deflections after the binders were paved would be due to the binders and not some unknown subsurface effect. Three

deflection parameters were investigated and those which showed uniformity as being within one standard deviation of the average were selected for FWD analysis.

After the stations were selected, FWD test data obtained on November 5, 1993, and October 5, 1994 were chosen for detailed analysis of the binder's elastic properties. These test dates were selected for detailed analysis because pavement temperatures were very similar and the effect of one year of traffic could be investigated.

The deflection parameters used to select the FWD stations for data analysis were; (i) deflection area parameter, (ii) the deflection basin area and (iii) the impulse stiffness modulus (ISM).

Deflection Area Parameter

The area parameter is one method which calculates the response of the pavement system to a load. It is an area value that is normalized to the deflection at an offset of zero inches and is calculated using the following equation:

$$A = 6(D_0 + 2D_1 + 2D_2 + D_3)/D_0$$

where:

D_0 = deflection at an offset of zero inches (0 cm)

D_1 = deflection at an offset of 12 inches (30.5 cm)

D_2 = deflection at an offset of 24 inches (61.0 cm)

D_3 = deflection at an offset of 36 inches (91.4 cm).

This equation gives the area of a slice of the deflection basin between an offset of zero inches and 36.0 inches (91.4 cm). The May 5, 1993 asphalt stabilized base data were normalized to a 9000 pound axle load and the D_0 , D_1 , D_2 and D_3 deflections were obtained (see Table 8.3 for typical values of D_i). The area parameters were then

calculated for each FWD station within the thickened sections and the results plotted as a function of station. Figures 8.26 and 8.27, Northbound and Southbound respectively, show the plotted data with one standard deviation. The stations of the area parameter which fall within $\pm$ one standard deviation were selected as potential candidates for analysis.

Impulse Stiffness Modulus

The ISM number is similar to a spring constant which is calculated by dividing the normalized load by deflection D_0 , the zero inch offset. As an example (see Table 8.3), the approximate value of ISM for Figure 8.11 is:

$$\begin{aligned}\text{ISM} &= 9,000 \text{ pounds} / 8.84 \times 10^{-3} \text{ inches} (4,082.4 \text{ kg} / 0.022 \text{ cm}) \\ &= 1.018 \times 10^6 \text{ pounds per inch} (0.182 \text{ Mg/cm}).\end{aligned}$$

These data were then plotted as a function of stationing showing one standard deviation. As with the area parameter, the points that fell within one standard deviation were chosen for analysis. The data for the Northbound and Southbound lanes are shown in Figures 8.28 and 8.29 respectively.

Basin Area

The area above the deflection profile, which is created by the deflection readings of the seven sensors on the FWD, is called the basin area. Basin areas can show the effect of moisture, temperature, and different subgrade materials on the pavement system. The basin area for a given location decreases with increased pavement stiffness, such as caused by the ground freezing, and decreases with a reduced stiffness, such as caused by a spring thaw. The basin area was used, as with the two other deflection

Figure 8.26: Area Parameter for Northbound Lane

Figure 8.27: Area Parameter for Southbound Lane

Figure 8.28: Impulse Stiffness Modulus Number for Northbound Lane

Figure 8.29: Impulse Stiffness Modulus Number for Southbound Lane

parameters, to determine stations that were similar. The basin area can also be calculated by finding the area above the FWD deflection profile. The approximate basin area of Figure 8.11 is:

$$\text{Basin Area} = 209 \times 10^{-3} \text{ in}^2 (1.348 \text{ cm}^2).$$

The data were plotted and those that were within one standard deviation were chosen as points for back calculation. Figures 8.30 and 8.31 show the stations in the four inch section which were used for analysis.

Binder Modulus

Independent analysis of the three deflection parameters led to the selection of the stations which were used to estimate the layer coefficients. A list of the selected Northbound and Southbound stations are as follows.

Northbound	Southbound
159+12	159+22
159+22	159+32
159+32	159+42
159+42	159+62
159+62	159+72
160+12	160+22
160+22	160+42
160+52	160+52
160+62	

Back Calculation

A program called WESDEF, developed by the Army Corps of Engineers at the Waterways Experiment Station, was used to estimate the elastic properties of the pavement materials. This program uses load and deflection information from the

Figure 8.30: Basin Area for Northbound Lane

8.31: Basin Area for Southbound Lane

normalized FWD testing and layer information to calculate the modulus values of the layers in question.

The WESDEF program uses inputted values for deflection, thickness and type of layers. It requires “seeding” values for the modulus of the layers it is to calculate.

Modulus seed values of the HMA were estimated using actual field pavement temperatures and an equation from AASHTO. Temperature readings of the surface, 1, 3, and 5 inch (2.5, 7.5, and 12.5 cm) depths were used to calculate an average binder temperature. This average was then used to obtain a theoretical modulus value for the binder using the following AASHTO equation:

$$E = 15,000 - 7,900 * \text{Log } T(\text{in } ^\circ\text{C}) \text{ MPa.}$$

The E value based on the average binder temperature was used to initiate the iterative calculation process.

Upper and lower limits must also be specified. Pavement input layer thickness for the wearing and binder courses was five inches. The asphalt stabilized base thicknesses were seven inches for the Southbound lane and nine inches for the Northbound lane. The subbase gravel layer was inputted as 30 inches (76.2 cm). These layers were assumed to be resting on an infinite layer.

WESDEF calculates modulus values for any particular layer in question and in the process produces an error term. The error term is the result of a comparison of the calculated deflection basin versus the actual deflection basin from the FWD analysis. The absolute sum of the percentage differences at each transducer location 0.0, 8.0, 12.0, 18.0, 24, 36, and 60 inches (0, 20.3, 30.5, 45.7, 61.0, 91.4. and 152.4 cm) is the error term. An acceptable error term is commonly limited to two times the number of FWD sensors which is 14.0 percent for the FWD used in this study.

Two test dates, November 5, 1993, and October 13, 1994, at the stations listed above were used to estimate the HMA elastic moduli.

Back calculation was performed at each selected station keeping the layer thickness constant, while the seed values were changed. This process was repeated until the lowest possible error was obtained. Table 8.4 show the error term and modulus values that were obtained from the WESDEF back calculation program. The typical error term was much less than the 2% per transducer commonly used (total error was 14 percent).

The binder moduli for the test and control sections decreased significantly from November 5, 1993 to October 13, 1994. This could be due to many different factors, most of which involve the materials in the base, subbase, and subgrade materials.

Asphalt stabilized base material increases its strength and stiffness with time due to the effect of traffic. As per the WESDEF model, an increase in the stiffness of the base would reduce the modulus value calculated for the asphaltic concrete. Another factor which could explain a decrease is in the subbase materials which are very hard to characterize as stated by Frazier Parker, Jr. (Parker, Jr., 1991) in a recent Transportation Record paper. This is due to the fact that subgrade materials are never in a state of moisture equilibrium due to the elements. It is also difficult to determine the exact in situ layer material properties, particularly the thickness. A variation in layer thickness alone can cause the estimated layer properties above it to be either too stiff or too soft.

The low error terms obtained in the analysis strongly suggest that the back calculated moduli are correct and therefore did change after one year of traffic. What ever the cause of the decreased binder moduli, it is independent of the binder materials.

Table 8.4: Modulus Values by WESDEF Back Calculation

BACK CALCULATION - MODULUS VALUES					
SOUTHBOUND - NOVEMBER 5, 1993			SOUTHBOUND - OCTOBER 13, 1994		
STATION	Error	E	STATION	Error	E
TEST			TEST		
159.22	3.5	821000	159.22	9.7	980000
159.32	3.0	770000	159.32	13.9	807000
159.42	3.0	873000	159.42	9.8	406000
159.62	2.0	1089000	159.62	7.1	1792000
159.72	3.0	987000	159.72	11.6	807000
AVE.		933000 ¹	AVE.		933000 ¹
NORTHBOUND - NOVEMBER 5, 1993			NORTHBOUND - OCTOBER 13, 1994		
STATION	Error	E	STATION	Error	E
TEST			TEST		
159.12	1.7	1290000	159.12	8.7	596000
159.22	4.0	1000000	159.22	9.0	697000
159.32	5.0	1410000	159.32	14.0	837000
159.42	3.9	1240000	159.42	6.7	773000
159.62	1.2	1300000	159.62	8.1	528000
AVE.		1248000 [*]	AVE.		686000 [*]
SOUTHBOUND - NOVEMBER 5, 1993			SOUTHBOUND - OCTOBER 13, 1994		
STATION	Error	E	STATION	Error	E
CONTROL			CONTROL		
160.22	1.6	1112000	160.22	7.8	457000
160.42	1.4	1482000	160.42	5.5	526000
160.52	5.1	1036000	160.52	4.8	648000
AVE.		1210000 [*]	AVE.		544000 [*]
NORTHBOUND - NOVEMBER 5, 1993			NORTHBOUND - OCTOBER 13, 1994		
STATION	Error	E	STATION	Error	E
CONTROL			CONTROL		
160.12	2.6	1400000	160.12	7.7	689000
160.22	3.2	1050000	160.22	7.8	673000
160.52	3.2	835000	160.52	9.4	431000
160.62	2.3	667000	160.62	9.3	470000
AVE.		988000 [*]	AVE.		566000 [*]

*Statistically significant with time at the 95% confidence level.

¹No difference with time at 95% confidence interval.

Based on the available information it appears the decreased moduli are due to the increase in the stiffness of the asphalt stabilized base.

Layer Coefficients

In designing a pavement by the AASHTO method, a value called the structural number is calculated based on soil conditions, traffic, and regional factors. This structural number is calculated using the following equation:

$$SN = a_1D_1 + a_2D_2 + a_3D_3$$

where a_x = layer coefficient and D_x = layer thickness. In design, the structural number of the subgrade is known as well as the layer coefficients. The equation is then solved for layer thickness needed to obtain the structural number. The object of the FWD investigation was to define the layer coefficients for the test and control binders. This was achieved by using two methods: Rhode's method and the AASHTO design method.

Rhode's Method

The Rhode's method (Rhode 1995) is a way to determine the structural number of asphaltic concrete by using FWD testing. This technique was developed as a way to determine the structural number of the pavement without backcalculation of the material beneath the pavement. Rhode's method was developed based on the ideal situation that a pavement behaves as a linear elastic system. It is well known that a pavement system has many discontinuities and complex conditions that challenge this assumption.

To validate his work, Rhode ran tests on 7,776 different combinations of layer thickness in order to develop the relationship between the structural number (SN) and the structural index of the pavement (SIP). This research led to the development of the

equation for the structural number. In order to test this method on actual pavement systems, FWD testing was done on 10 different sections of road in Texas and 52 different roads in South Africa. The equation was used to determine the structural number of the pavement at these sites. To verify these numbers, several standard methods were used to calculate the structural number: (i) AASHTO NDT Method I which involves backcalculation of the layer moduli, (ii) DCP (Direct Cone Penetration) analysis, and the (iii) AASHTO NDT Method II which uses surface deflections. Rhode's method correlates well ($r^2 = 0.928$) with AASHTO NDT Method I as this method accounts for the complexities of the entire system. However, the DCP ($r^2 = 0.684$) and AASHTO NDT Method II ($r^2 = 0.841$) did not. This is due to the fact that the DCP analysis considers a layer's shear strength which is not characteristic of the entire layered system's elastic response, and the AASHTO NDT Method II does not consider the nonlinear response of subgrade layers or rigid layers that lie below. The excellent correlation that this method has with backcalculation makes this method very appealing for this project as the subsurface materials were not known accurately at the Laconia site.

The procedure is first to convert the FWD deflections to a standard temperature of 68°F (20°C) by using Figure 5.6 from the 1993 AASHTO Guide for Design of Pavement Structures. Once the temperature adjusted deflections are known, the following equation is used to calculate the structural number:

$$SN = k_1 S I P^{k_2} H_p^{k_3}$$

where:

k_1, k_2, k_3 = coefficients based on pavement thickness

H_p = total pavement thickness (base and pavement)

SIP = structural index of the pavement

$$= D_0 - D_{1.5Hp}$$

where:

D_0 = peak deflection under a 9000 pound (4090 kg) load

$D_{1.5Hp}$ = surface deflection measured at 1.5 times H_p under a 9000 pound (4090 kg) load which is obtained by interpolation of the three sensors closest to 1.5 H_p .

Then by knowing the structural number, the layer coefficient is determined by dividing by the thickness of the asphaltic concrete layer. Table 8.5 shows the layer coefficients calculated for the stations that were previously chosen. From the table it can be seen that there is a difference between the test section from test date to test date, but not in the control. These differences are consistent with the binder moduli changes as previously discussed. The values calculated by Rhode's method are greater than the NHDOT accepted maximum value of 0.38 for all HMA binder courses.

AASHTO Design Method

The second method for determining the layer coefficients is to use Figure 2.5 from AASHTO Guide for Design of Pavement Structures. This figure uses the back calculated resilient moduli to determine the layer coefficients. Table 8.6 shows the layer coefficients based on this procedure. AASHTO limits the layer coefficient to a maximum of 0.44. In that all the moduli were high the conversion to the layer coefficient resulted in values of the maximum for all stations of the test and control. It can be seen from the table that, in terms of layer coefficients, there is no difference between the test and control section as the value is 0.44 for all test points. As with the

Table 8.5: Layer Coefficient by Rhode's Method

LAYER COEFFICIENTS - RHODE'S METHOD					
SOUTHBOUND - NOVEMBER 5, 1993			SOUTHBOUND - OCTOBER 13, 1994		
STATION	SN	Coef.	STATION	SN	Coef.
TEST			TEST		
159.22	1.78	0.36	159.22	1.97	0.39
159.32	1.76	0.35	159.32	1.89	0.38
159.42	1.79	0.36	159.42	1.80	0.36
159.62	1.89	0.38	159.62	2.09	0.42
159.72	1.82	0.36	159.72	1.94	0.39
Ave.		0.36*	Ave.		0.39*
NORTHBOUND - NOVEMBER 5, 1993			NORTHBOUND - OCTOBER 13, 1994		
STATION	SN	Coef.	STATION	SN	Coef.
TEST			TEST		
159.12	2.21	0.44	159.12	2.00	0.40
159.22	2.01	0.40	159.22	1.87	0.37
159.32	2.14	0.43	159.32	1.93	0.39
159.42	2.12	0.42	159.42	1.88	0.38
159.62	2.12	0.42	159.62	1.81	0.36
Ave.		0.42*	Ave.		0.38*
SOUTHBOUND - NOVEMBER 5, 1993			SOUTHBOUND - OCTOBER 13, 1994		
STATION	SN	Coef.	STATION	SN	Coef.
CONTROL			CONTROL		
160.22	2.15	0.43	160.22	2.26	0.45
160.42	2.46	0.49	160.42	2.14	0.43
160.52	2.39	0.48	160.52	2.41	0.48
Ave.		0.46 ¹	Ave.		0.46 ¹
NORTHBOUND - NOVEMBER 5, 1993			NORTHBOUND - OCTOBER 13, 1994		
STATION	SN	Coef.	STATION	SN	Coef.
CONTROL			CONTROL		
160.12	2.24	0.45	160.12	1.99	0.40
160.22	2.06	0.41	160.22	1.94	0.39
160.52	1.9	0.38	160.52	1.84	0.37
160.62	1.8	0.36	160.62	1.82	0.36
Ave.		0.39 ¹	Ave.		0.39 ¹

*Statistically significant with time at the 95% confidence level.

¹No difference between Northbound and Southbound directions with time.

