Didaktische Anregungen zum Thema „Schmalspurbahn“ für die Altersstufen ab 14 Jahren

A. Zur Entwicklung der Bahnen (Historie)

Quellen: https://de.wikipedia.org/wiki/Geschichte_der_Eisenbahn

- Schienensysteme — Holzräder auf Holzbalken (Bergwerke), Metallräder auf metallbeschlagenen Holzbalken
  Metallräder auf Ganzmetallschienen
- Antriebsysteme — Mensch schob Loren (Bergwerk, Landwirtschaft)
  Pferd zog Loren und Waggons
  Dampflok (in Bergwerken später Pressluft)
  Dampfspeicher (für explosionsgefährdete Betriebe)
  Elekrolok, (in Bergwerken Akkulok), Solarantrieb
  Verbrennungsmotorlokomotiven (Benzol/Diesel)
- Waggons — Güter auf Holzwaggons (Bergwerke)
  Güter auf Waggons aus Metall mit Holzaufbau
  Güter auf Waggons aus Ganzmetall
  Personen in Kutschen mit Eisenbahnrädern
  Personenwagen aus Holzaufbau + Metallrahmen
  Personenwagen aus selbsttragenden Karosserien

B. Entwicklung der Infrastruktur der Bahn in Deutschland

Zuerst wurden die Industrie- und Handelszentren sowie größere Städte untereinander verbunden. Später wurden auch Gebiete außerhalb der Zentren erschlossen. Aus Kostengründen hat man sich mancherorts für die kosten-günstigere Schmalspur entschieden. Durchschnittliches Anlagekapital 1880 je km Regelspur = 111.362 Mark; Schmalspurbahnen kamen bei 1 km Strecke auf 23.085 Mark

Quelle: https://bahn-extra.de/leseprobe/die-entwicklung-der-schmalspurbahnen-deutschland

Die günstigste Variante der Schmalspur war die 600-mm-Spurweite, die heute noch in Torfwerken zur Anwendung kommt. Durch den zunehmenden Individualverkehr auf ausgebauten Straßen gerieten die Schmalspurbahnen mit ihrem Umladen/Auf- oder Abschemeln an den Schnittstellen zur Regelspur ins Hintertreffen und wurden nach und nach eingestellt. Die letzten öffentlichen Bahnen in 600-mm-Spurweite verkehrten bis 1993 im heute zu Polen gehörenden Posener Raum. Als Touristenattraktion fahren heute einige Bahnen wie z.B. die Böhmetal-Kleinbahn, Parkeisenbahnen und kleinere Museumsbahnen und Sammlungen.

Quelle: www.klbg-ggmbh.de/wirtschaftlichkeit.html

C. Vergleich Streckennetze Deutschland und Schweiz

- In der Schweiz gibt es heute noch Schmalspurbahnen, die sowohl Güter als auch Personen befördern (Glacier-Express). Der Grund liegt einerseits in der Topographie und andererseits am politischen Willen.

Quelle:https://www.planet-wissen.de/technik/verkehr/geschichte_der_eisenbahn/index.html ; https://www.rhb.ch/de/unternehmen/kennzahlen/gueterverkehr

D. Ökonomie und Ökologie im Vergleich Schiene – Straße – Wasser – Luft

- Berechnungen anstellen im Personen- und Warenverkehr unter Berücksichtigung der Erstellungs- und Unterhaltungskosten für die Verkehrswege.

Quelle:https://www.jstor.org/stable/23422823?seq=1#page_scan_tab_contents
WASHO-Road-Test: http://onlinepubs.trb.org/Onlinepubs/hrbbulletin/177/177-003.pdf

Die Entwicklung der Schmalspurbahnen in Deutschland

Ende des 19. Jahrhunderts setzte in Deutschland ein Bauboom ein, der zig Schmalspurbahnen entstehen ließ. Diese Vielfalt brachte den Regionen Anschluss an die »weite Welt«. Text: Wolf-Dietger Machel

© H. Brinker Dampflok der Mecklenburg-Pommerschen Schmalspurbahn (600mm-Spur)

Systematisch trieben Europas Staaten seit Mitte des 19. Jahrhunderts den Bau von Eisenbahnen mit der Regelspurweite 1.435 Millimeter voran. Das ermöglichte einen zeitgemäßen Personen- und Güterverkehr zwischen Industrie- und Handelszentren einerseits und größeren Städten andererseits. Daneben sollte die Eisenbahn aber auch Gebiete außerhalb dieser Zentren erschließen. Die hierfür gebauten »Zweigbahnen« spielten aufgrund des geringeren Verkehrsaufkommens oftmals nicht die Baukosten ein.

Internationale Erfahrungen zeigten, dass Eisenbahnen im Bau und Betrieb billiger sein konnten, wenn man eine geringere Spurweite wählt. In England gab es seit 1832 eine Schmalspurbahn mit der Spurweite 597 Millimeter, von 1828 bis 1836 entstand in Österreich die Strecke Budweis – Linz mit der Spurweite 1.106 Millimeter. In beiden Fällen zogen Pferde die Wagen.

785-Millimeter-Spur
Die Geburtsstunde der deutschen Schmalspurbahnen schlug am 24. März 1851 im heute zu Polen gehörenden Oberschlesien. An jenem Tage konzessionierte die preußische Regierung den Bau von Strecken in einer Spurweite von 785 Millimetern (2,5 Fuß). Die Strecken wurden 1855 eröffnet und zunächst mit Pferden betrieben; als zwei Jahre später die ersten Dampflokomotiven zum Einsatz kamen, war das Schmalspurnetz bereits auf 134,5 Kilometer Länge gewachsen. Im Juni 1904 übernahmen die Königlich Preußischen Staatseisenbahnen die Strecken. Teile des Netzes blieben bis in die 1990er-Jahre in Betrieb.