Table 8.6: Layer Coefficient by AASHTO Method

AASHTO METHOD - LAYER COEFFICIENTS					
SOUTHBOUND - NOVEMBER 5, 1993			SOUTHBOUND - OCTOBER 13, 1994		
STATION	E	Coef.	STATION	E	Coef.
TEST			TEST		
159.22	821000	0.44	159.22	980000	0.44
159.32	770000	0.44	159.32	807000	0.44
159.42	873000	0.44	159.42	406000	0.44
159.62	1089000	0.44	159.62	1792000	0.44
159.72	987000	0.44	159.72	807000	0.44
NORTHBOUND - NOVEMBER 5, 1993			NORTHBOUND - OCTOBER 13, 1994		
STATION	E	Coef.	STATION	E	Coef.
TEST			TEST		
159.12	1290000	0.44	159.12	596000	0.44
159.22	1000000	0.44	159.22	697000	0.44
159.32	1410000	0.44	159.32	837000	0.44
159.42	1240000	0.44	159.42	773000	0.44
159.62	1300000	0.44	159.62	528000	0.44
SOUTHBOUND - NOVEMBER 5, 1993			SOUTHBOUND - OCTOBER 13, 1994		
STATION	E	Coef.	STATION	E	Coef.
CONTROL			CONTROL		
160.22	1112000	0.44	160.22	457000	0.44
160.42	1482000	0.44	160.42	526000	0.44
160.52	1036000	0.44	160.52	648000	0.44
NORTHBOUND - NOVEMBER 5, 1993			NORTHBOUND - OCTOBER 13, 1994		
STATION	E	Coef.	STATION	E	Coef.
CONTROL			CONTROL		
160.12	1400000	0.44	160.12	689000	0.44
160.22	1050000	0.44	160.22	673000	0.44
160.52	835000	0.44	160.52	431000	0.44
160.62	667000	0.44	160.62	470000	0.44

Rhode's method the estimated layer coefficients exceed the NHDOT allowable value of 0.38, presently the maximum allowed for all HMA binder courses.

Strain Gauges

Horizontal strain at the bottom of the binder courses in the four inch sections (five inches total) were collected for control strain gauges one, five and six and test strain gauge four on all test dates except the last one on April 29, 1994 when only electronic noise was collected. The data collected for strain gauges two and three were not useable due to the FWD not being centered over the top of the gauges. The strain gauge testing procedure was identical to the standard FWD testing procedure except only three drops were made at each load level. Some data were lost due to lack of synchronization as previously discussed.

Tables 8.7 through 8.10 contain the peak strain gauge data for the four FWD test loads on the three test dates. Figures 8.32 and 8.33 show the time response of test gauge four and control gauge five for June 2, 1993. These Figures show the strain that occurred in μ (micro inches i.e. 10^{-6} in/in) as a function of time and are typical of all the strain gauge measurements. For these particular data the test peak strains are slightly higher than the control. Assuming the two sections had the same temperature the data suggest the stiffness of the control is slightly larger than that of the test section or the thickness of the test binder is less than the control.

The individual peak strain data obtained during the November testing are presented in Figures 8.34 through 8.37. The variation of the strain at constant load levels was relatively small with a maximum of less than 10μ .

Table 8.7: Strain Gauge Data for the 9 kip Load

Strain Gauge and Falling Weight Deflectometer Data

Strain Gauge		June 6, 1993			November 5, 1993			March 6, 1994			
Control #1	Load, P	9486	9538	9482	9150	9161	9118	11568	11576	11544	15400
	Strain, u	238	214	210	71	67	70	499	466	437	538
	P/u	40	44	45	129	137	130	23	25	26	29
	Avg P			9502			9143				11563
	Avg u			221			69				467
	Avg P/u, K			43			132				25
Test #4	Load, P	9641	9629	9534	9240	9309	9300	11656	11712	11744	11728
	Strain, u	163	155	149	70	69	73	227	198	269	248
	P/u	59	62	64	132	135	128	51	59	44	47
	Avg P			9601			9283				11710
	Avg u			156			71				236
	Avg P/u, K			62			132				50
Control # 5	Load, P	9439	9502	9514	9081	9110	9153				
	Strain, u	132	115	112	75	73	72				
	P/u	71	83	85	121	124	128				
	Avg P			9485			9115				
	Avg u			120			73				
	Avg P/u, K			79			124				
Control #6	Load, P				9073	9216	9137	11640	11680	11576	11624
	Strain, u				63	68	70	513	465	465	536
	P/u				145	136	130	23	25	25	22
	Avg P						9142				11630
	Avg u						67				495
	Avg P/u, K						137				24

Table 8.8: Strain Gauge Data for the 12 kip Load

Strain Gauge		June 6, 1993			November 5, 1993			March 6, 1994			
Control #1	Load, P	12502	12502	12474	12320	12367	12323	15400	15384	15320	15312
	Strain, u	298	293	294	91	99	94	538	503	476	427
	P/u	42	43	42	135	125	131	29	31	32	36
	Avg P			12493			12337				15354
	Avg u			295			95				486
	Avg P/u, K			42			130				32
Test #4	Load, P	12398	12478	12521	12375	12378	12426	15696	15664	15672	15584
	Strain, u	215	212	191	93	95	95	1359	1423	1404	1435
	P/u	58	59	66	134	130	131	12	11	11	11
	Avg P			12466			12393				15654
	Avg u			206			94				1405
	Avg P/u, K			61			132				11
Control # 5	Load, P	12537	12537	12510	12161	12185	12185	11728	15560		15504
	Strain, u	167	170	170	97	98	99	248	326		364
	P/u	75	74	73	126	124	123	47	48		43
	Avg P			12528			12177	11710			
	Avg u			169			98	236			
	Avg P/u, K			74			124	50			
Control #6	Load, P	12335	12335	12402	12371	12315	12307	11624	15504	15472	15464
	Strain, u				88	89	88	536	531	495	489
	P/u				140	138	140	22	29	31	32
	Avg P						12331	11630			
	Avg u						88	495			
	Avg P/u, K						139	24			

Table 8.9: Strain Gauge Data for the 16 kip Load
Strain Gauge and Falling Weight Deflectometer Data

Strain Gauge		June 6, 1993			November 5, 1993			March 6, 1994			
Control #1	Load, P	16232	16248	16268	16229	16248	16221	18856	18816	18856	18856
	Strain, u	418	410	405	125	119	123	468	456	459	487
	P/u	39	40	40	130	136	132	40	41	41	39
	Avg P			16249			16233				18846
	Avg u			411			122				467
	Avg P/u, K			40			133				40
Test #4	Load, P	16121		16140	16097	16189	16205	19088	19096	19072	19032
	Strain, u	264		260	121	123	125	365	444	412	424
	P/u	61		62	133	131	130	52	43	46	45
	Avg P			16131			16164				19072
	Avg u			262			123				411
	Avg P/u, K			62			131				46
Control # 5	Load, P	16244	16216	16212	16049	16030	16057				
	Strain, u	232	224	228	131	136	128				
	P/u	70	72	71	123	118	125				
	Avg P			16224			16045				
	Avg u			228			132				
	Avg P/u, K			71			122				
Control #6	Load, P	16097	16097	16113	16141	16237	16229	19000	19016	18992	
	Strain, u				110	117	112	474	387	362	
	P/u				147	139	145	40	49	53	
	Avg P						16202			19003	
	Avg u						113			408	
	Avg P/u, K						143			47

Figure 8.32: Strain Versus Time for Test Strain Gauge 4 for June 2, 1993

Figure 8.33: Strain Versus Time for Control Strain Gauge 5 for June 2, 1993

Figure 8.34: Load Versus Strain for Control Gauge 1 for November 5, 1993

Figure 8.35: Load Versus Strain for Test Gauge 4 for November 5, 1993

Figure 8.36: Load Versus Strain for Control Gauge 5 for November 5, 1993

Figure 8.37: Load Versus Strain for Control Gauge 6 for November, 1993

The averages for the different testing dates are presented by Figures 8.38 through 8.41. The response of control strain gauge one for June shows the binder to be much more flexible than the response in November. This is due to the difference in temperature and increased stiffness due to aging that occurred between the two test dates. The temperatures of the binders were approximately 25°C and 5°C when tested on June 2, 1993 and November 5, 1993 respectively.

Asphaltic materials are very sensitive to temperature and their physical properties vary by several orders of magnitude over the range of its service temperature. As temperature decreases the stiffness modulus of an asphaltic concrete increases in accordance with the thermal properties of the asphaltic cement used in the mix. At some unique low temperature the asphalt cement goes through a glass transition temperature (T_g). Above the T_g the material is flexible and behaves as a viscous nonlinear viscoelastic material and plastic flow and creep result when the material is stressed. Below the T_g the molecules are not free to move about and thermal vibrations are the only factor causing molecular rearrangement. The material also becomes hard, brittle and more resistant to shear stress. The increased stiffness, caused by lower temperatures, for a given pavement section results in less strain and increased stress.

Comparing the June and November strains as a function of load shows a nonlinear relationship. Although both temperatures are above the asphalt cement's T_g , the colder November temperature makes the mix much more rigid and inflexible. The higher June temperature allows more freedom of molecular movement under stress (plastic flow). The magnitude of the effect of plastic flow is more on control gauge one and approximately equal for test gauge four and control gauge five. Figure 8.42 shows this relationship very clearly. Gauge one, which shows much higher strains, most

Figure 8.38: Load Versus Strain for Control Strain Gauge 1

Figure 8.39: Load Versus Strain for Test Strain Gauge 4

Figure 8.40: Load Versus Strain for Control Strain Gauge 5

Figure 8.41: Load Versus Strain for Control Strain Gauge 6

Figure 8.42: Load Versus Strain for Gauges 1, 4 and 5 for June, 1993

probably has a thicker layer of asphalt cement around the strain gauge than the other two gauges and is therefore more affected by increased temperature.

To investigate the effect of load on strain the data were normalized by dividing the actual FWD average loads by their average peak strains to create a hypothetical stiffness coefficient, k , somewhat like a spring constant. These data are presented for the four FWD loads 9, 12, 16 and 22 kips (4,082, 5,443, 7,258 and 9,979 kg) in Tables 8.11 through 8.14 respectively. The units on the spring coefficient are kips per μ i.e. thousands of pounds per unit of horizontal micro strain at the bottom of the binder courses. Figure 8.43 shows the coefficient plotted as a function of the FWD load. All gauges show constant k during the November testing day and a slight decrease with increased FWD load during the June testing day. The significance of these data is that both the test and control have about the same stiffness when the asphalt cement has a low temperature 41°F (5°C). At higher temperatures constant k decreases due to the influence of plastic flow and other nonlinear properties.

Figure 8.44 shows the relationship with k relative to the March data. Although the k coefficient dropped by approximately a factor of two the test gauge four data still show a linear relationship with load. Control gauges one and six show increased stiffness with increased load. The decreased strain with increased load is the direct effect of the condition of the underlying materials.

The pavement was in a very unstable temperature state when tested on March 17, 1994. The temperature varied from around 32°F (0°C) at the surface to about 34°F (1°C) at eight inches (20.3 cm) to about +/- 32°F (0°C) at 18 inches (45.7 cm) and 48 inches (122 cm). This frozen layer, approximately 30 inches (76.2 cm) thick, would be expected to greatly increase the stiffness of the pavement and therefore reduce the strain

Figure 8.43: Stiffness Coefficient Versus Falling Weight Deflectometer Load for June and November, 1993

Figure 8.44: Stiffness Coefficient Versus Falling Weight Deflectometer Load for November, 1993 and March, 1994

measured in the binder layers. This was not the case as shown by control gauges one and six and test gauge four. The strains were drastically different than those previously encountered. This was most likely the result of the effect of the resilient modulus of the thawed layers above the frozen layer and not due to the binder and surface courses. The end result is that after the load is increased to some threshold the underlying materials are realigned and then become more capable of taking increased load with reduced displacement. Whatever the cause the binder course responds to the loss of support by increasing stress and strain.

According to AASHTO, for extreme cases like these the modulus of the frozen and thawed layers can be assumed to vary from 20,000 to 50,000 psi (137.9 to 344.8 KPa) for the frozen layers. When the layers thaw a 20 to 30 percent reduction of the normal resilient modulus is typical. These unexpected strains are the consequence of the pavements physical condition commonly referred to as the "Spring Thaw".

The November data, shown together on Figure 8.42, are all approximately equal even though the strains in June were different. The primary difference between these two test dates is decreased temperature. At the lower temperature the effect of different asphalt contents, film thickness, etc. are not as important and the two mixes approach equality. Whatever the cause, the resulting equal strains show there is little to no difference between the test and control binders.

Laboratory Data

Laboratory testing was conducted on samples removed from the test and control sections on two different dates and on samples fabricated during the paving of the

binders. The tests performed on the field samples were resilient modulus, indirect tension, thermal stress, and Marshall properties.

Resilient Modulus and Indirect Tension

The resilient modulus data are presented in Table 8.15. Standard deviation is noted to be high and increasing with decreasing temperature. This is typical of the resilient modulus test due to the difficulty in measuring small deformations at low temperatures and is not dependent upon the materials being evaluated. The average resilient modulus values are presented below.

Temperature, °C						
-10		0		25		
Test	Control	Test	Control	Test	Control	Test
Resilient Modulus	240	338	281	247	98	105
in ksi / GPa	1.66	2.33	1.94	1.70	0.67	0.72

The laboratory moduli of the two binders as these data show are much lower than those calculated by back calculation of the FWD data. Lower values are typically encountered in laboratory analysis of pavement samples due to the sample being unconfined during testing. In reality when a pavement is loaded, a triaxial state of stress is induced on the material which significantly affects the displacement and increases the stiffness. Temperature is also a major factor in the high variation of the data.

The asphalt cement becomes brittle at low temperatures and testing becomes more difficult because the deformations are very small and difficult to measure. However, at high temperatures plastic flow is dominant and the strains become very

Table 8.15: Resilient Modulus Data

Resilient Modulus ¹ , psi								
-10 °C			0 °C			25 °C ²		
Sample	Control	Test	Sample	Control	Test	Sample	Control	Test
FC 1	206,441		FC 1	367,647		FC 2	92,105	
FC 2	572,519		FC 2	114,547		FC 4	72,082	
FC 3	259,336		FC 3	392,157		FC 5	79,949	
FC 4	168,667		FC 4	125,000		FC 6	183,139	
FC 5	133,262		FC 5	203,252		FC 7	86,777	
FC 6	168,560		FC 6	180,832		FC 9	122,568	
FC 7	187,188		FC 7	222,222		FC 10	94,880	
FC 9	203,435		FC 9	398,406		FT 4		69,845
FC 10	311,634		FC 10	471,698		FT 5		104,651
FT 2		222,772	FT 4		196,580	FT 6		87,989
FT 3		330,882	FT 5		178,731	FT 7		70,470
FT 4		207,948	FT 6		179,212	FT 9		89,985
FT 5		193,632	FT 7		199,203	FT 10		111,307
FT 6		132,353	FT 9		381,680	OC 1	56,962	
FT 7		341,945	FT 10		328,947	OC 3	89,235	
FT 9		370,066	OC 3	331,125		OT 7		105,351
FT 10		468,750	OT 4		236,407	OT 8		109,756
OC 1	228,195		OT 5		404,858	OT 9		194,444
OC 2	208,720		OT 6		220,751			
OT 1		1,424,051	OT 7		220,750			
OT 2		513,699	OT 8		200,400			
OT 3		530,660	OT 9		216,920			
OT 4		439,453						
OT 5		487,013						
OT 6		489,237						
OT 7		240,055						
OT 8		198,413						
OT 9		243,056						
Average	240723	338121 ³		280689	247037		97522	104866
Standev	120010	293986		126535	78868		36707	37091

Note: ¹ Determined with a 10 pound contact pressure and a 90 pound cyclic load

² Poisson's ratio assumed to be 0.35 for 25 °C and 0.25 at 0°C and -10 °C

³ F series were removed from pavement August, 1994

O series were removed May, 1993

C Control

T test

large and measurement of deformations are much easier. This is apparent at the higher temperatures, especially at 25°C.

Figure 8.45 shows the high variation of the data at the lower temperatures.

Although the data cannot be compared directly with the FWD moduli, a comparison between the test and control may be made at each test temperature. The average values for the test and control samples are statistically equal at each temperature due to the high standard deviation of the test. The variation of the control and test are approximately equal which implies the control and test have equal resilient modulus at each of the tested temperatures.

Table 8.16 presents the indirect tensile strength data for the three test temperatures. A summary of the data is shown below.

Temperature, °C						
	-10		0		25	
Test	Control	Test	Control	Test	Control	Test
Indirect Tensile	538	483	461	448	113	118
in psi / MPa	3.71	3.33	3.18	3.09	0.78	0.81

These data show much less variation than the resilient modulus. The indirect tensile strengths are presented above with the resilient modulus data. Figure 8.46 shows the indirect tensile strength for the three tested temperatures. Again as with resilient modulus the variation is such that the strengths of the control and the test are statistically equal. The dominant effect of temperature is apparent in these data as shown by an approximate five fold increase in strength over a temperature range of only 63°F (35°C).

Figure 8.45: Resilient Modulus Versus Temperature

Table 8.16: Indirect Tensile Strength Data

Sample ¹	Indirect Tensile Strength, psi					
	- 10 oC		0 oC		25 oC	
	Control	Test	Control	Test	Control	Test
FC E	537					
FC F	588					
FT D		484				
FT E		601				
FC C			458			
FC D			500			
FT C				446		
FC A					182	
FC B					117	
FT A						129
FT B						119
Note ²						
FC X	not tested					
OC 2	489					
FT 1		439				
FT 8		441				
OT 1		497				
OT 2		427				
OT 3		493				
FC 3			514			
FC 8			373			
FT 2				397		
FT 3				411		
OT 4				481		
OT 5				478		
OT 6				473		
OC 3					85	
FC 5					92	
FC 9					96	
FC 10					106	
OT 7						130
OT 8						134
OT 9						88
FT 5						118
FT 9						111
FT 10						114
Average	538	483	461	448	113	118
Standev	50	59	63	36	36	15

Note: ¹ F series were removed from the pavement on August, 1994

O series were removed on May, 1993

C Control

T Test

² The following were obtained after the Resilient Modulus testing was completed

Figure 8.46: Indirect Tension Versus Temperature

Thermal Stress

The thermal stress data are presented in Table 8.17 and plotted on Figure 8.47. This test also has an expected high degree of variation due to the mechanical nature of the test apparatus and sample preparation. The primary intention of the test is to determine the relationship of the asphalt cement and the aggregate as affected by cold temperature. In concept the results should correlate with the properties of the asphalt cement and its film thickness coating the aggregate particles.

Three sets of data are shown on Figure 8.47. The test and control samples taken from the road during August of 1994 have the same average cracking temperature of -20°C . The SA series specimens removed from the pavement in May of 1993, prior to the placement of the wearing course, were tested with the samples removed in August, 1994. The SA series samples storage conditions aged the asphalt due to the easy access of O_2 from both the top and bottom of the samples. This aging lowered the average cracking temperature by about 5°C to -15°C . Little inference was made of the stress at failure due to the magnitude of the variation of the results.

Marshall Properties

Marshall properties were determined on hot samples taken from the trucks at the batch plant and on cores extracted from the large samples taken from the test and control pavement sections.

Table 8.18 contains the Marshall properties obtained from testing the field cores taken from the large slabs removed from the test and control pavement sections. The average values for the stability, flow, unit weight and air voids were 1,644 and 1,863 lb (746 and 845 kg), 0.16 and 0.22 in (0.41 and 0.56 cm), 147.6 and 139.2 lb/ft³ (2.37 and

Table 8.17: Thermal Stress Data

THERMAL STRESS						
SAMPLE	Control			Test		
	Load lbs	Stress psi	Temperature at failure, oC	Load lbs	Stress psi	Temperature at failure, oC
SA ¹ 2				2500.0	393.2	-15.0
SA ¹ 3				2130.0	334.8	-10.0
SA ¹ 5				2348.0	386.9	-18.7
Average				2326.0	371.6	-14.6
Standev				186.0	32.1	4.4
FC 7B	2758.0	434.1	-25.0			
FC 7D	1436.0	227.1	-10.0			
FC 6D	2367.0	373.1	-30.0			
FT 5B				2604.0	412.1	-24.0
FT 5H				2794.0	448.5	-10.0
FT 9D				2400.0	380.1	-23.0
Average	2187.0	344.7	-21.7	2599.3	413.6	-19.0
Standev	679.1	106.3	10.4	197.0	34.2	7.8

Note: ¹ The S series samples were removed from the road in 1993 but not tested until Fall 1994.
The results may have been affected by unrealistic ageing due to laboratory storage conditions

Figure 8.47: Thermal Stress Versus Temperature

Table 8.18: Field Core Marshall Data

Field Core Marshall Properties								
Control					Test			
Sample	Stability	Flow	Unit Weight	Air Voids	Stability	Flow	Unit Weight	Air Voids
	pounds	.01 in	lb/ft ³	%	pounds	.01 in	lb/ft ³	%
FC-2	1,646	13	146.5	6.6				
FC-4	1,749	19	147.1	6.2				
FC-6	1,775	14	147.9	5.7				
FC-7	1,689	14	148.1	5.5				
OC-1	1,359	18	148.3	5.4				
Average	1,644	15.6	147.6	5.9				
Standev	203	3.5	1.3	0.8				
FT-4					1,702	22	140.3	5.7
FT-6					1,940	22	138.7	6.7
FT-7					1,947	22	138.5	6.9
				Average	1,863	22	139.2	6.4
				Standev	173	0	1.3	0.9

Note: ¹ F series were removed from pavement August, 1994

O series were removed May, 1993

C Control

T test

² Assumes maximum theoretical density of control = 2.513 and test = 2.384

2.23 gm/cm³) and 5.9 and 6.4% for the control and test respectively. The stability of the test was stronger than the control due to the increased angularity of the grate ash. The values for the other variables are appropriate for the two mixes.

Batch Plant

Tables 8.19 and 8.20 present the control and test Marshall properties determined from samples fabricated at the batch plant the day the pavement was constructed. The control's average (15 samples) unit weight, air voids, stability and flow were 151.19 lbs/ft³ (2.423 gm/cm³), 3.6%, 2,109 lbs (957 kg) and 0.13 in (0.33 cm) respectively. The test's average unit weight, air voids, stability and flow were 143.21 lbs/ft³ (2.295 gm/cm³), 3.5%, 2,109 lbs (957 kg) and .13 in (0.33 cm) respectively. The averages are equal except for the unit weights and a slight difference in air voids (3.5 vs. 3.6).

These data are different than the data from the field cores. The unit weights are much higher and therefore the air voids are much lower than the field core values. Based on these data it appears the batch plant Marshall compactor produced higher unit weights than obtained in the field.

Pavement Density

The field density presented in Table 5.1 and previously discussed were also lower than the batch plant values. The average control Northbound and Southbound had unit weights of 139.9 and 139.8 lb/ft³ (2.241 and 2.240) and the average test Northbound and Southbound had unit weights of 135.7 and 137.2 lb/ft³ (2.175 and 2.199 gm/cm³).

Figure 8.48 shows the unit weight data plotted as a function of the asphalt content for the Marshall and Gyratory mix designs. The batch plant Marshall data are

Table 8.19: Batch Plant Control Marshall Data

Control Batch Plant Marshall Properties					
Puck	Specific Gravity		Air Voids %	Stability pounds	Flow .01 in
	Bulk	Maximum			
IA	2.412		3.7	2,054	10
IB	2.400		4.2	2,076	14
IC	2.405		4.0	1,708	14
ID	2.408		3.8	1,801	13
IE	2.400		4.2	1,946	13
Average	2.405	2.504	4.0	1,917	12
Stdev	0.005			160	2
Unit Weight	150.1 pcf				
IIA	2.426		3.6	1,946	12.0
IIB	2.422		3.8	1,816	11
IIC	2.442		3.0	2,216	10.0
IID	2.428		3.5	2,830	13.0
IIE	2.438		3.1	1,946	13
Average	2.431	2.517	3.4	2,151	12
Stdev	0.008		0.3	407	1.3
Unit Weight	151.7 pcf				
IIIA	2.443		2.9	2,465	12
IIIB	2.433		3.3	2,327	13
IIIC	2.429		3.5	2,013	15
IIID	2.440		3.1	2,378	14
IIIE	2.417		4.0	2,108	12
Average	2.432	2.517	3.4	2,258	13
Stdev	0.010			190	1
Unit Weight	151.8 pcf				
Total Average	2.422	2.512	3.6	2,109	13
Total Stdev	0.015	0.008	0.4	294	2

Table 8.20: Batch Plant Test Marshall Data

Test Batch Plant Marshall Properties					
Puck	Specific Gravity		Air voids %	Stability Pounds	Flow .01 in
	Bulk	Maximum			
IA	2.316		2.1	2,474	7
IB	2.288		3.3	2,333	18
IC	2.236		1.7	3,081	23
ID	2.320		3.7	2,474	16
IE	2.279			2,432	17
Average	2.306	2.366	2.5	2,559	16
Stdev	0.034		0.9	298	5.8
Unit Weight	143.9 pcf				
IIA	2.284		4.6	2,526	24
IIB	2.270		5.2	2,403	25
IIC	2.306		3.7	2,355	15
IID	2.290		4.4	2,479	17
IIIE	2.225		7.1	1,885	19
Average	2.275	2.395	5.0	2,330	20
Stdev	0.031		1.3	257	4
Unit Weight	142.0 pcf				
IIIA	2.312		3.3	2,562	24
IIIB	2.317		3.1	2,534	19
IIIC	2.332		2.4	2,919	missed
IIID	2.332		2.4	2,249	10
IIIE	2.318		3.0	2,239	27
Average	2.322	2.39	2.8	2,500	20
Stdev	0.009		0.4	279	7
Unit Weight	144.9 pcf				
Total Average	2.295	2.384	3.5	2,109	13
Total Stdev	0.032	0.016	1.4	294	1.5

Figure 8.48: Unit Weight by Gyratory Design (60 R) Versus Asphalt Content

significantly higher than both the Marshall and Gyratory initial (60 rev) and final densities (300 rev). The field core data, plotted at the mix design asphalt contents (5.1 and 7.0%), have values consistent with the Gyratory 300 revolution design curves. This creates a paradox in evaluating the data because in concept Marshall unit weights are always less than those obtained from the Gyratory as shown on Figure 8.48.

The only obvious difference between the plant Marshall testing and the laboratory procedures used to develop the design curves was the asphalt cement and compaction temperatures. Asphalt cements have unique viscosity temperature relationships. The laboratory Marshalls were mixed at a viscosity of 170 ± 20 centistokes and compacted at 280 ± 30 centistokes. The corresponding temperatures to achieve these viscosities with the laboratory AC-20 cement were 295°F (146°C) and 284°F (140°C). The batch plant mixes were mixed immediately upon removal from the trucks, their compaction temperatures were 325°F, 305°F and 305°F (163, 152 and 152°C) for the control and 320°F, 330°F and 325°F (160, 166 and 163°C) for the test. The viscosity temperature relationship is not known for the cement used in the field mixes, but based on typical asphalt cements it is very reasonable to assume the batch plant mixes would be easier to compact at the higher temperatures and therefore would be expected to achieve higher densities.

The hot mix temperature would be expected to have cooled somewhat during the transportation from the batch plant to the Laconia project and it also would have aged due to being held at elevated temperatures. Cooling would result in lower viscosity of the asphalt cement and therefore lower densities than were obtained in the laboratory when the design curves were developed. The laboratory gyratory mixes were aged for 3 hours at 275°F (135°C) before being compacted at 230°F (110°C). Although it is difficult

to make any exact reference to what would have occurred if a gyratory procedure was performed immediately at these high batch temperatures, it does however seem reasonable that the densities would also have increased.

Comparing the initial and final gyratory density shows an increase in density of about 3 to 4 lb/ft³ (0.048 to 0.064 gm/cm³) for the control and 2 to 7 lb/ft³ (0.032 to 0.112 gm/cm³) for the test. The difference between the two mixes at any point in their compaction is about 7 to 10 lb/ft³ (0.112 to 0.160 gm/cm³). These differences are also noted on the relative positions (see Figure 8.48) of the batch and field densities which again suggest the batch Marshalls are unrealistically high relative to what was placed in the field.

Similar results were found when the air voids were analyzed, as would be expected, due to the direct relationship between the two properties. Figure 8.49 shows the initial air void Gyratory mix design data for 60 revolutions. The field core data fall within the 5% to 8% range expected upon completion of paving. The batch Marshall results do not satisfy the lower limit of 5% air voids.

Figure 8.50 combines the design data of Marshall and Gyratory with the air void results. The batch Marshall results appear to have achieved their final compaction (reduced air voids) whereas the field cores show they have not achieved final compaction which would be expected because traffic has not been sufficient to cause compaction. Again the results of the batch Marshall appear to be inconsistent with the field data.

Table 8.20 presents the batch plant extraction data obtained from the same samples which were used to fabricate the pucks for the batch Marshall properties. The average of the 3 samples (I, II and III) indicated the asphalt cement contents for the control and test binder mixes were 5.1% and 7.06% respectively. These samples were not corrected for loss of fines on the filter paper. If the ash filter correction had been

Figure 8.49. Initial Air Voids by Gyratory Design (60 R) Versus Asphalt Content

Figure 8.50: Final Air Voids by Marshall and Gyratory Design (300R) Versus Asphalt Content

performed the asphalt contents of the test would have decreased to approximately the design asphalt contents of 7.0 %. NHDOT requires the variation of asphalt content by the extraction test to be within $\pm 0.4\%$. Both the test and control were well within the NHDOT specifications.

Sieve analyses were run on the extracted samples to determine if the gradations were within the limits set by the specifications. The last two columns of Table 8.21 show the grain size distribution of the six extractions. The control was out of specification on test I and the test was out of specification on each of the three tests. Figure 8.51 shows the grain size distribution for the control mixes. The aggregate was blended in the job mix formula to achieve a mid point target gradation. This was achieved only on the sieves below the #10. In general the gradation tended to be on the coarse side on the #10 sieve and above. Overall the gradation control was adequate except for mix I.

An identical curve for the test extractions is shown on Figure 8.52. The three gradations fit the mid target very well on the top side of the curve where the control did not. The gradation starts to go out of specification on the #20 sieve. The shift in the gradation below the #10 is the result of the grate ash particles breaking during the drying, resieving and mixing of the hot aggregate with the asphalt cement.

The average of the three control and test extracted samples are presented on Figure 8.53. These data also show the test to be slightly too fine (+ 1.1 % on the # 200 sieve) on the lower end of the gradation curve and the control to be too coarse (- 3.8 % on the # 4 sieve) on the mid to upper end. This is not a major problem because under full production of using grate ash as an aggregate substitute, gradation would be controlled by reducing the conventional aggregate fines added to the mix. Although not evaluated by this research it is also anticipated that the use of a drum mixer would

Table 8.21: Batch Plant Extraction Data

Sieve Size	Batch Plant Extraction Data											
	Type B Binder Specifications				Control Sample			Test Sample			Average	
	NH minimum	NH Target	NH maximum	Grate Ash	I	II	III	I	II	III	Control	Test
1"					100	100	100	100	100	100	100.0	100.0
3/4	95	97.5	100	99.7	93.8	99.4	97.1	98.1	97.3	100	96.8	98.5
1/2	70	81	92	88.0	75.3	76.7	77.4	81.5	80.8	85.2	76.5	82.5
3/8	60	71	80	75.4	65.8	67.1	67.3	71.9	71.6	75.7	66.7	73.1
#4	42	50	57	48.9	39.2	45.4	44.8	50.1	48.6	51.3	43.1	50.0
#10	28	32	38	30.6	27.7	29	29.6	31.8	32.7	34.9	28.8	33.1
#20	16	20	24	18.4	19.4	20.1	20.9	22.3	23.5	25.7	20.1	23.8
#40	9	13	17	12.2	12.6	13.7	14.3	16.3	17	19	13.5	17.4
#80	3	7	11	7.2	6.3	7.8	8	10.1	8.6	9.6	7.4	9.4
#200	0	3	4	4.1	2.3	3.6	3.8	5.6	4.5	5.3	3.2	5.1
pan	0	0	0	0	0	.0	0	0	0	0	0	0
Percent Asphalt Content					4.8	5.4	5.1	6.7	6.9	7.6	5.1	7.1

Note: ¹Outside the NHDOT Binder Specifications

² Target asphalt contents were Control 5.1% and Test 7.0%

Figure 8.51: Grain Size Distribution of Extracted Control Mixes

Figure 8.52: Grain Size Distribution of Extracted Test Mixes

Figure 8.53: Average Grain Size Distribution of Extracted Mixes

minimize or even eliminate degradation of the friable particles. Therefore the increased fines is not problematic relative to the future use of MSW grate ash as an aggregate substitute in asphaltic concrete.

CHAPTER 9. CONCLUSIONS

The following conclusions are based on the research conducted on the Laconia bottom ash project and are based on the data found herewithin.

The results of this project show the test and control sections to be performing very well after being in service for almost 2 years and withstanding the effects of over 400,000 trucks as well as over 5,000,000 vehicles. The test is performing as well as the control in all aspects. The estimated layer coefficients for the test and control binders were found to be approximately equal and range from an average of 0.41 to 0.38. The recommended design layer coefficient for the test and control is 0.38, the value presently being used by NHDOT for all asphalt mixes.

The grate ash mix was placed and compacted with conventional asphalt paving equipment. The grate ash took several more roller passes than the control to achieve initial compaction due to its increased angularity and higher stability.