Als zweite deutsche Schmalspurbahn entstand ab 1862 die Bröltalbahn. Sie erschloss mit ihrem später auf 88 Kilometer ausgebauten Streckennetz verschiedene Hütten- und Bergwerksstandorte – ähnlich wie in Oberschlesien. Das 1921 in Rhein-Sieg-Eisenbahn umbenannte Unternehmen hatte bereits 1863 den Lokomotivbetrieb und 1872 den Personenverkehr eingeführt. Nachdem 1956 der Personenverkehr eingestellt worden war, endete der Güterverkehr auf der verbliebenen Reststrecke Beuel – Eulenburg am 27. Mai 1967. Damit existierte die zweitälteste deutsche Schmalspurbahn immerhin 105 Jahre.
Das durchschnittliche Anlagekapital betrug in Deutschland anno 1880 je Kilometer Normalspurbahn 111.362 Mark. Bei der Bröltalbahn kamen auf einen Kilometer Strecke lediglich 23.085 Mark. Der Bau und Betrieb von Schmalspurbahnen war also wesentlich günstiger. Andererseits musste man höhere Aufwendungen in Kauf nehmen, wenn Fracht über den Einzugsbereich dieser Bahnen transportiert wurde; sie musste zwischen Schmal- und Normalspur umgeladen werden. Wohl auch deshalb blieben die beiden Erstlinge bis 1879 allein auf weiter Flur.

Der Durchbruch gelingt
Aber auf Dauer sprach der Kostenvorteil doch für die Schmalspurbahnen. Besonders landwirtschaftlich geprägte Gebiete benötigten für die Weiterentwicklung Eisenbahnanschlüsse, und immer öfter fehlte in solchen Fällen das Geld für Normalspurstrecken. Und so billigte die am 1. Juli 1878 reichsweit in Kraft getretene »Bahnordnung für deutsche Eisenbahnen untergeordneter Bedeutung« auch den Bau von Schmalspurbahnen mit den Spurweiten 750 und 1.000 Millimeter zu.

In den folgenden Jahren setzte ein regelrechter Schmalspurbahnboom ein. So zum Beispiel im Königreich Sachsen, dessen Regierung 1879 beschloss, den Bau von Bahnen mit 750 Millimetern Spurweite anstelle der etwas teureren »vollspurigen Secundairbahnen« in Angriff zu nehmen. Nach Inbetriebnahme der Linie Wilkau – Kirchberg entstanden zahlreiche Schmalspurstrecken und -netze.

Als 1923 die letzte Linie fertig gestellt wurde, gab es in Sachsen 542 Kilometer Schmalspurbahnen. Das Königreich Preußen wiederum überließ die Schmalspurbahnen weitgehend Kreisen, Kommunen und Einzelinteressenten. Grundlage hierfür war das am 28. Juli 1892 verabschiedete »Gesetz über die Kleinbahnen und Privatanschlußbahnen«. Es ermöglichte sowohl den Bau und Betrieb von Normalspurbahnen als auch von Schmalspurbahnen, vorzugsweise mit den Spurweiten 600, 750 und 1.000 Millimeter.

Der Begriff »Kleinbahn« bezog sich dabei nicht auf die kleineren Spurweiten, sondern auf die mit dem Gesetz festgelegten kleineren (besser: beschränkten) Verkehrsbefugnisse, die insbesondere den Durchgangsgüterverkehr auf derartigen Bahnen ausschlossen. Damit wollte Preußen verhindern, dass die Kleinbahnen den Staatseisenbahnen Transporte abspenstig machten.
Tatsächlich zeigte das Gesetz Wirkung. Nach 1892 entstanden in Preußen zahlreiche schmalspurige Kleinbahnen, die bis 1913 eine Länge von 6.349 Kilometern erreichen sollten. Damit besaß Preußen im Vergleich zu den anderen Bundesstaaten des Deutschen Reichs mit Abstand den größten Anteil an Schmalspurbahnen. Die Kleinbahngesetzgebung wurde auch von anderen Ländern übernommen, allerdings in modifizierter Fassung und ohne den Durchgangsgüterverkehr grundsätzlich zu verbieten.

Spurweitenvielfalt
Ungeachtet der in Deutschland bevorzugten Spurweiten 600, 750 und 1.000 Millimeter wurden Ausnahmen genehmigt. Neben der 785-Millimeter-Spur setzte sich auf Grund besonderer örtlicher Verhältnisse für den öffentlichen Verkehr auch die 900-Millimeter-Spur durch, wie auf der Insel Borkum, an der Ostseeküste für den von Bad Doberan ausgehenden und weithin bekannten »Molli« und für die Spessartbahn. Hinzu kamen Ausnahmen wie die 800-Millimeter-spurige Ernstbahn von Braunsfeld nach Philippstein.

Zu den Besonderheiten bei einigen Schmalspurbahnen zählten drei- und vierschienige Streckenabschnitte. Hier war es möglich, auch normalspurige Wagen zu befördern. Sie wurden mit Hilfe schmalspuriger Zwischenwagen von Schmalspurlokomotiven gezogen (so bei Kröslin – Wolgast in Vorpommern und den Kleinbahnen des Kreises Jerichow I bei Magdeburg) oder man setzte zusätzlich Normalspurloks ein (wie bei der Nordhausen-Wernigeroder Eisenbahn und der Strecke Freital-Potschappel – Freital-Zauckerode).

Hinzu kamen Teilstrecken, auf denen eine dritte bzw. vierte Schiene verlegt war, die aber separat im Normal- und Schmalspurbetrieb befahren wurden (etwa die Mitnutzung der Nebenbahn Flöha — Bärenstein durch die Schmalspurbahn nach Jöhstadt im Bereich Wolkenstein, Teile der Mindener Kreisbahnen). Allgemein lassen sich bei den Spurweiten aber landesbezogene Tendenzen erkennen. Neben Sachsen bevorzugte Württemberg die 750 Millimeter, Bayern und Baden setzten auf 1.000 Millimeter. Auffallend ist zugleich, dass Sachsen konsequent die Schmalspurbahnen durch die eigenen Staatseisenbahnen bauen und betreiben ließ. Eine Ausnahme machte nur die 1890 gebaute, 1906 verstaatlichte Zittau-Oybin-Jonsdorfer Eisenbahn.