The grate ash was found to undergo degradation during the manufacture of the asphaltic mix. This most likely occurred during the drying process when the aggregates tumble their way through the dryer. The anticipated grate ash degradation can be negated by using less washed and dry manufactured fines for meeting the gradation requirements of the specifications and the job mix formulation.

Based upon the physical properties and testing of the field pavements the use of grate ash for an aggregate substitute up to 50% by weight is a viable form of utilization of a waste product and is recommend for use.

A detailed list of the research project conclusions are as follows:

Mix Design

1. The Gyratory Testing Machine was found to be a viable and appropriate method of mix design for grate ash mixtures.
2. The Marshall mix design method was found to over-estimate the predicted asphalt content for grate ash mixtures (7.0% versus > 8.0%) and is not recommended for use.
3. Predicting the asphalt content of the control was slightly reduced by the GTM method (5.1% versus 5.2%). Either mix design procedure is appropriate for the control.
4. The test mix meeting the appropriate GTM mix design criteria was found to require 1.9 percent additional asphalt cement than the control due to increased absorption.
5. The recommended asphalt contents were 5.1 and 7.0 percent by weight of total mix for the control and test mix.
6. Both the test and control mixes were shown to have low water absorptions.
7. Degradation occurred in both mixes during compaction with the GTM. The test mix degraded more than the control mix for particles in the 3/8 inches (0.95 cm) to No 20 sieve size (0.0331 inches)(0.850 mm) range. The degradation occurs prior to 60 revolutions of the GTM and since 60 revolutions represents the energy applied to HMA by field compaction equipment, future degradation under traffic is not problematic.

Field Placement of Binder Mixes

Batch Plant

1. Extraction testing of the two binder mixes showed the two mixes to be well within the NHDOT acceptable limits of ± 0.4 percent. The average uncorrected asphalt contents were 5.1 and 7.06 percent for the control and test mix respectively. If an ash filter correction (loss on ignition) had been done the grate asphalt would have most likely reduced to the design content of 7.0 percent.
2. The test mix was out of specification on all tests due to the increase of fines caused by degradation during the drying, sieving, and mixing process. The particles passing the No 20 sieve (0.850 mm) increased with an average increase of 1.1 % passing the # 200 sieve (74 μ m).
3. Gradation of the control was out of specification on one of three tests. The amount of material passing the # 4 sieve (1.16 mm) was too small by 3.8 %.
4. Gradation control on future projects is not problematic. One solution is to reduce the quantity of manufactured conventional fines added to the mix such that the difference is made up by the increase in the grate ash fines caused by degradation.

Paving

1. Compaction of the test and control mixes were easily accomplished by means of conventional rollers. Vibratory compaction was not used. The

test required more passes of the compaction equipment due to the increased angularity of the aggregate.

2. Control of compaction was accomplished with a standard nuclear density testing device. The presence of ferrous and nonferrous metals was not problematic in determining and controlling the compaction of the test binder with the nuclear density device.

Pavement Evaluation

Moisture

1. The TDR moisture contents of the pavement granular materials were found to increase with depth. Moisture migration from the lower sections of the granular materials may have contributed to build up of ice in the upper granular layers.
2. Depth of frost penetration was in excess of 52 inches (132.1 cm) during the winter of 1994.

Traffic

1. Over 5.5 million vehicles have traveled on the pavement during the testing period of the project. Approximately 400,000 trucks have used the pavement since it was completed.

ARAN

1. The PSI and IRI for years 1993 and 1994 were not significantly different at the 95 percent level of confidence for the Northbound and Southbound lanes of the control and the Northbound lane of the test. IRI values were high and PSI values were low for a new pavement. The profile of the

pavement was affected by the short based paver used to place the HMA and on the test Northbound lane the placement of a tender wearing course mix. The waviness of the pavement profiles may have rendered the IRI and PSI data suspect. Visually both the test and control pavements are holding up very well and there appears to be no difference between them.

FWD

Deflections

1. The FWD was shown to be an effective non-destructive testing device to evaluate the elastic properties of the pavements and subsurface materials.
2. Weak and strong areas in the pavement's asphalt stabilized base, subbase, and subgrade materials were identified by the FWD testing.
3. The pavement deflections decreased significantly with decreased temperature, increased thickness of base materials and the addition of the binder courses. Deflections increased significantly during the spring thaw.
4. The test and control pavements responded similarly in terms of deflections caused by the FWD loadings throughout the testing period.

Binder Elastic Modulus

1. The binder elastic moduli for the test and control were found to be statistically equal at the 95 percent confidence level for:
 - a. test Southbound and Northbound lanes for the 1993 (1090 ksi, 7.5 GPa) and 1994 (822 ksi, 5.7 GPa) November test dates

- b. control Southbound and Northbound lanes for the 1993 (1100 ksi, 7.6 GPa) and 1994 (555 ksi, 3.8 GPa) November test dates
 - c. test and control for 1993 (1095 ksi, 7.6 GPa).
- 2. The binder moduli decreased after one year of traffic due to the increase in stiffness of the stabilized base.

Layer Coefficients

- 1. The binder layer coefficients were found to be statistically equal at the 95 percent confidence level for:
 - a. test and control Northbound lane for 1993 (0.41) and 1994 (0.38)
 - b. test Southbound and Northbound lanes for 1994 (0.38)
 - c. test and control for 1993 (.41) and 1994 (.40).
- 2. The average layer coefficients of the grate and control were found to decrease from 0.41 to 0.40 between 1993 and 1994, suggesting increased stiffness of the stabilized base.
- 3. The recommended design layer coefficient for the test and control mix is 0.38, the present maximum allowed by the NHDOT.

Strain Gauges

- 1. The binder course strains under constant load were found to increase with increased temperature and spring thaw loss of subgrade support.
- 2. Both binders responded similarly to load and temperature showing the elastic properties of the materials in the test and control to be approximately equal.

Laboratory Testing of Pavement Samples

Resilient Modulus and Indirect Tension

1. The resilient modulus and indirect tension of the test and control were approximately equal and both decreased with increased temperature.

Thermal Stress

1. Thermal stress was approximately equal for both the test and control and for equal aging had cracking temperatures of approximately - 20°C.

Marshall Properties

Batch Plant

1. The test and control Marshall properties determined at the plant had higher densities and lower air voids than were achieved in the field and laboratory.

Field Cores

1. The Marshall properties of the field cores were consistent with the laboratory GTM and Marshall designs.
2. The binders have not achieved the GTM final air void and density design properties after two years of traffic.

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APPENDIX I

Normalized FWD Deflections

Northbound - November 5, 1993

Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
Station	Deflections, 0.001 inches						
Test							
159+12	8.35	7.04	6.13	5.07	4.14	2.83	1.37
159+22	8.52	6.81	5.81	4.68	3.74	2.55	1.44
159+32	7.77	6.32	5.39	4.32	3.48	2.43	1.47
159+42	8.09	6.61	5.66	4.52	3.64	2.51	1.42
159+62	8.84	7.42	6.43	5.17	4.17	2.82	1.52

Control

160+12	7.14	5.83	4.99	4.00	3.27	2.35	1.46
160+22	7.45	5.86	4.88	3.82	3.04	2.13	1.30
160+52	8.25	6.34	5.21	4.01	3.16	2.17	1.29
160+62	8.67	6.52	5.26	4.00	3.15	2.17	1.29

Northbound - October 13, 1994

Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
Station	Deflections, 0.001 inches						
Test							
159+12	8.05	6.07	5.28	4.15	3.32	2.14	1.03
159+22	8.54	6.32	5.34	4.20	3.34	2.25	1.33
159+32	7.48	5.37	4.48	3.44	2.71	1.79	1.16
159+42	5.58	6.46	5.44	4.23	3.36	2.26	1.31
159+62	9.14	6.74	5.74	4.49	3.54	2.34	1.29

Control

160+12	7.44	5.44	4.65	3.69	3.00	2.20	1.42
160+22	7.13	5.14	4.26	3.28	2.59	1.83	1.17
160+52	7.48	5.10	4.18	3.18	2.54	1.72	1.07
160+62	7.57	5.18	4.19	3.14	2.52	1.71	1.07

Normalized Deflections - Northbound

Normalized FWD Deflections

Southbound - November 5, 1993

Offset from Center of Load, inches

D ₀	D ₁	D ₂	D ₃			
0	8	12	18	24	36	60

Station Deflections, 0.001 inches

Test

159+22	10.06	7.94	6.58	5.12	4.05	2.73	1.55
159+32	10.02	7.83	6.45	4.99	3.93	2.67	1.53
159+42	9.87	7.76	6.42	4.94	3.86	2.59	1.49
159+62	9.07	7.26	6.01	4.61	5.59	2.42	1.44
159+72	9.67	7.74	6.34	4.81	5.73	2.51	1.50

Control

160+22	7.98	6.57	5.64	4.54	3.67	2.55	1.38
160+42	6.30	5.22	4.52	3.65	3.02	2.14	1.22
160+52	6.19	5.00	4.30	3.48	2.83	2.00	1.17

Southbound - October 13, 1994

Offset from Center of Load, inches

D ₀	D ₁	D ₂	D ₃			
0	8	12	18	24	36	60

Station Deflections, 0.001 inches

Test

159+22	8.20	6.29	5.35	4.20	3.37	2.22	1.32
159+32	8.46	6.29	5.34	4.17	3.31	2.20	1.28
159+42	8.50	5.99	5.03	3.90	3.12	2.09	1.22
159+62	8.11	6.55	5.59	4.33	3.36	2.09	1.21
159+72	7.71	5.68	4.77	3.67	2.87	1.88	1.12

Control

160+22	7.23	5.73	5.07	4.24	3.55	2.50	1.36
160+42	6.64	4.94	4.24	3.43	2.80	1.94	1.10
160+52	5.47	4.13	3.59	2.95	2.45	1.75	1.05

APPENDIX II

Normalized Deflections - Northbound, Centerline, Base - 05/08/93

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	29.01	16.61	10.00	6.12	4.04	2.45	1.15
153.00	22.16	12.85	7.80	4.90	3.61	2.40	1.27
154.00	24.42	15.62	9.51	6.24	4.46	2.59	0.93
155.00	26.77	13.99	8.47	5.30	3.58	1.86	0.55
156.00	27.15	13.55	8.73	6.13	4.51	3.05	1.12
157.00	22.81	11.64	6.99	4.50	3.07	2.00	0.84
158.00	26.99	15.38	9.78	6.89	5.17	3.71	1.62
158.92	24.34	17.24	12.34	8.62	8.04	4.36	2.17
158.97	27.91	17.88	12.25	8.72	6.41	4.29	2.30
159.00	24.39	16.78	11.45	7.99	5.99	4.15	2.01
159.02	27.34	17.44	11.61	8.34	5.91	3.97	2.12
159.07	29.23	17.68	12.50	8.72	6.36	4.02	2.07
159.12	30.77	18.01	12.22	8.24	5.88	3.60	1.58
159.17	30.44	18.27	12.22	8.32	6.02	3.78	1.86
159.22	30.01	17.70	11.43	7.78	5.32	3.32	1.80
159.27	25.94	17.02	11.61	7.97	5.69	3.61	1.91
159.32	26.81	16.01	10.00	6.93	4.87	3.30	2.04
159.37	25.73	17.74	11.47	7.74	5.44	3.31	1.82
159.42	28.25	15.73	10.71	7.49	5.67	3.58	1.94
159.47	26.79	17.44	11.81	8.33	6.15	3.84	2.17
159.52	32.39	16.59	11.50	8.09	6.19	4.14	2.21
159.57	32.02	19.55	13.11	8.86	6.47	4.17	2.64
159.62	31.53	20.70	14.42	10.14	7.35	4.70	2.57
159.67	33.59	22.73	15.77	11.35	8.18	5.03	2.60
159.72	36.38	21.62	14.91	10.41	7.41	4.74	2.36
159.77	35.48	20.58	14.36	9.99	7.61	4.48	1.82
159.82	26.46	16.41	10.98	7.73	5.86	4.33	2.46
159.87	28.63	15.23	9.82	7.01	4.95	3.66	2.09
159.92	29.99	16.43	10.07	6.83	4.98	3.29	1.84
159.97	23.46	14.66	9.02	6.17	4.71	3.41	1.98
160.00	27.53	14.50	9.73	6.58	5.04	3.44	2.11
160.02	27.13	14.26	9.36	6.58	5.03	3.81	2.30

Normalized Deflections - Northbound, Centerline, Base - 05/08/93 Contd.

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
160.07	27.66	17.05	10.16	7.15	5.43	3.83	2.19
160.12	29.66	16.40	10.13	6.69	5.16	3.55	2.37
160.17	26.98	13.33	9.03	6.07	4.47	3.19	2.00
160.22	26.27	14.74	9.35	6.08	4.20	2.93	1.59
160.27	26.51	14.04	9.06	6.04	4.21	2.78	1.56
160.32	24.74	14.64	9.09	6.12	4.44	2.84	1.69
160.37	29.01	13.81	9.02	6.13	4.35	3.00	1.63
160.42	30.19	13.03	8.27	5.77	4.32	2.91	1.53
160.47	26.78	14.80	9.42	6.20	4.30	3.10	1.72
160.52	29.13	14.87	8.96	6.37	4.68	3.01	1.80
160.57	29.64	14.60	8.55	5.91	4.40	3.09	1.84
160.62	27.39	15.36	8.81	6.16	4.59	2.53	1.66
160.67	28.61	13.76	9.42	5.20	3.95	2.98	1.47
160.72	30.19	14.74	8.84	5.62	4.09	2.94	1.69
160.77	26.42	14.66	8.10	5.31	3.91	2.58	1.55
160.82	26.29	13.66	8.21	5.28	3.69	2.69	1.77
161.00	24.21	13.79	8.10	5.18	3.79	2.90	1.66
162.00	29.09	13.69	7.81	5.23	3.86	2.66	1.45
163.00	24.62	13.44	7.44	4.64	3.55	2.56	1.49
164.00	23.83	16.72	10.72	7.37	5.39	3.24	1.69
165.00	28.14	16.05	9.69	6.47	4.56	2.92	1.43
166.00	25.92	13.08	8.00	6.22	4.65	3.47	1.98
167.00	30.68	16.43	10.39	7.51	5.90	4.19	2.63
168.00	29.10	15.00	11.01	7.90	6.32	5.68	2.75
169.00	30.25	16.62	11.15	7.82	6.36	4.63	2.66
170.00	33.60	18.92	11.77	8.30	6.40	4.44	2.67
171.00	27.50	15.29	9.75	6.51	4.99	3.55	1.98

Normalized Deflections - Southbound, Centerline, Base - 05/08/93

Station Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	27.72	13.94	9.39	5.93	4.14	2.61	1.5
153.00	23.23	11.79	7.5	5.07	3.5	2.2	1.31
154.00	22.78	11.93	7.83	4.86	3.49	2.25	1.07
155.00	25.52	13.57	8.12	5.8	3.36	1.65	0.52
156.00	27.47	14.83	9.44	6.43	3.66	2.64	1.01
157.00	29.69	16.69	9.56	6.62	4.73	2.72	1.16
158.00	25.68	15.74	10.14	7.01	5.33	3.58	2.04
158.82	27.12	16.26	10.27	6.92	5.11	3.03	1.74
158.87	27.19	14.24	8.77	6.13	4.5	3.03	1.84
158.92	26.33	13.88	8.84	6.06	4.72	2.8	1.95
158.97	26.31	13.82	9.42	6.36	4.48	2.96	1.77
159.00	27.49	14.42	9.55	6.43	4.69	2.94	1.63
159.02	27.18	14.98	9.35	6.4	4.62	2.85	1.65
159.07	28.82	14.83	9.66	6.58	4.64	3.02	1.81
159.12	31.52	16.22	10.88	7.26	5.18	3.48	1.73
159.17	31.21	19.23	13.2	9.86	7.16	4.57	1.85
159.22	32.9	20.16	14.23	10.29	7.08	4.28	2.13
159.27	26.58	15.46	10.29	7.08	5.06	3.12	1.87
159.32	25.86	14.99	10.12	6.95	4.92	3.26	1.92
159.37	24.28	14.62	9.99	6.82	4.95	3.39	1.97
159.42	23.3	14.9	10.22	7.05	5.06	3.26	2.03
159.47	30.13	16.67	11.12	7.61	5.41	3.43	1.93
159.52	25.54	16.88	11.59	8	5.88	3.71	1.79
159.57	30.89	17.01	11.86	7.94	5.67	3.52	2.17
159.62	25.87	15.36	11.25	7.71	5.35	3.52	2.29
159.67	21.18	14.33	9.63	6.28	4.38	2.96	1.82
159.72	24.76	14.51	9.39	6.19	4.41	2.86	1.85
159.77	20.75	13.36	9	5.84	4.39	2.47	1.96
159.82	26.86	14.24	9.32	6.24	4.65	3.3	2.07
159.87	28.49	16.93	10.27	6.44	4.51	3.18	1.98
159.92	26.88	12.49	8.16	5.68	4.45	3.48	2.27
160.00	22.16	14.11	9.63	6.56	4.75	3.46	2.02
160.07	30.31	15.99	11.64	8.6	6.77	4.82	2.83

Normalized Deflections - Southbound, Centerline, Base - 05/08/93 Contd.

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
160.12	38.57	25.39	19.13	14.04	10.63	7.2	3.11
160.17	34.56	22.97	17.31	13.31	10.13	6.63	3.21
160.22	39.98	26.99	19.92	14.32	10.55	6.4	2.78
160.27	37.68	24.8	18.46	13.25	9.47	5.43	2.46
160.32	35.58	25.48	18.68	13.06	9.26	5.38	2.7
160.37	36.71	23.39	17.72	12.67	9.12	5.4	2.65
160.42	41.79	28.23	19.95	13.95	9.66	5.37	2.83
160.47	36.32	25.53	18.47	13	8.9	5.16	2.67
160.52	38.94	26.15	18.82	12.48	8.5	5.1	2.62
160.57	36.54	21.82	15.19	10	6.72	4.04	2.55
160.62	31.83	19.94	13.09	8.66	6.13	3.86	2.34
160.67	29.68	16.67	11.36	7.86	5.66	3.71	2.06
160.72	29.09	17.18	11.44	7.62	5.61	3.48	2.02
160.82	24.83	15.71	10.2	7.03	5.04	3.4	2.04
161.00	21.92	13.03	8.66	6.02	4.57	3.23	1.85
162.00	27.15	11.62	7.34	5.04	3.87	2.21	1.77
163.00	30.14	16.2	11.25	7.04	4.77	2.79	1.66
164.00	24.92	12.71	8.06	5.37	3.76	2.33	1.32
165.00	25.25	13.91	8.43	5.58	3.91	2.69	1.56
166.00	25.26	13.41	8.64	5.87	4.51	3.57	2.14
167.00	26.73	14.66	10.28	7.35	5.45	3.86	2.21
168.00	27.92	15.92	10.56	7.34	5.83	4.07	2.52
169.00	29.58	16.19	10.99	7.85	5.86	4.2	2.57
170.00	25.2	15.4	10.95	7.7	5.86	3.92	2.32
171.00	27.09	15.52	10.24	7.05	5.24	3.68	2.18

Normalized Deflections - Northbound, Centerline, Binder - 05/08/93

Station Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	22.87	13.64	8.07	5.14	3.79	2.42	1.24
154.00	19.67	11.68	6.97	4.27	3.03	1.99	0.99
156.00	22.95	14.76	9.25	5.79	4.06	2.35	0.91
158.00	23.73	14.94	9.51	6.36	4.86	3.60	2.12
158.92	16.90	13.08	10.20	7.54	5.70	3.74	2.16
159.02	17.75	13.53	10.53	7.66	5.58	3.54	1.99
159.12	17.59	13.35	10.49	7.62	5.56	3.36	1.60
159.22	18.22	13.74	10.36	7.27	5.14	3.08	1.71
159.32	18.02	13.60	10.30	7.28	5.24	3.33	1.97
159.42	17.60	13.39	10.47	7.56	5.49	3.43	1.87
159.52	18.26	14.02	11.02	8.12	5.96	3.79	2.15
159.62	18.40	14.37	11.20	8.24	6.14	3.86	2.29
159.72	18.44	14.48	11.56	8.62	6.41	4.02	2.08
159.92	18.46	14.17	11.09	8.14	5.94	3.65	1.97
160.00	22.08	14.64	9.91	6.48	4.72	3.27	2.00
160.02	17.30	13.34	10.55	7.78	5.81	3.85	2.24
160.12	16.91	12.63	9.78	7.12	5.33	3.56	2.11
160.22	16.73	12.30	9.04	6.31	4.49	2.86	1.72
160.32	15.65	11.65	8.82	6.31	4.55	2.85	1.62
160.42	14.98	11.44	8.78	6.35	4.62	2.92	1.67
160.52	15.84	11.71	8.67	6.06	4.28	2.76	1.64
160.62	17.26	12.14	8.65	5.74	4.04	2.62	1.61
160.72	17.24	11.96	8.58	5.68	3.89	2.50	1.47
162.00	17.41	10.53	6.47	4.38	3.32	2.43	1.53
164.00	19.35	12.18	7.45	4.84	3.43	2.23	1.24
166.00	18.44	11.17	7.06	4.97	3.81	2.84	1.77
168.00	22.77	14.71	9.71	6.81	5.32	3.84	2.33
170.00	22.62	14.22	9.45	6.92	5.24	3.79	2.23

Normalized Deflections - Northbound, Wheelpath, Binder - 05/08/93

Station Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	24.95	15.12	9.65	6.76	4.95	3.23	1.59
154.00	17.40	10.70	6.62	4.26	3.12	2.02	0.97
156.00	21.03	13.09	8.46	5.55	4.10	2.46	0.92
158.00	24.53	15.15	9.46	6.68	5.12	3.66	2.08
158.92	19.06	14.43	11.30	8.36	6.27	4.12	2.40
159.02	19.48	15.10	11.83	8.78	6.45	4.06	2.26
159.12	18.69	14.29	11.17	8.16	6.02	3.88	2.16
159.22	21.32	16.49	12.58	8.97	6.40	3.84	2.18
159.32	20.50	16.14	12.67	9.24	6.69	4.03	2.15
159.42	20.53	15.96	12.51	9.22	6.75	4.19	2.16
159.52	19.85	15.30	12.00	8.72	6.37	4.01	2.29
159.62	18.88	14.32	11.24	8.12	5.86	3.57	1.90
159.72	19.70	15.41	12.24	9.09	6.67	4.11	2.13
159.92	16.75	13.08	10.33	7.48	5.39	3.26	1.82
160.00	20.49	13.98	9.81	6.93	5.17	3.72	2.24
160.02	16.56	12.74	10.05	7.24	5.13	3.02	1.75
160.12	16.16	12.58	9.97	7.26	5.21	3.12	1.75
160.22	15.00	11.58	9.22	6.74	4.93	3.02	1.75
160.32	17.26	13.54	10.78	7.99	5.83	3.54	1.89
160.42	16.10	12.48	9.78	7.07	5.11	3.15	1.85
160.52	19.77	15.20	11.71	8.19	5.66	3.18	1.76
160.62	19.85	15.37	11.90	8.37	5.77	3.37	1.90
160.72	21.86	16.64	12.74	8.82	6.06	3.48	1.86
162.00	19.97	12.42	8.05	5.66	4.18	2.85	1.67
164.00	17.68	10.91	6.89	4.74	3.49	2.44	1.37
166.00	21.22	13.78	9.51	6.86	5.24	3.62	2.10
168.00	21.31	13.85	9.76	7.30	5.72	4.05	2.46
170.00	22.04	14.12	9.96	7.63	5.83	4.06	2.45

Normalized Deflections - Southbound, Centerline, Binder - 05/08/93

Station Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
154.00	23.20	15.30	9.49	6.31	4.50	2.77	1.17
156.00	24.61	15.53	9.98	6.84	5.05	3.31	1.55
158.00	22.10	14.09	9.27	6.89	5.38	3.76	1.89
158.92	19.19	13.25	9.38	6.10	4.15	2.67	1.56
159.02	19.14	13.36	9.61	6.34	4.38	2.73	1.57
159.12	20.77	14.86	10.84	7.36	5.15	3.01	1.47
159.22	21.45	15.77	11.72	8.10	5.75	3.55	1.94
159.32	18.00	12.96	9.62	6.61	4.66	2.99	1.78
159.42	19.20	13.62	9.94	6.78	4.83	3.11	1.88
159.52	19.86	14.28	10.43	7.11	5.05	3.21	1.83
159.62	23.95	17.77	13.16	8.87	5.96	3.33	1.74
159.72	18.23	12.66	9.00	5.93	4.05	2.63	1.61
159.92	17.29	13.13	9.89	6.84	4.87	3.15	1.78
160.00	23.22	16.91	12.31	8.57	6.28	4.09	2.11
160.02	17.49	12.74	9.41	6.57	4.81	3.25	2.03
160.12	20.88	16.41	13.19	10.08	7.81	5.33	2.88
160.22	22.38	17.80	14.08	10.55	7.98	5.17	2.57
160.32	21.00	16.20	12.77	9.41	6.93	4.35	2.21
160.42	22.30	16.98	13.10	9.47	6.78	4.15	2.06
160.52	21.20	16.07	12.33	8.86	6.38	3.88	2.00
160.62	17.16	12.99	10.03	7.27	5.28	3.25	1.79
162.00	22.75	13.90	7.98	5.07	3.73	2.59	1.59
164.00	23.59	15.32	10.07	6.73	4.94	3.20	1.63
166.00	21.72	13.41	8.38	5.61	4.39	3.15	1.90
168.00	24.88	16.07	10.32	7.38	5.81	4.26	2.49
170.00	26.92	17.20	11.21	7.77	5.79	4.06	2.36

Normalized Deflections - Southbound, Wheelpath, Binder - 05/08/93

Station Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152	26.98	17.14	10.28	6.84	5.05	3.29	1.57
154	24.72	15.69	10.56	7.4	5.72	3.79	2.01
156	26.67	17.44	10.99	7.53	5.55	3.6	1.61
158	24.85	14.21	8.77	6.36	4.91	3.53	1.8
158.92	20.27	14.3	10.01	6.54	4.6	2.94	1.76
159.02	21.08	14.45	10	6.66	4.68	2.95	1.7
159.12	21.89	15.65	11.35	7.83	5.74	3.58	1.57
159.22	22.29	15.99	11.59	8.04	5.83	3.76	2.05
159.32	20.51	14.65	10.56	7.18	5.14	3.37	1.94
159.42	19.72	14.04	10.07	6.78	4.85	3.29	1.99
159.52	19.58	13.95	9.94	6.79	4.89	3.27	1.9
159.62	20.6	15.05	10.65	6.88	4.63	2.94	1.74
159.72	18.71	12.86	9.01	5.97	4.23	2.85	1.67
159.92	20.14	14.41	10.33	6.87	4.86	3.28	1.96
160	25.96	18.34	13.14	8.86	6.1	3.88	2.35
160.02	16.45	12.09	9.13	6.61	5.2	3.83	2.4
160.12	22.95	18.22	14.61	11.21	8.74	5.85	2.98
160.22	19.55	14.6	11.33	8.55	6.67	4.62	2.47
160.32	19.98	15	11.4	8.21	6.06	4.04	2.17
160.42	22.04	16.83	12.77	9.12	6.68	4.29	2.14
160.52	21.13	16.26	12.65	9.27	6.82	4.26	2.14
160.62	19.35	14.56	11.12	8.03	5.85	3.68	2
160.72	18.91	13.82	10.55	7.54	5.49	3.54	1.91
162	25.57	15.89	9.49	6.63	4.95	3.19	1.98
164	22.79	15.39	10.56	7.55	5.64	3.76	1.85
166	25.81	16.88	11.1	7.93	6.06	4.21	2.35
168	23.56	15.86	10.81	7.97	6.16	4.36	2.46
170	25.07	16.4	11	7.77	5.85	4.04	2.44

Normalized Deflections - Northbound, Centerline, Wearing Course - 11/05/93

Station Offset from Center of Load, inches

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	9.16	6.99	5.69	4.25	3.15	1.86	0.84
154.00	12.01	8.94	7.00	5.02	3.68	2.17	0.88
156.00	13.59	10.29	8.24	6.10	4.62	2.83	1.13
158.00	10.05	7.41	6.03	4.71	3.79	2.63	1.39
158.92	8.09	6.81	5.94	4.89	4.06	2.91	1.68
159.02	7.85	6.52	5.67	4.63	3.78	2.63	1.46
159.12	8.35	7.04	6.13	5.07	4.14	2.83	1.37
159.22	8.52	6.81	5.81	4.68	3.74	2.55	1.44
159.32	7.77	6.32	5.39	4.32	3.48	2.43	1.47
159.42	8.09	6.61	5.66	4.52	3.64	2.51	1.42
159.52	8.05	6.43	5.49	4.37	3.52	2.42	1.39
159.62	8.84	7.42	6.43	5.17	4.17	2.82	1.52
159.72	8.86	7.34	6.29	5.09	4.07	2.75	1.51
159.92	7.04	5.76	4.99	4.06	3.33	2.38	1.41
160.00	7.35	6.03	5.21	4.26	3.51	2.54	1.54
160.02	7.42	6.11	5.27	4.28	3.53	2.57	1.58
160.12	7.14	5.83	4.99	4.00	3.27	2.35	1.46
160.22	7.45	5.86	4.88	3.82	3.04	2.13	1.30
160.32	7.68	6.05	5.03	3.91	3.10	2.13	1.27
160.42	7.91	6.17	5.11	4.00	3.15	2.17	1.28
160.52	8.25	6.34	5.21	4.01	3.16	2.17	1.29
160.62	8.67	6.52	5.26	4.00	3.15	2.17	1.29
160.72	8.91	6.66	5.36	4.05	3.16	2.18	1.31
162.00	9.05	6.83	5.52	4.16	3.24	2.20	1.32
164.00	10.34	7.87	6.27	4.64	3.56	2.36	1.35
166.00	13.30	9.92	7.70	5.52	4.11	2.62	1.43
168.00	12.93	10.00	8.12	6.20	4.90	3.40	2.00
170.00	14.64	11.54	9.55	7.49	6.06	4.30	2.48

Normalized Deflections - Northbound, Wheelpath, Wearing Course - 11/05/93

Station	Offset from Center of Load, inches						
	D ₀ 0	8	D ₁ 12	18	D ₂ 24	D ₃ 36	60
152.00	11.38	9.28	7.73	5.87	4.40	2.60	1.14
154.00	13.03	10.14	8.16	6.07	4.56	2.76	1.30
156.00	14.69	11.29	8.90	6.46	4.84	2.95	1.26
158.00	10.01	7.67	6.32	5.00	3.99	2.71	1.41
158.92	8.01	6.83	6.06	5.11	4.23	3.06	1.76
159.02	8.21	6.83	6.01	5.00	4.11	2.88	1.59
159.12	8.88	7.25	6.28	5.11	4.16	2.83	1.43
159.22	8.86	7.29	6.24	4.96	3.99	2.66	1.45
159.32	7.99	6.45	5.59	4.53	3.67	2.52	1.40
159.42	7.74	6.50	5.64	4.67	3.81	2.64	1.40
159.52	7.74	6.46	5.64	4.60	3.71	2.50	1.34
159.62	8.02	6.89	6.11	5.02	4.12	2.81	1.51
159.72	8.15	6.81	5.97	4.87	3.97	2.71	1.50
159.92	7.27	6.21	5.47	4.52	3.71	2.59	1.49
160.00	7.48	6.33	5.57	4.57	3.77	2.68	1.59
160.02	7.52	6.40	5.62	4.64	3.80	2.69	1.61
160.12	7.33	6.30	5.53	4.54	3.71	2.61	1.54
160.22	8.14	6.82	5.87	4.75	3.78	2.52	1.39
160.32	7.74	6.49	5.65	4.61	3.73	2.52	1.38
160.42	8.21	6.80	5.89	4.78	3.83	2.60	1.44
160.52	8.62	6.96	5.91	4.68	3.74	2.52	1.47
160.62	9.21	7.30	6.09	4.72	3.72	2.52	1.47
160.72	9.12	7.21	5.97	4.63	3.61	2.40	1.39
162.00	10.61	8.65	7.30	5.71	4.53	3.01	1.68
164.00	13.54	10.62	8.68	6.48	4.90	3.04	1.52
166.00	14.69	11.55	9.42	7.15	5.54	3.70	2.01
168.00	13.90	11.30	9.47	7.39	5.91	4.03	2.20
170.00	14.34	11.81	9.91	7.65	6.03	4.01	2.33