Ein Artikel aus Bahn Extra 03/08

„Sterbehilfe“ Rentable Zweigstrecken

(Auszug aus dem Artikel „Fünf Rote aus Mosbach (und eine aus Kiel)“ aus Eisenbahn Geschichte Heft 89, August/September 2018, 16.Jahrgang) von Hans-Joachim Knupfer

Entgegen heutiger allgemeiner Lesart war es weder der Lkw, der damals im ländlichen Raum die Existenz der Bahnen bedrohte, noch war es günstiger, noch waren die Kosten der Zweigstrecken ein reales Problem für die Bundesbahn. Der Stuttgarter Verkehrswirtschaftler Carl Pirath (1884-1955) wies seinerzeit nach, dass die – damals sehr schmalen und schlechten – Landstraßen für die Aktivität von Lkw-Speditionen nicht interessant seien und die Frachtkosten bei Auflassung einer Bahn deshalb stark stiegen. Hingegen dringe der Lkw-Verkehr zwischen den großen Städten in den Knotenpunktverkehr der Eisenbahn ein und schädige sie damit an ihrer Hauptsubstanz – wodurch sie gezwungen sei, dort zu sparen, wo der politische Widerstand am geringsten sei: bei den Zweigstrecken. So war es der Bundesverkehrsminister Seebohm, der ab 1954 die Bundesbahn aufforderte, Ergebnisse bei der Stilllegung von Strecken vorzuweisen, um das Defizit der Bahn zu verringern.
Freilich ließen sich die Zweigstrecken auch seinerzeit meist nicht eigenwirtschaftlich betreiben. Im Gesamtnetz warfen aber auch Zuschusslinien unter dem Stich noch gute Erträge ab, die weitgehend verloren gingen, wenn man die Zweigbahnen aufließ. Das galt für alle erwähnten Betriebe nach Mudau, Buchau oder Güglingen. Beispielhaft sei Mosbach – Mudau erwähnt: Trotz Einnahmen von 177.000 Mark bei Ausgaben von 533.000 Mark ergab sich ein Gesamtumsatz für die Bundesbahn 1957 von 787.000 Mark. Da der Aufwand für die zusätzliche Beförderung des Aufkommens, das von und auf das Hauptnetz überging, gering war, verblieb noch immer ein guter Gewinn für die Bundesbahn, hier über 200.000 Mark. Dies wurde jedoch verschwiegen und nur mit den Defizitwerten der eigentlichen Zweigstrecken operiert.

Die Spiegelauer Waldbahn

von Ludwig Reiner, Hermann Beiler, Richard Sliwinski – Ohetaler Verlag

Auszug:
16.1 Einnahmen, Ausgaben, Überschuss (*Geschäftsjahr 1931)*
Indirekte Einnahmen wurden in der Bilanz ausgewiesen.
a) Einsparungen von Mitarbeitern durch den Betrieb der Bahn,
b) Weg- und Triftbaukosten: Einsparung: Wege-Neubau, Wege-Reparatur, Triftbau
c) Gewinne an Holzwert (Holz, das nur durch das Vorhandensein der Bahn gewonnen werden konnte. Der Unterzeichner)

Ergebnis *(vom Autor Carsten Recht erarbeitet)**:***
Die Einnahmen aus dem Betrieb einer Bahn trotz erheblicher Aufwendungen für deren Erhalt höher als die Ausgaben. Es konnte ein Gewinn erwirtschaftet werden. Auch heute würde eine Gegenüberstellung Erstaunliches ergeben.
Unter Berücksichtigung des WASHO-Road Test ist der Unterhalt selbst einer Zweigstrecke kostengünstiger als der laufende Unterhalt einer parallelen Straße.
Der wahre Sachverhalt bei der Bewertung der Zweigstrecken wurde seit 1954 sowohl in der BRD als auch in der DDR erfolgreich verschleiert! Damit sind viele Bahnlinien – selbst Schmalspurbahnen – stillgelegt worden, die man hätten erfolgreich weiter betreiben können.
Carsten Recht (Geschäftsführender Gesellschafter der Böhmetal-Kleinbahn Betriebs-gGmbH)

Der AASHO-Straßentest war eine Reihe von Experimenten, die von der American Association of State Highway- und Transportbeamten (AASHTO) [a] durchgeführt wurden, um festzustellen, wie der Verkehr zur Verschlechterung der Straßenbeläge beitrug. Offiziell bestand der Straßentest darin, "... die Leistung von Fahrbahnstrukturen bekannter Dicke unter sich bewegenden Lasten bekannter Größe und Häufigkeit zu untersuchen". Diese Studie, die Ende der 1950er Jahre in Ottawa, Illinois, durchgeführt wurde, wird häufig als Hauptquelle für experimentelle Daten angeführt, wenn der Verschleiß von Fahrzeugen auf Autobahnen für die Zwecke der Straßenplanung, der Fahrzeugbesteuerung und der Kostenberechnung berücksichtigt wird.

Der Straßentest bestand aus sechs zweispurigen Schleifen entlang der zukünftigen Ausrichtung der Interstate 80. Jede Spur wurde wiederholt durch einen bestimmten Fahrzeugtyp und ein bestimmtes Gewicht belastet. Die Fahrbahnstruktur innerhalb jeder Schleife wurde variiert, so dass das Zusammenspiel von Fahrzeuglasten und Fahrbahnstruktur untersucht werden konnte. In anderen Teilen des Landes wurden "Satellitenstudien" geplant, um Klima- und Untergrundeffekte untersuchen zu können, die jedoch nie durchgeführt wurden.

Die Ergebnisse des AASHO-Straßentests wurden verwendet, um einen Leitfaden für die Gestaltung von Gehwegen zu entwickeln, der erstmals 1961 als "AASHO Interim Guide für die Gestaltung von starren und flexiblen Gehwegen" herausgegeben wurde. Die wichtigsten Aktualisierungen wurden 1972 und 1993 veröffentlicht. Die Version von 1993 ist noch vorhanden in den Vereinigten Staaten weit verbreitet. Ein neuer Leitfaden, der ursprünglich für die Veröffentlichung im Jahr 2002 geplant war, sich aber noch in der Entwicklung befindet, wäre der erste AASHTO-Leitfaden für die Gestaltung von Straßenbelägen, der nicht in erster Linie auf den Ergebnissen des AASHO-Straßentests basiert.

Der AASHO-Straßentest führte viele Konzepte in der Straßenbautechnik ein, einschließlich des Lastäquivalenzfaktors. Es ist nicht überraschend, dass die schwereren Fahrzeuge die Wartungsfreundlichkeit in viel kürzerer Zeit als leichte Fahrzeuge reduzierten, und die oft zitierte Zahl, das so genannte Generalized Fourth Power Law [2], besagt, **dass durch Fahrzeuge verursachte Schäden mit der 4. Potenz ihres Achsgewichts zusammenhängen** , 'leitet sich daraus ab. Das andere direkte Ergebnis der Tests waren neue Qualitätssicherungsstandards für den Straßenbau in den USA, die bis heute angewendet werden.

Der Ende der 1950er Jahre durchgeführte AASHO-Straßentest ist die Grundlage für die meisten Verfahren zur Gestaltung von Straßenbelägen, bei denen die Methode der Äquivalenzfaktoren der American Association of State Highway and Transportation Officials (AASHTO) angewendet wird. Es wird anerkannt, dass die Beziehungen zwischen Verkehrsbelastung und Fahrbahnleistung, die sich aus dem AASHO-Straßentest ergeben, nur für die Bedingungen gelten, unter denen sie entwickelt wurden. …

Fazit:

Die Belastungen von Straßen durch große Busse und Lkw im Vergleich zum Pkw kann hieraus errechnet werden. Bei der Eisenbahn wird in der Regel 20 to Achsdruck für die meisten Strecken zugelassen. Eine Schmalspurbahn hat meist einen Achsdruck von max. 6 – 10 to. Die Böhmetal-Kleinbahn bringt max. 6 to Achsdruck auf den Bahnkörper.

Ein Problem lösen

Die Schüler sollen den Energieverbrauch von Bus, Auto, Straßenbahn und Fahrrad vergleichen.

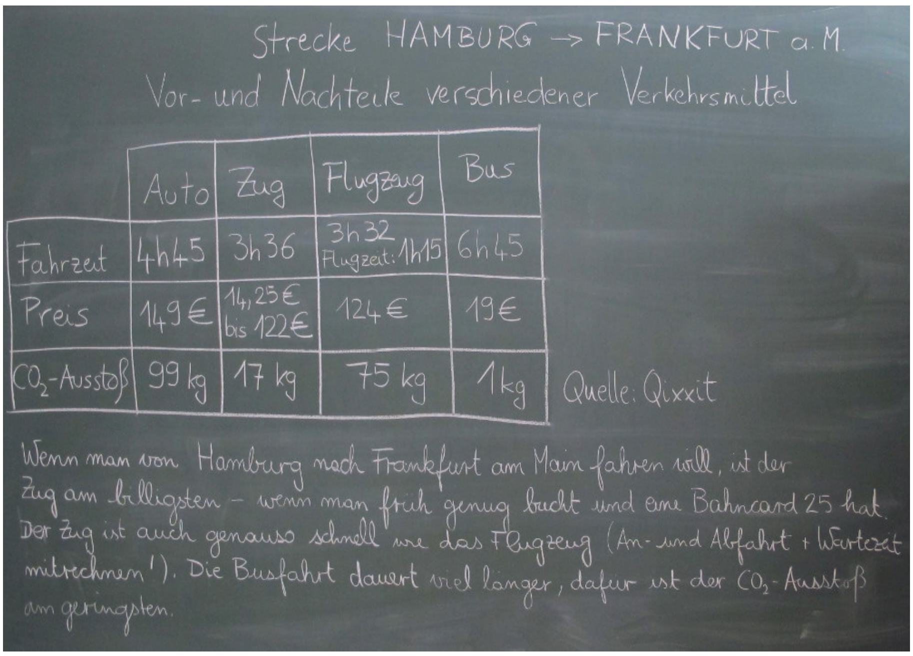
Die Schüler stellen fest, dass das nicht möglich ist, weil der Energieverbrauch in verschiedenen Maßeinheiten angegeben ist (Liter Benzin oder Diesel pro 100 km; kWh/km; kcal/km). Zunächst müssen also alle Energieangaben in die gleiche Maßeinheit umgerechnet werden. Erst dann lässt sich der Energieverbrauch für den Transport von 300 Personen über eine Strecke von einem Kilometer – mit Bussen, Autos, Straßenbahnen und Fahrrädern – vergleichen.Abb. 1: Vergleich des Energieverbrauchs von Bus und Auto [1]

Anmerkung: Oft sind die Umrechnungsübungen für die Schüler nicht einfach. Daher sollte man für ein Verkehrsmittel (zum Beispiel für das Fahrrad oder den Bus) die Rechnung gemeinsam durchführen. Erst wenn sichergestellt ist, dass diese Rechenübung gut verstanden wurde, können die Schüler allein weitermachen.

Gemeinsame Erörterung

Die Schüler kommen zu folgenden Ergebnissen.

	Gelenk-Bus	**Auto**		**Straßenbahn**	**Fahrrad**
Energieverbrauch	55 Liter Diesel pro 100 km	8 Liter Benzin pro 100 km		5 kWh pro km	20 kcal pro km
Energieverbrauch (in kWh)	297 kWh pro 100 km	28 kWh pro 100 km		5 kWh pro km	0,023 kWh pro km
Energieverbrauch pro Kilometer (in kWh)	3,0	0,28		5,0	0,023
Anzahl der transportierten Personen	100	1	5	180	1
Anzahl der benötigten Fahrzeuge, um 300 Personen zu transportieren	3	300	60	1,7	300
Energieverbrauch, um 300 Personen über 1 km zu transportieren(in kWh)	9,0	84	16,8	8,5	6,9



TRANSPORTATION RESEARCH

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Pavement Lessons Learned from the AASHO Road Test and Performance of the Interstate Highway System

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TRANSPORTATION RESEARCH CIRCULAR E-C118

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Foreword

The design and construction of the Interstate Highway System (IHS) is one of the most significant and costly public works projects in the history of the United States. Given that the 50th anniversary of the enabling legislation for the IHS occurred during 2006, it is appropriate to reflect on the performance of the system, specifically its pavements. The American Association of State Highway Officials (AASHO)[1] Road Test was authorized by the IHS legislation and occurred about 5 years after the legislation was enabled. It is also assessed in light of what we know today.

Reflections on the performance of IHS pavements and lessons learned from the AASHO Road Test were presented in two sessions organized by the Transportation Research Board (TRB) Pavement Management section (AFD00) for the 2006 TRB Annual Meeting. The two sessions were Session 353: Pavement Lessons from the 50-Year-Old Interstate System and Session 470: Views on AASHO Road Test after 50 Years. Each of the presenters was requested to prepare a manuscript for inclusion in this circular. They are included as six papers, not necessarily in the order they were presented in the sessions. The name of each paper and a brief summary of their content are

- **Pavement Design in the Post-AASHO Road Test Era** This paper outlines the application of the Road Test results and its impact on pavement design in the United States. The efforts under way to develop mechanistic–empirical (M-E) design procedures to replace the AASHO Road Test based procedure are also explained. In addition, the difficulties encountered in the development and implementation of M-E procedures and recommended future actions to insure the success of M-E design are provided.
- **AASHO Road Test Effect on Pavement Design and Evaluation after 50 Years** The AASHO Road Test construction and testing occurred from 1956 to 1961. Significant results from the Road Test still influence pavement design worldwide, including
  - Equivalent single-axle loads (ESALs),
  - The serviceability–performance concept,
  - Effects of layer thickness and strength, and
  - Effectiveness of dowels and joint spacing.