Normalized Deflections - Southbound, Centerline, Wearing Course - 11/05/93

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	9.71	7.06	5.44	3.79	2.75	1.63	0.72
154.00	10.48	7.96	6.35	4.64	3.45	2.04	0.77
156.00	10.02	7.58	6.11	4.67	3.69	2.55	1.40
158.00	8.97	7.00	5.73	4.36	3.41	2.31	1.32
158.92	8.56	6.81	5.68	4.37	3.43	2.31	1.32
159.02	9.00	7.30	6.18	4.88	3.91	2.64	1.36
159.12	9.66	7.79	6.55	5.14	4.11	2.77	1.44
159.22	9.86	7.85	6.55	5.09	4.03	2.70	1.51
159.32	10.06	7.94	6.58	5.12	4.05	2.73	1.55
159.42	10.02	7.83	6.45	4.99	3.93	2.67	1.53
159.52	9.87	7.76	6.42	4.94	3.86	2.59	1.49
159.62	10.29	8.16	6.72	5.09	3.90	2.55	1.45
159.72	9.07	7.25	6.01	4.61	3.59	2.42	1.44
159.92	9.67	7.74	6.34	4.81	3.73	2.51	1.50
160.00	8.29	6.57	5.53	4.34	3.56	2.60	1.64
160.02	8.42	6.90	5.92	4.81	4.04	3.01	1.84
160.12	8.48	7.17	6.31	5.27	4.42	3.22	1.77
160.22	8.31	6.98	6.08	4.97	4.06	2.80	1.46
160.32	7.98	6.57	5.64	4.54	3.67	2.55	1.38
160.42	7.75	6.18	5.25	4.19	3.39	2.36	1.30
160.52	6.30	5.22	4.52	3.65	3.02	2.14	1.22
160.62	6.19	5.00	4.30	3.48	2.83	2.00	1.17
160.72	7.31	5.75	4.80	3.71	2.93	2.02	1.18
162.00	9.49	7.23	5.76	4.20	3.18	2.07	1.17
164.00	13.17	9.98	8.06	6.22	4.97	3.48	1.98
166.00	12.77	9.99	8.30	6.55	5.32	3.83	2.30
168.00	12.44	9.94	8.36	6.64	5.44	3.90	2.31
170.00	12.81	10.10	8.37	6.64	5.44	3.92	2.36

Normalized Deflections - Southbound, Wheelpath, Wearing Course - 11/05/93

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	9.82	7.85	6.54	5.06	3.90	2.45	1.17
154.00	8.86	6.80	5.47	4.03	3.01	1.79	0.78
156.00	9.81	7.77	6.36	4.81	3.59	2.12	0.77
158.00	9.22	7.27	6.05	4.79	3.87	2.75	1.65
158.92	7.69	6.42	5.53	4.41	3.53	2.39	1.33
159.02	7.50	6.18	5.28	4.28	3.49	2.43	1.42
159.12	8.45	7.12	6.16	5.06	4.13	2.83	1.40
159.22	9.17	7.81	6.81	5.59	4.53	3.06	1.64
159.32	8.73	7.55	6.58	5.37	4.33	2.94	1.61
159.42	8.87	7.43	6.42	5.20	4.19	2.84	1.57
159.52	8.56	7.13	6.14	4.93	3.95	2.65	1.51
159.62	10.46	8.63	7.27	5.66	4.34	2.70	1.46
159.72	8.45	6.97	5.92	4.65	3.67	2.47	1.39
159.92	9.89	8.30	7.04	5.46	4.22	2.77	1.64
160.00	7.49	6.17	5.29	4.30	3.53	2.60	1.74
160.02	7.67	6.46	5.65	4.76	4.05	3.10	1.97
160.12	8.07	6.90	6.12	5.20	4.41	3.27	1.89
160.22	8.48	7.15	6.23	5.23	4.31	3.03	1.59
160.32	8.03	6.69	5.81	4.79	3.94	2.77	1.47
160.42	7.71	6.49	5.67	4.69	3.82	2.65	1.43
160.52	6.54	5.57	4.91	4.10	3.39	2.41	1.36
160.62	6.35	5.28	4.63	3.84	3.17	2.25	1.27
160.72	7.54	6.01	5.07	3.97	3.17	2.19	1.28
162.00	10.40	8.21	6.75	5.17	3.99	2.61	1.41
164.00	10.03	7.77	6.26	4.67	3.57	2.32	1.34
166.00	12.64	10.30	8.64	6.85	5.47	3.72	2.05
168.00	10.74	9.11	7.88	6.43	5.26	3.76	2.21
170.00	11.67	9.57	8.12	6.61	5.48	3.97	2.40

Normalized Deflections - Northbound, Centerline, Wearing Course - 03/06/94

Station Offset from Center of Load, inches

	D ₀ 0	8	D ₁ 12	18	D ₂ 24	D ₃ 36	60
152.00	12.03	9.52	7.81	5.89	4.39	2.52	1.60
153.00	12.45	8.88	6.75	4.64	3.36	1.98	1.40
154.00	12.57	8.48	6.06	3.80	2.50	1.36	1.00
155.00	14.11	9.22	6.52	3.97	2.58	1.29	0.83
156.00	14.11	9.76	7.21	4.80	3.38	1.81	1.16
157.00	11.97	7.84	5.61	3.66	2.52	1.34	0.84
158.00	9.67	6.06	4.30	2.67	1.76	1.11	0.90
158.82	5.00	4.05	3.35	2.75	2.28	1.67	1.34
158.87	4.79	3.66	3.09	2.43	2.03	1.57	1.30
158.92	4.73	3.55	2.95	2.37	2.01	1.54	1.28
158.97	4.61	3.45	2.85	2.36	2.10	1.56	1.23
159.00	4.54	3.53	2.97	2.37	2.07	1.55	1.28
159.02	5.17	3.98	3.41	2.71	2.36	1.68	1.30
159.07	5.12	3.96	3.36	2.63	2.19	1.57	1.25
159.12	5.69	4.28	3.65	2.87	2.38	1.69	1.31
159.17	5.47	4.09	3.63	2.70	2.29	1.62	1.25
159.22	4.79	3.39	2.83	2.25	1.81	1.52	1.21
159.27	4.72	3.37	2.71	2.12	1.77	1.35	1.17
159.32	5.17	3.75	3.11	2.34	1.90	1.41	1.16
159.37	5.11	3.83	3.17	2.49	2.08	1.55	1.21
159.42	5.88	4.43	3.55	2.77	2.24	1.61	1.27
159.47	7.79	6.06	5.01	3.72	2.95	1.88	1.41
159.52	8.07	6.39	5.57	4.20	3.32	2.13	1.47
159.57	7.61	6.19	5.20	4.11	3.20	1.99	1.40
159.62	6.35	5.02	4.21	3.26	2.58	1.71	1.32
159.67	5.76	4.35	3.58	2.79	2.23	1.58	1.23
159.72	7.54	5.76	4.71	3.52	2.87	1.79	1.33
159.77	5.83	4.58	3.75	2.98	2.44	1.77	1.37
159.82	6.55	5.02	4.15	3.15	2.35	1.65	1.27
159.87	5.34	4.02	3.32	2.57	2.06	1.40	1.09
159.92	5.21	3.86	3.39	2.48	2.04	1.38	1.09
159.97	5.01	3.81	3.17	2.49	1.88	1.48	1.19

Normalized Deflections - Northbound, Centerline, Wearing Course - 03/06/94 Contd.

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
160.00	5.30	3.93	3.34	2.53	2.14	1.48	1.22
160.02	5.41	4.03	3.34	2.55	2.09	1.47	1.23
160.07	5.21	3.82	3.22	2.50	2.05	1.54	1.25
160.12	5.32	3.85	3.13	2.43	1.93	1.37	1.16
160.17	5.67	4.09	3.27	2.47	2.00	1.43	1.19
160.22	6.77	4.98	4.00	2.99	2.33	1.53	1.20
160.27	6.66	4.83	3.86	2.83	2.20	1.43	1.12
160.32	5.86	4.15	3.17	2.31	1.77	1.19	1.01
160.37	5.95	4.26	3.30	2.26	1.72	1.17	0.96
160.42	6.55	4.58	3.62	2.57	1.98	1.33	1.06
160.47	7.62	5.60	4.20	2.85	2.08	1.29	1.04
160.52	7.05	4.76	3.40	2.39	1.80	1.30	1.08
160.57	7.24	4.94	3.53	2.34	1.76	1.23	1.05
160.62	7.78	4.82	3.63	2.40	1.75	1.25	1.06
160.67	7.93	5.27	3.91	2.62	1.92	1.33	1.10
160.72	8.83	6.16	4.66	3.13	2.29	1.49	1.24
160.77	9.12	6.05	4.53	3.19	2.37	1.51	1.16
160.82	8.80	5.54	3.95	2.62	1.92	1.38	1.17
161.00	9.04	5.91	4.51	3.13	2.40	1.60	1.31
162.00	11.37	8.02	6.20	4.35	3.25	2.03	1.55
163.00	12.38	8.96	6.86	4.70	3.30	1.70	1.28
164.00	15.14	10.36	7.65	5.14	3.59	1.98	1.39
165.00	12.34	8.56	6.37	4.49	3.30	2.04	1.53
166.00	15.21	11.10	8.79	6.57	5.24	3.49	2.63
167.00	15.79	11.90	9.74	7.60	6.14	4.22	3.19
168.00	17.73	13.34	10.65	8.07	6.55	4.42	3.34
169.00	17.38	12.89	10.34	7.94	6.44	4.53	3.53
170.00	17.87	13.41	10.87	8.24	6.47	4.27	3.27
171.00	15.21	11.29	9.13	6.89	5.45	3.63	2.79

Normalized Deflections - Northbound, Wheelpath, Wearing Course - 03/06/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	23.13	18.48	15.04	10.85	7.71	3.95	2.40
153.00	19.79	14.34	10.68	6.94	4.65	2.47	1.74
154.00	19.45	12.95	9.46	6.23	4.39	2.63	1.87
155.00	15.22	10.32	7.43	5.03	3.65	2.23	1.59
156.00	20.02	13.31	9.46	6.06	4.13	2.25	1.49
157.00	17.15	11.09	7.70	4.69	2.97	1.46	1.02
158.00	9.96	6.59	4.65	2.93	2.01	1.26	1.01
159.00	7.02	5.68	4.75	3.81	3.16	2.21	1.73
160.00	6.67	5.35	4.45	3.41	2.62	1.70	1.31
161.00	11.07	8.10	6.19	4.32	3.12	1.91	1.48
162.00	14.02	10.23	8.07	5.74	4.32	2.79	1.95
163.00	14.96	10.68	7.99	5.38	3.75	2.07	1.48
164.00	16.70	12.09	9.32	6.37	4.54	2.67	1.82
165.00	14.55	10.67	8.28	5.88	4.35	2.61	1.85
166.00	17.18	12.93	10.20	7.70	5.91	3.83	2.82
167.00	16.95	12.81	10.36	7.93	6.27	4.32	3.24
168.00	17.10	12.94	10.53	8.01	6.30	4.28	3.12
169.00	16.87	13.00	10.53	8.14	6.47	4.46	3.32
170.00	18.31	13.93	11.17	8.28	6.47	4.26	3.18
171.00	15.73	12.13	9.63	7.25	5.64	3.68	2.76

Normalized Deflections - Southbound, Centerline, Wearing Course - 03/06/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	10.75	7.85	6.20	4.46	3.39	2.04	1.43
153.00	11.49	7.63	5.65	3.77	2.66	1.50	1.03
154.00	9.86	5.98	4.16	2.42	1.55	0.76	0.53
155.00	9.81	6.16	4.24	2.44	1.51	0.67	0.43
156.00	9.60	6.18	4.35	2.67	1.76	0.96	0.65
157.00	13.91	8.91	6.39	4.10	2.77	1.48	0.93
158.00	9.43	6.28	4.71	3.40	2.64	1.81	1.41
158.82	11.21	7.29	5.58	3.99	2.97	1.88	1.42
158.87	6.68	4.99	4.03	3.00	2.35	1.59	1.31
158.92	6.66	4.78	3.66	2.56	1.89	1.29	1.09
158.97	6.61	4.74	3.67	2.54	1.90	1.26	1.05
159.00	6.68	4.87	3.80	2.74	2.05	1.34	1.05
159.02	7.05	5.05	4.02	2.91	2.26	1.51	1.17
159.07	7.06	5.27	4.20	3.20	2.52	1.73	1.31
159.12	8.85	6.88	5.58	4.26	3.31	2.12	1.50
159.17	7.97	6.19	5.02	3.88	3.15	2.13	1.57
159.22	8.77	6.50	5.04	3.60	2.66	1.62	1.22
159.27	5.82	3.98	2.99	2.26	1.82	1.39	1.19
159.32	5.50	3.87	3.01	2.22	1.77	1.34	1.12
159.37	5.82	4.11	3.17	2.53	1.90	1.36	1.13
159.42	7.15	5.13	3.89	2.83	2.17	1.52	1.24
159.47	7.63	5.53	4.23	3.04	2.31	1.57	1.27
159.52	7.68	5.63	4.45	3.33	2.60	1.77	1.39
159.57	8.41	6.41	5.21	3.94	3.07	2.02	1.53
159.62	9.68	7.58	6.05	4.43	3.31	2.11	1.63
159.67	8.18	6.31	5.17	3.93	3.12	2.03	1.49
159.72	9.30	7.15	5.71	4.35	3.31	2.11	1.58
159.77	8.70	6.67	5.37	4.13	3.19	2.08	1.57
159.82	8.15	6.16	4.98	3.84	2.97	1.96	1.49
159.87	7.45	5.61	4.47	3.38	2.66	1.80	1.40
159.92	8.35	6.36	5.05	3.64	2.68	1.71	1.35
159.97	8.52	6.30	4.85	3.45	2.56	1.62	1.30

Normalized Deflections - Southbound, Centerline, Wearing Course - 03/06/94 Contd.

	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
160.00	7.67	5.70	4.42	3.35	2.52	1.72	1.37
160.02	7.48	5.64	4.39	3.29	2.54	1.74	1.40
160.07	6.50	4.98	4.03	3.15	2.57	1.83	1.47
160.12	6.11	4.66	3.80	3.06	2.53	1.84	1.47
160.17	5.89	4.61	3.88	3.16	2.63	1.98	1.58
160.22	7.58	6.26	5.34	4.30	3.44	2.32	1.65
160.27	7.26	5.77	4.75	3.75	3.00	2.01	1.50
160.32	5.60	4.23	3.46	2.78	2.23	1.57	1.28
160.37	5.87	4.46	3.61	2.85	2.26	1.64	1.32
160.42	6.76	5.08	4.12	3.28	2.58	1.78	1.37
160.47	7.75	6.28	5.31	4.25	3.49	2.33	1.75
160.52	7.02	5.77	4.99	4.05	3.35	2.26	1.68
160.57	6.38	5.33	4.63	3.79	3.13	2.14	1.59
160.62	6.49	5.38	4.61	3.77	3.14	2.16	1.62
160.67	6.83	5.71	4.87	3.93	3.23	2.18	1.64
160.72	7.46	6.08	5.17	4.01	3.27	2.17	1.63
160.77	7.93	6.31	5.26	4.07	3.21	2.07	1.57
160.82	9.58	7.04	5.42	4.00	3.12	2.13	1.64
161.00	10.57	7.88	6.20	4.56	3.51	2.24	1.64
162.00	16.27	11.86	8.89	6.05	4.34	2.45	1.70
163.00	14.89	10.13	7.33	4.77	3.28	1.83	1.29
164.00	11.97	8.25	6.16	4.07	2.86	1.57	1.08
165.00	9.56	6.43	4.74	3.11	2.24	1.41	1.13
166.00	12.25	9.31	7.59	5.83	4.69	3.22	2.42
167.00	12.56	9.66	7.99	6.28	5.15	3.47	2.58
168.00	13.90	10.81	8.92	6.93	5.65	3.97	2.97
169.00	14.44	10.98	8.99	7.07	5.74	4.14	3.21
170.00	13.84	10.38	8.46	6.68	5.63	3.96	3.07
171.00	14.40	10.90	8.82	6.79	5.46	3.81	2.88