  The Road Test results are the basis for pavement design still widely applicable and currently used. It also changed the way that people conduct pavement research by illustrating the power of factorial experiments, high-quality data, and statistical analyses.
- **What Pavement Research Was Done Following the AASHO Road Test and What Else Could Have Been Done But Was Not** The AASHO Road Test provided significant results that led to improved pavement design following the Road Test and to an expanded research effort by the pavement engineering community worldwide. In particular, it resulted in the development of what is termed today M-E pavement design. Results of the Road Test also contributed to the development of nondestructive pavement evaluation, including overlay pavement design and to the development of pavement management concepts.
- **A Historical Look at Interstate Highway System Pavements in the North Central Region** This paper focuses on the pavements constructed on the Interstate system in the North Central region of the United States (North Central includes 13 states: Illinois, Indiana, Iowa, Kansas, Kentucky, Michigan, Minnesota, Missouri, Nebraska,

North Dakota, Ohio, South Dakota, and Wisconsin). Included is a brief summary of the AASHO Road Test, which was located in the North Central region and served as the basis for the design of much of the Interstate system.

- **Pavement Lessons from the 50-Year-Old Interstate Highway System: California, Oregon, and Washington** Pavements constructed for the Interstate system in the West Coast states of California, Oregon, and Washington were examined to see what lessons could be learned with respect to design and performance during the past 50 years. The focus was on the performance of new or reconstructed pavement structures.
- **Interstate Highway System Challenges: North Atlantic States** This paper summarizes some of the reconstruction and rehabilitation practices for Interstate pavements in the North Atlantic States including New York, New Hampshire, Vermont, and North Carolina.

We know that you will find the information provided by the authors of interest. It was a pleasure to work with these individuals and we sincerely appreciate their collective efforts to conduct the sessions and prepare the papers for this circular. We also want to thank Stephen Maher, TRB Engineer of Design, for his initiative and support in bringing this circular to completion.

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[1] Renamed the American Association of State Highway and Transportation Officials (AASHTO) in 1973.

Contents

Pavement Design in the Post-AASHO Road Test Era

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The AASHO Road Test was probably the most significant pavement research performed in the 20th century. The results of the AASHO Road Test served as the basis for nearly all the pavement designs used in the original construction of the Interstate Highway System (IHS) after 1961. When we look back on the overall performance of the pavements on the Interstate system, we find that most of the pavements have lasted the expected 20 years while carrying traffic volumes far in excess of those predicted at the time of design.

This paper outlines the application of the road test results and its impact on pavement design in the United States. Efforts under way to develop mechanistic–empirical (M-E) design procedures to replace AASHO Road Test Procedure are also provided, including the difficulties encountered in the development and implementation of M-E procedures and recommended future actions to insure the success of M-E design.

THE BEGINNING

The AASHO Road Test was the last of a series of road tests conducted by state highway agencies and the Bureau of Public Road starting in the 1920s (*1*). The primary purpose of the road tests was to determine the relationship between axle loading and pavement structure on pavement performance. This knowledge was needed to assist in the design of pavements, to provide an engineering basis for establishing maximum axle load limits, and to provide a basis for the allocation of highway user taxation

The AASHO Operating Committee on Design was assigned the responsibility of developing pavement design procedures based on the findings of the AASHO Road Test. During the early stages of the road test, a subcommittee on pavement design practices was formed to accomplish this task. This subcommittee, working with the AASHO Road Test research team, developed the *AASHO Interim Guide for the Design of Flexible Pavement Structures* (*2*) and the *AASHO Interim Guide for the Design of Rigid Pavement Structures* (*3*). The Interim Guides for flexible and rigid pavements were completed in October 1961 and April 1962, respectively, and were issued as separate documents. Originally the Interim Guides were to be tested during a 1-year trial period in parallel with existing state procedures. Therefore the guides were not formally published. Rather the original typed manuscripts were copied using the relatively crude techniques available in 1962. Many of the original copies of the guides were mimeographed copies and have faded to the point that they can no longer be read.

At the end of the 1-year evaluation period it was determined that revisions were not required at that time (*4*). However, the Interim Guide was not formally published until its first revision in 1972. In retrospect, the implementation of the road test findings was an outstanding achievement for two primary reasons: (1) the Interim Design Guides were issued less than 2 years after the last traffic loadings were applied to the test section on November 30, 1960, and (2) the fundamental relationships included in the original Interim Guides have endured to the present time. The performance relationships contained in the Interim Guide along with the design nomographs are described in a paper presented at the May 1962 Conference on the AASHO Road Test in St. Louis, Missouri (*5*).

The design procedures presented in the guides are based on the general AASHO Road Test equations that relate the loss in pavement serviceability to the pavement structural section and load applications. The pavement serviceability concept developed at the road test is a measure of the ability at the time of observation of a pavement to serve the traffic that uses the facility. At the road test, the concept of serviceability evolved from the concept that the prime function of a pavement was to serve the traveling public (*6*). Since serviceability is a subjective rating, as part of the Road Test, a panel of raters consisting of both truck and automobile drivers were used to rate 138 sections of pavement in three different states (*7*). They rated each section on the following scale: (0–1) very poor, (1–2) poor, (2–3) fair, (3–4) good, and (4–5) very good. These numerical ratings were referred to as the present serviceability rating (PSR). At the same time the pavements were being rated by the panels, road test crews were making physical measurements of the pavement condition. These measurements included longitudinal roughness (profile) of all pavements and cracking, patching, and rutting of flexible pavements, and cracking, spalling, and patching of rigid pavements. With regression analysis, a relationship was developed to predict a present serviceability index (PSI) based on the physical measurements of the pavement. At the road test physical measurements were made on the test sections, and the present service index was predicted for each section at 2-week intervals (*6*).

Pavement performance is the overall appraisal of the serviceability history of a pavement. Thus the performance of a pavement may be described by observations of its serviceability at the completion of construction and at the end of selected time periods subsequent to completion.

The overall concept of the AASHO pavement design procedure is to provide a pavement structure that is adequate in thickness, composition, and quality to ensure the pavement section does not reach a terminal serviceability level during its design life. Terminal serviceability defines a pavement that is considered unacceptable by the highway user. The value for terminal serviceability used in the design procedure was also based on user input. At the road test, in addition to providing a rating of 0 to 5, each panel member was to indicate whether the pavement was acceptable. In the design procedure a terminal serviceability index value of 2.5 was selected for Interstates and 2.0 for all other roads. The selection of these terminal index values was validated by a study conducted in the late fall and early winter of 1961 to 1962. Three teams inspected 134 pavements in 35 states that were scheduled for resurfacing in the summer of 1962. For each pavement section, the PSI was predicted. Since all of the pavements were scheduled for rehabilitation this was considered their terminal serviceability. The average terminal serviceability was 2.2 for rigid pavements on the primary highway system and 2.1 for flexible pavements on the primary system, and 1.9 for pavements on the secondary system (*8*).

Traffic is input into the design procedure as the average daily 18-kip equivalent single-axle loads (ESALs) that will occur over a 20-year design period. This is accomplished by converting each axle load in mixed traffic to an equivalent number of ESALs. The factors developed to convert an axle to its ESAL are based on relationships of pavement performance to application of axle loads of fixed magnitude which were developed at the road test.

At the time of the introduction of the AASHO design procedure, calculators and personal computers (PCs) had not been introduced. Pavement thickness design calculations were generally performed using design charts. The AASHO procedure was implemented through the use of nomographs. Figure 1 illustrates the nomograph used for the thickness design of flexible pavements to be constructed on the Interstate system.

Since the road test was conducted on one subgrade type and in a single climate, provisions were included for accommodating different subgrades and climates. The total structural section of a flexible pavement section may consist of a subbase, base, and the asphalt surface. This was represented by the structural number (SN) in the following relationship:

$$\mathrm{SN} = a_1 D_1 + a_2 D_2 + a_3 D_3$$

where

a_1, a_2, and a_3 = coefficients determined at the road test and
D_1, D_2, and D_3 = thickness of the bituminous surface coarse, base layer, and subbase layer, respectively, in inches.

The values of the coefficients for the Road Test materials were a_1 (plant mix, high stability = 0.44), a_2 (crushed stone = 0.14), and a_3 (sandy gravel = 0.11).

The point where a line drawn from the SN and traffic repetitions intersected the soil support line was given a value of 3. Loop 4 at the Road Test had a heavy crushed-stone base and it was determined that the subgrade soils had negligible impact on the performance of the loop. The point where a line for the Loop 4 performance intersected the soil support line was assigned a value of 10. A linear scale was assumed between 10 and 3 and extended to 0. Correlation charts were then developed between soil support value and *R*-value and California bearing ratio (CBR) tests and the group index. The regional factor was included the design procedure to permit an adjustment in design because of changes in the climatic and environmental conditions. The 1961 Interim Guide states that “at present, there is no way to determine the regional factor directly. It must be estimated by analyzing the duration of certain typical conditions during an annual period.”

The Interim Guide presented a method for developing an average regional factor at a project site on the basis of the time period during the year that the roadbed soil was frozen, dry, or saturated. The Interim Guide recommended that in general the regional factor be between 3.0 and 0.5 for conditions in the United States

The development of all of the design factors are more fully described in the reports on the Road Test (*5*, *6*).

Figure 2 presents the nomograph developed for the design of rigid pavements to be constructed on the Interstate system.

At the AASHO Road Test, the modulus of elasticity of the concrete, E_c; the modulus of rupture of the concrete, S_c; the modulus of subgrade reaction K; environmental conditions; life both

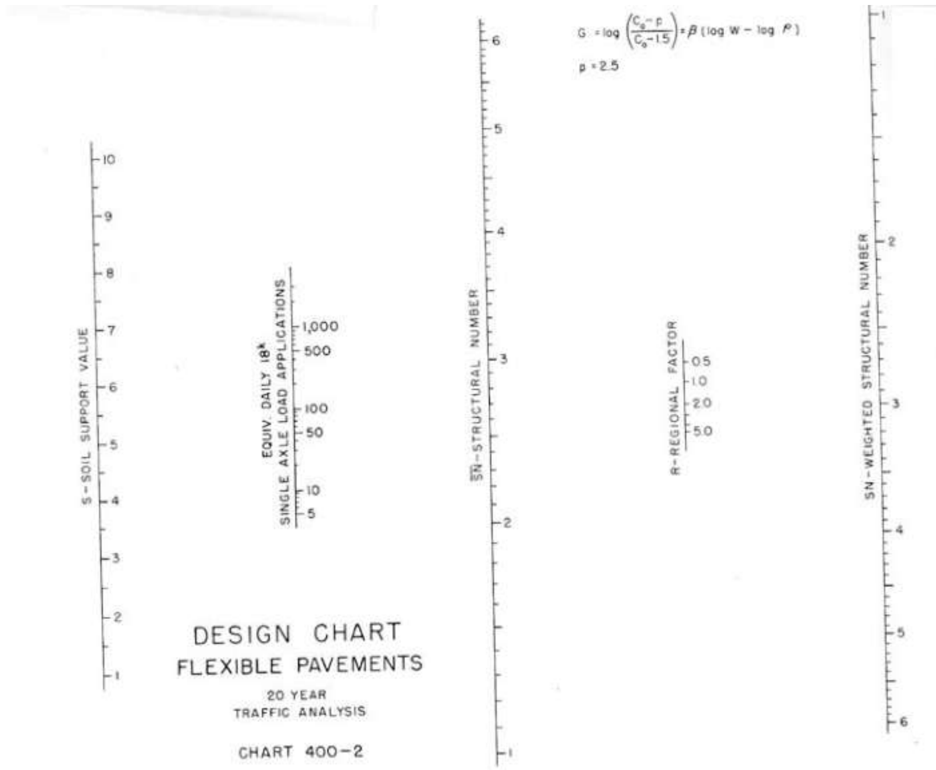

FIGURE 1 Design chart for flexible pavements on the Interstate system.