Normalized Deflections - Southbound, Wheelpath, Wearing Course - 03/06/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	19.80	14.91	11.67	8.06	5.87	3.19	2.05
153.00	16.54	11.60	8.52	5.40	3.45	1.61	1.04
154.00	10.32	7.04	5.25	3.27	2.13	1.03	0.68
155.00	9.43	6.03	4.20	2.48	1.46	0.65	0.46
156.00	10.56	7.14	5.15	3.30	2.16	1.11	0.78
157.00	15.49	10.18	7.29	4.59	3.17	1.70	1.00
158.00	10.51	7.49	5.94	4.28	3.17	2.13	1.64
159.00	7.61	5.59	4.42	3.18	2.32	1.50	1.12
160.00	7.78	5.88	4.69	3.54	2.75	2.02	1.57
161.00	11.62	9.00	7.24	5.34	4.05	2.53	1.84
162.00	14.60	10.25	7.67	5.22	3.84	2.48	1.89
163.00	12.76	8.48	6.08	4.00	2.93	1.87	1.40
164.00	10.02	6.83	5.01	3.45	2.56	1.64	1.27
165.00	9.83	6.94	5.19	3.50	2.46	1.50	1.17
166.00	12.74	9.70	7.80	5.99	4.71	3.22	2.39
167.00	12.38	9.55	7.87	6.16	4.99	3.40	2.42
168.00	13.73	10.78	8.88	7.04	5.71	3.92	2.95
169.00	13.59	10.57	8.86	7.10	5.86	4.18	3.21
170.00	14.29	10.78	8.93	7.18	5.89	4.16	3.15
171.00	16.21	11.95	9.71	7.38	5.88	4.00	2.96

Normalized Deflections - Northbound, Centerline, Wearing Course - 04/05/94

Station	Offset from Center of Load, inches						
	D ₀ 0	8	D ₁ 12	18	D ₂ 24	D ₃ 36	60
152.00	11.96	10.30	8.00	6.15	4.72	2.60	1.39
154.00	21.45	16.85	12.73	9.32	6.75	3.81	1.67
156.00	15.95	12.40	9.29	6.67	4.81	2.82	1.23
158.00	13.80	10.72	8.17	6.35	5.03	3.16	1.76
158.82	8.80	7.79	6.52	5.37	4.43	3.08	2.03
158.92	9.02	7.85	6.68	5.57	4.69	3.10	2.04
159.02	8.40	7.43	6.33	5.29	4.44	3.04	1.87
159.12	10.26	9.11	7.79	6.56	5.42	3.46	1.86
159.22	9.16	7.95	6.61	5.49	4.48	2.98	1.86
159.32	8.27	7.47	6.03	4.93	3.98	2.74	1.82
159.42	9.29	8.00	6.52	5.35	4.30	2.90	1.84
159.52	8.77	7.97	6.56	5.38	4.39	2.94	1.80
159.62	9.45	8.43	7.08	5.86	4.78	3.17	1.98
159.72	9.42	8.71	7.10	5.91	4.68	3.10	1.92
159.82	8.55	7.44	6.24	5.22	4.33	3.00	1.61
159.92	7.81	6.90	5.75	4.83	3.93	2.76	1.85
160.00	8.13	7.07	5.99	4.94	4.15	2.87	1.80
160.02	8.02	7.14	5.89	4.93	4.08	2.92	2.10
160.12	7.75	6.95	5.58	4.64	3.80	2.68	1.87
160.22	8.31	7.11	5.60	4.50	3.57	2.43	1.68
160.32	8.53	7.33	5.86	4.71	3.71	2.48	1.66
160.42	9.44	7.91	6.41	5.14	4.11	2.66	1.72
160.52	10.78	8.58	6.65	5.21	4.08	2.67	1.74
160.62	11.47	9.00	6.77	5.20	4.01	2.64	1.33
160.72	10.78	8.61	6.80	5.28	4.05	2.52	1.51
160.82	12.83	8.85	7.13	5.23	3.96	2.59	1.74
162.00	11.26	8.60	6.60	5.05	3.90	2.60	1.69
164.00	15.02	11.01	8.17	6.03	4.56	2.47	0.75
166.00	13.70	10.50	8.15	6.21	5.08	3.31	1.67
168.00	14.98	11.44	9.28	7.41	6.19	4.29	2.22
170.00	15.85	12.48	9.58	7.67	6.30	4.27	2.21

Normalized Deflections - Northbound, Wheelpath, Wearing Course - 04/05/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	12.38	10.26	8.62	6.46	5.18	3.07	1.63
154.00	18.85	14.77	11.67	8.51	6.49	3.70	2.05
156.00	15.92	12.61	9.70	6.81	5.20	2.96	1.56
158.00	13.70	10.48	8.37	6.25	5.07	3.15	1.99
158.82	9.61	8.67	7.25	5.75	4.91	3.18	2.25
158.92	9.15	8.07	6.85	5.74	5.09	3.43	2.37
159.02	9.48	8.24	7.01	5.79	5.10	3.38	2.33
159.12	10.94	9.75	8.36	6.85	5.92	3.76	2.33
159.22	10.06	8.77	7.38	5.92	5.09	3.17	2.15
159.32	9.13	7.96	6.68	5.41	4.70	2.98	2.03
159.42	9.25	8.21	6.96	5.75	4.98	3.19	2.07
159.52	8.83	8.06	6.82	5.50	4.77	2.96	2.02
159.62	8.86	7.99	6.89	5.68	4.77	3.13	2.07
159.72	9.41	9.03	7.44	6.03	5.14	3.16	2.06
159.82	9.19	8.47	7.21	5.86	5.07	3.26	2.04
159.92	8.76	7.85	6.68	5.46	4.72	3.02	1.98
160.00	8.83	7.89	6.71	5.50	4.81	3.11	2.13
160.02	8.81	7.95	6.82	5.58	4.87	3.14	2.11
160.12	8.52	7.68	6.42	5.28	4.54	2.91	2.01
160.22	8.34	7.36	6.54	5.18	4.37	2.68	1.83
160.32	8.51	7.47	6.20	5.01	4.26	2.68	1.82
160.42	9.22	8.02	6.62	5.24	4.44	2.75	1.90
160.52	9.79	8.53	6.77	5.32	4.29	2.61	1.93
160.62	10.88	9.08	7.15	5.31	4.31	2.64	1.87
160.72	10.44	8.85	7.05	5.28	4.34	2.59	1.84
160.82	12.14	9.77	7.56	5.54	4.46	2.59	1.89
162.00	10.68	8.72	7.05	5.28	4.38	2.68	1.88
164.00	13.47	10.69	8.73	6.40	5.09	2.91	1.81
166.00	12.50	10.20	8.47	6.51	5.24	3.41	2.40
168.00	13.39	11.23	9.34	7.40	6.07	4.15	2.63
170.00	15.34	12.29	10.03	7.63	6.12	4.10	2.49

Normalized Deflections - Southbound, Centerline, Wearing Course - 04/05/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	11.57	8.51	6.83	5.27	3.90	2.24	1.34
154.00	12.76	9.48	6.93	4.70	3.64	2.04	0.65
156.00	14.25	10.56	8.13	6.00	4.54	2.62	0.97
158.00	12.43	9.43	7.38	5.77	4.66	3.03	1.23
158.82	14.76	9.79	7.66	5.71	4.71	2.84	1.75
158.92	10.54	7.83	6.53	4.78	3.98	2.35	1.02
159.02	10.10	7.99	6.27	4.77	3.88	2.39	0.98
159.12	11.17	9.20	7.54	5.95	4.81	3.07	1.13
159.22	12.28	9.51	7.80	5.99	4.79	2.91	1.24
159.32	10.72	8.60	7.12	5.26	4.29	2.59	2.13
159.42	11.90	9.29	7.28	5.78	4.44	2.96	2.45
159.52	10.98	8.24	6.82	5.52	4.57	2.75	1.72
159.62	11.54	8.83	7.84	6.02	4.72	2.73	1.51
159.72	9.73	7.84	6.23	5.01	4.32	2.45	1.39
159.82	10.50	7.96	6.92	5.42	4.52	2.47	2.00
159.92	11.79	9.33	7.29	5.70	4.63	2.63	1.28
160.00	10.60	8.46	6.79	5.20	4.34	2.89	1.83
160.02	10.57	8.27	6.71	5.55	4.78	3.25	1.89
160.12	10.14	8.44	7.27	5.92	4.95	3.44	1.72
160.22	10.08	8.18	7.02	5.81	4.89	3.13	1.67
160.32	10.09	8.19	6.84	5.50	4.57	2.69	1.30
160.42	9.67	7.60	6.45	5.21	4.11	2.41	1.59
160.52	8.11	6.60	5.41	4.47	3.86	2.36	1.48
160.62	7.74	5.98	5.20	4.19	3.44	2.18	1.39
160.72	8.93	6.91	5.61	4.38	3.44	1.98	1.40
160.82	10.75	7.94	6.05	4.43	3.65	2.25	1.14
162.00	14.32	10.45	7.65	5.38	4.16	2.50	1.51
164.00	13.02	9.10	6.56	4.74	3.64	1.97	1.01
166.00	15.19	10.40	8.25	6.32	5.29	3.43	1.81
168.00	15.65	11.47	9.02	7.12	5.93	4.09	2.44
170.00	14.67	10.72	8.66	6.90	5.92	3.95	2.28

Normalized Deflections - Southbound, Wheelpath, Wearing Course - 04/05/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	11.88	9.55	7.79	5.89	4.68	2.88	1.46
154.00	10.48	8.35	6.58	4.82	3.75	2.18	1.08
156.00	11.45	9.36	7.58	5.63	4.40	2.56	1.15
158.00	10.76	9.19	7.59	5.87	4.88	3.20	1.90
158.82	11.39	8.72	6.97	5.39	4.55	2.97	1.92
158.92	8.49	7.23	6.02	4.81	3.98	2.53	1.58
159.02	8.26	6.94	5.69	4.56	3.81	2.52	1.52
159.12	8.48	7.41	6.32	5.08	4.46	3.05	1.69
159.22	10.68	9.23	7.60	6.11	5.19	3.34	1.95
159.32	9.09	7.91	6.61	5.34	4.45	2.90	1.85
159.42	9.28	8.08	6.74	5.50	4.58	3.12	1.84
159.52	8.85	7.77	6.49	5.12	4.24	2.82	1.71
159.62	11.33	9.70	7.95	6.18	4.77	2.91	1.65
159.72	9.30	7.82	6.35	5.06	4.05	2.64	1.60
159.82	9.62	8.26	6.78	5.40	4.33	2.82	1.76
159.92	10.55	9.04	7.52	5.98	4.74	2.97	1.76
160.00	8.63	7.64	6.43	5.16	4.34	2.95	1.93
160.02	8.91	7.66	6.39	5.28	4.42	3.16	2.16
160.12	8.58	8.05	6.90	5.80	4.92	3.43	2.06
160.22	8.29	7.57	6.46	5.52	4.75	3.41	1.79
160.32	8.63	7.48	6.35	5.19	4.56	3.02	1.73
160.42	8.64	7.55	6.39	5.23	4.33	2.92	1.66
160.52	7.74	6.97	5.87	4.95	4.18	2.84	1.62
160.62	6.96	6.51	5.49	4.58	3.80	2.61	1.52
160.72	8.39	7.51	6.16	4.88	3.99	2.64	1.48
160.82	9.77	7.96	6.19	4.74	3.81	2.54	1.49
162.00	12.63	10.46	8.20	6.14	4.70	2.94	1.67
164.00	11.50	9.30	7.24	5.40	4.11	2.60	1.56
166.00	14.44	12.25	9.88	7.69	6.07	3.92	2.20
168.00	12.15	10.71	8.88	7.31	5.99	4.13	2.43
170.00	12.87	10.86	8.90	7.32	6.00	4.19	2.57

Normalized Deflections - Northbound, Centerline, Wearing Course - 04/26/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	11.10	9.44	7.74	5.99	4.53	2.70	1.17
154.00	15.30	12.80	9.94	7.16	5.30	3.09	1.36
156.00	15.20	12.70	9.84	6.92	5.13	3.05	1.16
158.00	11.60	9.57	7.58	5.63	4.39	3.01	1.49
158.82	8.97	8.10	6.83	5.39	4.47	3.40	1.80
158.92	8.61	7.99	6.63	5.45	4.69	3.30	1.88
159.02	8.46	7.80	6.61	5.31	4.40	3.12	1.88
159.12	9.35	8.59	7.27	5.84	4.79	3.17	1.64
159.22	9.29	8.35	6.88	5.54	4.46	2.99	1.72
159.32	8.15	7.39	6.21	4.93	4.03	2.73	1.68
159.42	8.69	7.74	6.52	5.28	4.33	2.91	1.72
159.52	8.88	8.01	6.68	5.44	4.40	3.03	1.73
159.62	8.62	8.24	7.05	5.62	4.64	3.11	1.70
159.72	9.38	8.85	7.12	5.79	4.76	3.02	1.75
159.82	8.31	7.58	6.50	5.44	4.53	3.11	1.68
159.92	8.03	7.59	6.30	5.03	4.13	2.82	1.57
160.00	8.37	7.74	6.49	5.30	4.33	3.05	1.80
160.02	8.41	7.59	6.58	5.32	4.36	2.99	1.82
160.12	7.76	7.15	5.80	4.87	4.01	2.78	1.61
160.22	8.00	7.35	5.90	4.43	3.54	2.42	1.37
160.32	7.90	7.33	5.85	4.51	3.75	2.73	1.45
160.42	8.33	7.27	5.96	4.75	3.75	2.53	1.49
160.52	8.21	7.61	6.21	4.82	3.74	2.55	1.40
160.62	9.47	8.24	6.53	4.77	3.74	2.43	1.49
160.72	9.31	8.22	6.45	4.91	3.70	2.67	1.38
160.82	11.10	9.41	6.93	4.94	3.80	2.42	1.40
162.00	9.73	8.31	6.43	4.93	3.89	2.55	1.43
164.00	13.80	11.00	8.50	6.13	4.54	2.75	1.30
166.00	12.00	9.97	7.93	6.11	4.88	3.33	1.98
168.00	14.00	11.60	9.51	7.44	6.00	4.24	2.41
170.00	15.10	12.50	9.98	7.72	6.08	4.21	2.34

Normalized Deflections - Northbound, Wheelpath, Wearing Course - 04/26/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	13.17	11.24	9.41	7.35	5.49	3.28	1.38
154.00	17.32	14.49	11.54	8.61	6.36	3.91	1.90
156.00	15.88	13.08	10.03	6.91	5.13	3.07	1.27
158.00	11.87	9.76	7.55	5.70	4.44	2.92	1.50
158.82	9.38	8.69	7.31	5.69	4.65	3.06	1.85
158.92	8.82	8.04	6.86	5.79	4.83	3.41	1.92
159.02	9.09	8.04	6.88	5.72	4.73	3.30	1.91
159.12	9.80	9.21	7.73	6.15	5.06	3.41	1.87
159.22	10.00	8.62	7.31	5.96	4.83	3.16	1.78
159.32	9.21	8.18	7.01	5.79	4.63	3.04	1.61
159.42	9.28	8.38	7.11	5.83	4.76	3.25	1.71
159.52	9.33	8.20	7.01	5.79	4.65	3.10	1.72
159.62	9.12	8.20	7.12	5.76	4.70	3.06	1.68
159.72	9.77	8.95	7.68	6.25	5.10	3.31	1.81
159.82	9.32	8.49	7.34	6.02	4.86	3.19	1.88
159.92	8.61	7.96	6.89	5.52	4.48	2.99	1.66
160.00	8.76	8.11	6.90	5.41	4.40	2.93	1.66
160.02	9.07	8.16	6.91	5.54	4.50	2.97	1.56
160.12	8.65	7.70	6.57	5.42	4.41	2.95	1.68
160.22	8.99	8.44	6.75	5.45	4.34	2.80	1.55
160.32	8.19	7.44	6.34	5.11	4.17	2.79	1.55
160.42	8.59	7.76	6.44	5.13	4.13	2.68	1.56
160.52	8.88	8.29	6.56	5.09	4.06	2.60	1.52
160.62	9.88	8.45	6.80	5.07	4.04	2.62	1.58
160.72	10.34	8.43	6.70	5.16	3.91	2.62	1.55
160.82	11.35	9.77	7.27	5.23	4.04	2.67	1.48
162.00	11.29	9.39	7.52	5.49	4.31	2.86	1.60
164.00	14.56	11.66	9.36	7.00	5.32	3.23	1.58
166.00	13.86	11.52	9.50	7.21	5.56	3.69	2.01
168.00	15.44	12.23	9.92	7.95	6.19	4.25	2.31
170.00	15.40	12.55	10.23	7.94	6.30	4.15	2.48