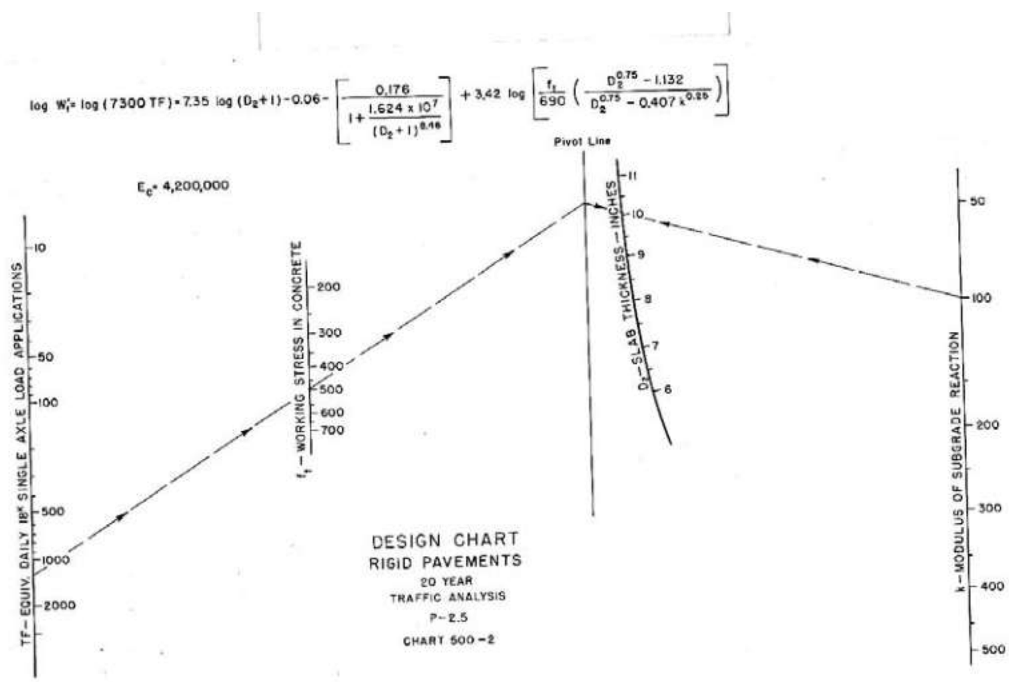

FIGURE 2 Design chart for design of rigid pavements on the Interstate system.

equations did an excellent job of linearizing the Road Test measurements. The Spangler equation for corner stresses was selected for use because of its simplicity (*3*, *5*, *9*). The Spangler equation provides a relationship between maximum tensile stress and load transfer at the joint, wheel load, slab thickness, Young's modulus of elasticity for concrete, modulus of subgrade reaction, and Poisson's ratio for concrete. The resulting procedure for the design of rigid pavements uses the AASHO Road Test equation with necessary modifications based on Spangler's theory added to it.

The Interim Guide was subsequently revised in 1972 (*4*). The basic design methods and procedures contained in the 1962 version of the guide were not changed in the 1972 revisions. Rather, explanatory material was added to facilitate implementation of the guide and overlay design procedures commonly in use were presented. In addition, both the rigid- and flexible-design procedures were incorporated into one published document.

In 1981, a revised Chapter III: Guide for the Design of Rigid Pavement was incorporated into the *AASHTO Interim Guide for Design of Pavement Structures 1972* (*10*). In the original Guide, the working stress for concrete was 0.75 times the expected 28-day modulus of rupture. In the 1981 revision, a safety factor of safety "C" was added to the procedure, where the working stress was equal to the expected 28-day modulus of rupture divided by C. For most conditions 1.33 was the recommended value for C. However, a C factor of up to 2.0 was recommended for freeways and other high-volume facilities where closing of a lane for possible rehabilitation would cause capacity problems. The use of a safety factor of 2.0 was expected to add 1 or 2 in. of pavement thickness.

In 1986 a significant revision was made to the guide, however, the procedure was still based on the performance equations developed at the AASHO Road Test (*11*). The revision to the guide included consideration of the following 14 items:

1. Reliability.
2. AASHTO Test T-274, resilient modulus for roadbed soils, was recommended as the definitive test for characterizing soil support.
3. The resilient modulus test was recommended as a procedure for determining layer coefficients in flexible pavement design.
4. Provisions were included for considering subsurface drainage.
5. Environmental factors such as frost heave, swelling soils, and thaw weakening were considered.
6. Provided procedures for the design of rigid pavements with tied shoulders or widened outside lanes.
7. Provided a method for considering the effects of subbase erosion under rigid pavements.
8. Information was provided on life-cycle cost analysis (LCCA).
9. A major section on rehabilitation was added.
10. Background information was provided on pavement management.
11. Load equivalency values were extended to heavier loads and terminal serviceability levels up to 3.0.
12. Extensive information for calculating ESALs was provided.
13. A design catalogue for design of low-volume roads was included.
14. A chapter discussing the state of knowledge of M-E design was included.

The adoption of the *1986 Guide for Design of Pavement Structures* was the first version of the guide not to be labeled as interim. While the 1986 guide still contained nomographs, issuance of the 1986 guide coincided with the widespread introduction of PCs into the workplace. In June of

1987 AASHTO announced the availability of DNPS86/PC$_{TM,}$ A Personal Computer Program for New Pavement Design. DNPS86/PC was a software program developed for AASHTO that incorporated many of the pavement design concepts and procedures presented in the 1986 guide. In 1991, AASHTO issued improved and updated software for pavement design called DARWin. Since its initial release in 1991, DARWin has undergone three major upgrades. The latest version, DARWin 3.1, is designed for the Windows 95, 98, 2000, and NT operating systems. Some features included in the current version of DARWin are on-the-fly unit conversion, enhanced project file management, combined material and pay-item libraries, enhanced pavement deflection data processing and analysis, enhanced graphical outputs, including pavement cross section, project cash flow diagrams, and pavement deflection profiles. DARWin 3.1 is divided into four modules, each of which addresses a specific item in the overall pavement design process. Collectively, these modules can be used to design and compare alternative pavement designs: flexible structural design, rigid structural design, overlay design, and life-cycle cost. The introduction of pavement design software has made the use of design nomographs obsolete.

In 1993 a revision to the 1986 guide was issued, containing modifications to the overlay design procedure (*12*).

In 1998 a *Supplement to the AASHTO Guide for Design of Pavement Structures, Part II, Rigid Pavement Design and Rigid Joint Design* was issued (*13*). This supplement was developed with data from the long-term pavement performance (LTPP) program to incorporate loss of support, improved selection of the modulus of subgrade reaction (*k*), and the effects of joint spacing on mid-panel cracking.