Normalized Deflections - Southbound, Centerline, Wearing Course - 04/26/94

Station	Offset from Center of Load, inches						
	D ₀	D ₁		D ₂		D ₃	
	0	8	12	18	24	36	60
152.00	11.97	9.48	7.29	5.16	3.85	2.34	1.14
154.00	10.87	8.40	6.22	4.29	3.14	1.85	0.83
156.00	13.05	10.34	7.80	5.41	4.00	2.33	0.91
158.00	12.32	9.65	7.39	5.36	4.23	2.84	1.76
158.82	12.25	9.41	7.44	5.51	4.30	2.83	1.71
158.92	9.82	7.92	6.30	4.66	3.56	2.32	1.37
159.02	10.34	8.18	6.42	4.69	3.62	2.32	1.50
159.12	11.51	9.24	7.37	5.51	4.29	2.77	1.40
159.22	11.74	9.50	7.49	5.60	4.35	2.92	1.60
159.32	11.08	8.94	6.98	5.28	4.12	2.77	1.50
159.42	11.61	9.20	7.25	5.42	4.26	2.90	1.74
159.52	10.87	8.83	6.99	5.27	4.12	2.76	1.61
159.62	13.25	10.91	8.60	6.23	4.62	2.81	1.61
159.72	11.23	8.90	6.91	5.08	3.91	2.63	1.59
159.82	11.71	9.29	7.39	5.50	4.25	2.81	1.71
159.92	12.34	9.90	7.90	5.74	4.35	2.80	1.65
160.00	11.00	8.75	6.90	5.18	4.13	2.88	1.85
160.02	10.77	8.82	7.06	5.47	4.44	3.17	1.96
160.12	10.65	8.93	7.45	6.03	4.96	3.58	2.01
160.22	10.28	8.76	7.26	5.80	4.73	3.20	1.70
160.32	9.57	8.32	6.78	5.31	4.29	2.92	1.62
160.42	10.13	8.22	6.63	5.07	4.06	2.76	1.57
160.52	8.81	7.41	6.09	4.82	3.89	2.67	1.60
160.62	8.40	7.02	5.67	4.42	3.56	2.45	1.45
160.72	9.70	7.82	6.09	4.58	3.55	2.41	1.40
160.82	11.04	8.44	6.25	4.52	3.57	2.47	1.41
162.00	13.53	10.33	7.77	5.56	4.30	2.91	1.63
164.00	13.46	9.84	7.07	4.91	3.74	2.47	1.38
166.00	13.43	10.74	8.42	6.51	5.23	3.64	2.07
168.00	14.75	11.84	9.22	7.07	5.86	4.23	2.50
170.00	14.29	11.35	8.98	7.12	5.95	4.32	2.62

Normalized Deflections - Southbound, Wheelpath, Wearing Course - 04/26/94

Station	Offset from Center of Load, inches						
	D ₀	D ₁		D ₂		D ₃	
	0	8	12	18	24	36	60
152.00	14.31	11.83	9.59	7.60	6.19	4.40	2.55
154.00	14.22	11.74	9.44	7.39	6.05	4.29	2.46
156.00	15.61	12.71	10.03	7.55	5.97	3.97	2.08
158.00	13.17	10.35	7.72	5.40	4.07	2.72	1.47
158.82	15.81	12.30	9.18	6.45	4.86	3.15	1.65
158.92	11.10	8.79	6.86	5.15	4.04	2.76	1.62
159.02	10.00	8.46	6.94	5.38	4.25	2.76	1.56
159.12	8.55	7.46	6.32	5.12	4.16	2.80	1.55
159.22	9.01	7.75	6.56	5.36	4.35	2.96	1.60
159.32	9.66	8.23	6.86	5.50	4.51	3.06	1.74
159.42	9.41	8.09	6.74	5.46	4.44	3.10	1.68
159.52	9.36	8.12	6.85	5.63	4.69	3.26	1.71
159.62	9.73	8.41	7.10	5.77	4.83	3.50	2.06
159.72	9.37	7.80	6.53	5.30	4.41	3.29	2.11
159.82	9.21	7.74	6.43	5.12	4.16	3.02	2.00
159.92	11.90	10.04	8.05	6.21	4.78	3.06	1.80
160.00	10.51	9.12	7.39	5.57	4.28	2.81	1.74
160.02	9.78	8.21	6.62	5.11	4.01	2.63	1.53
160.12	12.41	10.23	8.38	6.31	4.72	2.88	1.58
160.22	9.74	8.13	6.79	5.29	4.20	2.69	1.66
160.32	10.13	8.61	7.06	5.63	4.48	2.97	1.69
160.42	9.74	8.22	6.69	5.31	4.27	2.81	1.69
160.52	10.65	9.10	7.61	6.02	4.76	3.17	1.76
160.62	8.72	7.47	6.16	4.95	3.99	2.72	1.53
160.72	8.53	7.12	5.91	4.57	3.60	2.36	1.47
160.82	8.51	7.10	5.84	4.64	3.68	2.37	1.32
162.00	10.78	8.32	6.71	5.18	4.07	2.72	1.73
164.00	10.93	9.07	7.18	5.60	4.45	3.01	1.62
166.00	12.60	9.63	7.63	5.65	4.28	2.62	0.97
168.00	10.97	8.40	6.72	4.76	3.46	2.11	0.93
170.00	15.28	12.03	9.34	6.93	5.19	3.23	1.52

Normalized Deflections - Northbound, Centerline, Wearing Course - 10/13/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	9.49	7.04	5.66	4.10	3.01	1.77	0.84
154.00	10.48	6.81	5.38	3.87	2.87	1.80	0.82
156.00	11.11	7.07	5.58	4.06	3.06	1.98	0.98
158.00	8.96	5.92	4.88	3.90	3.16	2.22	1.22
158.82	7.34	5.50	4.75	3.78	3.14	2.21	1.38
158.92	9.10	6.94	6.01	4.83	3.94	2.77	1.61
159.02	8.15	6.20	5.37	4.31	3.52	2.36	1.42
159.12	8.05	6.07	5.28	4.15	3.32	2.14	1.03
159.22	8.54	6.32	5.34	4.20	3.34	2.25	1.33
159.32	7.48	5.37	4.48	3.44	2.71	1.79	1.16
159.42	8.58	6.46	5.44	4.23	3.36	2.26	1.31
159.52	7.79	5.51	4.67	3.59	2.86	1.88	1.19
159.62	9.14	6.74	5.74	4.49	3.54	2.34	1.29
159.72	7.96	5.94	5.06	3.93	3.11	2.01	1.17
159.82	6.93	5.27	4.60	3.67	3.01	2.16	1.31
159.92	6.43	4.60	3.97	3.13	2.52	1.78	1.13
160.00	6.47	4.63	3.96	3.15	2.57	1.91	1.20
160.02	6.83	5.20	4.50	3.58	2.95	2.12	1.37
160.12	7.44	5.44	4.65	3.69	3.00	2.20	1.42
160.22	7.23	5.14	4.26	3.28	2.59	1.83	1.17
160.32	6.82	4.78	3.95	3.01	2.36	1.66	0.98
160.42	7.07	4.93	4.08	3.12	2.48	1.72	1.06
160.52	7.48	5.10	4.18	3.18	2.54	1.72	1.07
160.62	7.57	5.18	4.19	3.14	2.52	1.71	1.07
160.72	7.64	5.12	4.16	3.10	2.44	1.65	1.02
160.82	8.34	5.28	4.15	3.05	2.41	1.65	1.03
162.00	8.74	5.77	4.63	3.46	2.72	1.86	1.13
164.00	14.19	9.22	7.02	4.94	3.64	2.26	1.23
166.00	11.67	8.14	6.55	4.93	3.95	2.71	1.60
171.00	13.80	9.54	7.88	6.16	5.05	3.66	2.13

Normalized Deflections - Northbound, Wheelpath, Wearing Course - 10/13/94

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	10.31	7.94	6.60	4.98	3.75	2.11	0.99
154.00	11.00	7.23	5.76	4.24	3.18	1.97	1.03
156.00	12.08	8.15	6.38	4.61	3.42	2.15	1.07
158.00	9.10	6.71	5.63	4.42	3.55	2.40	1.31
158.82	8.43	6.26	5.36	4.25	3.42	2.40	1.37
158.92	7.70	6.18	5.53	4.58	3.78	2.65	1.52
159.02	7.78	6.28	5.50	4.50	3.68	2.54	1.44
159.12	8.30	6.52	5.69	4.53	3.67	2.25	1.23
159.22	8.81	6.96	6.01	4.76	3.80	2.52	1.35
159.32	7.63	5.86	5.09	4.06	3.27	2.17	1.17
159.42	8.49	6.82	5.95	4.79	3.87	2.56	1.33
159.52	7.10	5.48	4.74	3.69	2.96	1.94	1.15
159.62	8.18	6.42	5.62	4.51	3.62	2.36	1.24
159.72	7.98	6.22	5.41	4.28	3.41	2.24	1.28
159.82	7.11	5.64	4.90	3.93	3.20	2.16	1.21
159.92	7.12	5.71	5.02	4.08	3.33	2.25	1.31
160.00	7.18	5.85	5.08	4.14	3.36	2.30	1.35
160.02	7.16	5.72	4.98	3.99	3.22	2.21	1.33
160.12	7.34	5.82	5.08	4.05	3.30	2.23	1.33
160.22	7.76	6.25	5.42	4.26	3.36	2.11	1.23
160.32	7.11	5.65	4.90	3.90	3.14	2.03	1.13
160.42	8.01	6.32	5.48	4.35	3.46	2.25	1.26
160.52	8.22	6.45	5.49	4.29	3.36	2.23	1.28
160.62	8.30	5.86	4.85	3.69	2.87	1.98	1.19
160.72	8.28	6.15	5.10	3.85	3.01	1.97	1.16
160.82	9.24	6.26	5.05	3.81	2.96	2.00	1.19
162.00	13.47	10.33	8.34	6.14	4.77	3.14	1.74
162.00	9.30	7.01	5.85	4.49	3.53	2.31	1.29
163.00	10.62	7.45	5.96	4.31	3.24	2.05	1.25
164.00	13.07	9.38	7.76	6.01	4.81	3.31	1.97
164.00	12.62	9.12	7.39	5.38	4.05	2.46	1.34
165.00	13.88	9.71	7.75	5.65	4.22	2.71	1.55
166.00	12.80	9.52	7.91	6.14	4.86	3.36	1.87
168.00	12.24	9.22	7.74	6.14	4.98	3.52	2.06
170.00	13.58	10.49	8.75	6.72	5.29	3.58	2.13
171.00	10.15	7.75	6.52	5.08	4.07	2.78	1.67

**Normalized Deflections - Southbound, Centerline, Wearing Course -
10/13/94**

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	8.15	5.76	4.70	3.54	2.68	1.69	0.85
154.00	7.89	5.43	4.28	3.11	2.34	1.38	0.74
156.00	8.38	5.91	4.85	3.65	2.85	1.73	0.81
158.00	8.32	5.93	4.91	3.82	3.11	2.21	1.35
158.82	9.72	7.07	5.92	4.55	3.51	2.23	1.24
158.92	7.86	5.90	4.87	3.70	2.84	1.89	1.14
159.02	7.27	5.32	4.44	3.37	2.62	1.66	1.04
159.12	7.62	5.71	4.85	3.80	3.02	2.02	1.11
159.22	8.20	6.29	5.35	4.20	3.37	2.22	1.32
159.32	8.46	6.29	5.34	4.17	3.31	2.20	1.28
159.42	8.50	5.99	5.03	3.90	3.12	2.09	1.22
159.52	8.29	6.15	5.17	4.04	3.21	2.13	1.26
159.62	8.11	6.55	5.59	4.33	3.36	2.09	1.21
159.72	7.71	5.68	4.77	3.67	2.87	1.88	1.12
159.82	7.66	5.63	4.78	3.71	2.92	1.87	1.16
159.92	7.51	5.46	4.58	3.55	2.78	1.88	1.18
160.00	7.06	5.39	4.63	3.75	3.09	2.26	1.41
160.02	6.85	5.23	4.50	3.71	3.08	2.29	1.45
160.12	7.42	5.85	5.18	4.35	3.69	2.70	1.58
160.22	7.23	5.73	5.07	4.24	3.55	2.50	1.36
160.32	7.06	5.51	4.79	3.90	3.18	2.15	1.34
160.42	6.64	4.94	4.24	3.43	2.80	1.94	1.10
160.52	5.47	4.13	3.59	2.95	2.45	1.75	1.05
160.62	5.44	4.06	3.52	2.87	2.35	1.68	1.00
160.72	6.22	4.83	4.12	3.27	2.64	1.76	1.06
160.82	7.82	5.53	4.46	3.38	2.68	1.84	1.12
162.00	9.99	6.82	5.46	4.09	3.15	2.07	1.25
164.00	10.10	6.86	5.33	3.83	2.84	1.80	1.04
166.00	11.58	8.25	6.80	5.41	4.38	3.07	1.77
168.00	11.24	8.04	6.73	5.42	4.41	3.25	1.95
170.00	10.89	7.88	6.65	5.40	4.49	3.29	2.03
171.00	11.66	8.37	6.94	5.56	4.50	3.22	1.96

**Normalized Deflections - Southbound, Wheelpath, Wearing Course -
10/13/94**

Station	Offset from Center of Load, inches						
	D ₀		D ₁		D ₂	D ₃	
	0	8	12	18	24	36	60
152.00	10.39	7.43	6.07	4.63	3.57	2.25	1.13
154.00	8.69	5.84	4.60	3.35	2.49	1.51	0.76
156.00	9.01	6.16	4.95	3.71	2.83	1.67	0.76
158.00	8.56	6.17	5.06	4.02	3.25	2.33	1.42
158.82	9.84	6.61	5.33	4.05	3.11	2.01	1.24
158.92	7.76	5.87	4.95	3.87	3.00	1.97	1.16
159.02	7.63	5.86	4.94	3.91	3.11	2.08	1.17
159.12	7.92	6.12	5.27	4.32	3.56	2.48	1.32
159.22	9.23	7.98	6.89	5.56	4.44	2.90	1.58
159.32	8.82	6.84	5.89	4.72	3.82	2.54	1.49
159.42	8.92	6.72	5.71	4.53	3.57	2.36	1.42
159.52	8.72	6.54	5.57	4.39	3.45	2.27	1.29
159.62	11.18	8.18	6.77	5.09	3.79	2.29	1.28
159.72	8.85	6.61	5.49	4.25	3.29	2.18	1.28
159.82	9.48	6.79	5.56	4.17	3.13	2.08	1.33
159.92	10.99	8.36	6.89	5.22	4.06	2.70	1.63
160.00	8.05	5.92	5.00	4.01	3.29	2.41	1.59
160.02	8.28	6.09	5.19	4.22	3.50	2.65	1.72
160.12	9.05	7.07	6.16	5.14	4.26	3.08	1.77
160.22	7.89	6.23	5.42	4.51	3.76	2.59	1.40
160.32	9.66	7.94	6.87	5.55	4.50	3.03	1.58
160.42	8.54	6.45	5.55	4.50	3.65	2.44	1.31
160.52	7.82	6.09	5.32	4.36	3.56	2.44	1.31
160.62	7.11	5.50	4.79	3.87	3.16	2.14	1.24
160.72	8.28	6.37	5.42	4.29	3.39	2.26	1.25
160.82	8.92	6.02	5.00	4.02	3.25	2.25	1.27
162.00	12.35	7.97	6.34	4.69	3.55	2.29	1.27
164.00	12.25	8.19	6.39	4.63	3.46	2.23	1.35
166.00	13.88	10.27	8.57	6.70	5.24	3.53	1.91
168.00	11.93	8.83	7.48	6.05	4.87	3.44	2.00
170.00	11.90	8.37	7.21	5.83	4.82	3.42	2.10
171.00	13.09	9.36	7.78	6.14	4.86	3.34	1.98

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13. ABSTRACT (Maximum 200 words) Report documents a study on the successful use of grate ash from a waste-to-energy facility for aggregate substitute in an asphalt binder course pavement as part of a demonstration project. As part of the demonstration, a test section and a control section were built. the report concludes that the use of grate as or bottom ash from waste-to-energy facilities in pavement is an acceptable use of this light-weight aggregate substitute.					
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