SIMPLICITY: THE KEY TO FULL-SCALE IMPLEMENTATION IN A SHORT PERIOD

While the AASHO guide was a very simple procedure to apply, it was a giant step forward in pavement design. The guide filled an important need at a critical time in United States history: the design of economical and structurally adequate pavements for the IHS. The AASHO Design Committee issued the Guide 1962 for a 1-year trial period to be used in parallel with their existing procedures. After the trial period, no reason was found to revise the guides and they were retained without modification (*4*). Further, the pavement procedure used in the United States through 2006 was based on the AASHO Road Test equations.

During the time period following the AASHO Road Test, the most pressing issue facing highway agency managers was the completion of the IHS. Staffing and resources for activities not in direct support completing the Interstate system were greatly reduced in many agencies. This, coupled with the early success of the AASHO guide, led to a reduction in highway agency research in the area of pavement design. A major loss caused by the movement of the states away from pavement research was not completing the satellite road studies. One of the key recommendations made at the completion of the AASHO Road Test was to perform a series of satellite road studies. The purpose of these studies was to provide guidance on determining appropriate corrections to designs based on climate and foundation differences between the various states and the AASHO Road Test site. These studies were described as relatively small road tests in different parts of the country. These studies were to consider variables not included in the AASHO Road Test (base type, subgrade types) (*6*).

The simplicity of the AASHO pavement design procedure may also have given agency managers the false impression that pavement design was not as technically challenging as other civil engineering specialty areas. As a result, during the 1960s and 1970s many agencies reduced the resources devoted to the development or employment of pavement engineering professionals. Many civil engineers with pavement-related advanced degrees (MS and PhD) tended to be employed either by a limited number of private consultants specializing in pavement engineering or university research centers. Also, during that time period, most pavement design research was performed by private consultants and universities.

This does not mean that pavement design research was not being undertaken in the United States. As discussed by Monismith et.al. (*14*), there was a considerable amount of research underway in the area of M-E analysis procedures at the academic level. This work is well documented in the *Proceedings of the International Conferences—Design of Asphalt Pavements* and the *Proceedings of the International Conferences on Concrete Pavement Design*, held on a 5-year cycle since 1967 and 1977, respectively. Research on asphalt pavement design is also published in the annual proceeding of the Association of Asphalt Paving Technologists.

DISPARITY BETWEEN DAY-TO-DAY DESIGNS AND THE NEED FOR PERFORMANCE PREDICTION MODELS

While many advances were being made in gaining an understanding of pavement performance, particularly at the university research level, little new design technology was being implemented at the operational level. During the 1960s and 1970s a communications gulf developed between the pavement designers in the states and the pavement researchers. One example of this gulf is the VESYS program, which was developed in the late 1970s under sponsorship of FHWA and continually updated through the 1980s (*15*). This pavement evaluation and design program for flexible pavements was never implemented at the practicing design level.

More important, little to no materials characterization testing was required to support pavement design using the AASHTO Design Guide procedure, while fundamental tests are required to use the mechanistic-based procedures. This disparity in materials testing between purely empirical and mechanistic-based methods only increased the gap in use. Thus, use of M-E methods stayed on the research sidelines and was not generally implemented.

An additional barrier to design innovation may have been the decision of the Bureau of Public Roads (BPR) to issue a policy requiring all pavement designs on the IHS be designed in accordance with the AASHO Interim Guide. This requirement arose from the Federal-Aid Highway Act of 1956, which called for uniform geometric and construction standards for the Interstate system. The BPR and, subsequently FHWA, zealously guarded this requirement to ensure completion of the Interstate system in a timely and uniform manner.

THE FUTURE: REPLACING SIMPLE EMPIRICAL DESIGN PROCEDURES WITH MORE COMPLEX METHODS

We have again reached an important milestone in the development of pavement design technology in the United States: the replacement of the largely empirical AASHO Road Test pavement design procedures with M-E analysis and design techniques. The development of M-E procedures has followed a difficult path and still faces many hurdles before it is fully adopted by AASHTO.

Following the development of the AASHTO Guide for Design of Pavement Structures in 1986, it was apparent that improved pavement design procedures based on M-E design procedures were needed. A number of problems were identified with the existing AASHTO procedure, including the following:

- The data set used to develop the AASHO Road Test performance equations was based on traffic loadings of approximately 10 million 18-kip ESALs. Today many pavements are being designed to carry loadings in excess of 100 million 18-kip ESALs during their design lives. Designs based on projections this far outside the data set must be considered highly unreliable. In many cases, thicknesses of pavements designed using the AASHTO procedures are much thicker than those predicted using rudimentary M-E procedures.
- There are minimal materials inputs into the existing design procedure, particularly as relates to flexible pavement design. This does not allow improvements and changes in materials properties to by fully considered in the design procedure.

The 1986 guide included a Part IV providing a brief overview outlining a framework for the development and implementation of a M-E design procedure. As a result, in 1987 NCHRP Project 01-26: Calibrated Mechanistic Structural Analysis Procedures for Pavements was awarded. This project was completed in 1992; however, it was determined not to go forward with implementation efforts at that time and the report was never published (*16*).

By 1996 it was becoming imperative that a new design procedure be developed that would include procedures for addressing pavement rehabilitation. NCHRP Project 01-37: Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures was awarded in December of 1996. This project was the initial step in the process of developing a new pavement design process. The project developed a draft plan for developing the design process and served as the basis for NCHRP Project 01-37A: Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures: Phase II, which was awarded February 1, 1998, and completed February 28, 2004. The objective of this project was to develop and deliver a guide for design of new and rehabilitated pavement structures based on M-E principles using existing models and databases (*17*).

Project 1-37A was one of the largest projects ever funded by the NCHRP. The design procedure developed under the project was a giant leap forward from existing practice. The complexity of the process presented a major challenge to both the researchers and the NCHRP project panel overseeing the work. One of the major challenges was bridging the gulf between the agency practicing engineer and the research community on what constituted a state-of-the-art M-E design procedure. Factors that contributed to this gulf included

- The simplicity of the existing procedure and the relative inexperience of the highway agency community in the application of M-E design procedures; and
- The apparent simplicity of the limited number existing M-E design procedures.

This was compounded by a lack of uniform interpretation of the requirement to use existing models and databases and by an underestimation of the difficulties that would be encountered when trying to apply them. Most of the existing models had only limited usage in specific climatic regions with average lifetime materials properties used as inputs. When varying materials properties and climatic conditions were applied, some of the models gave erroneous results. Regardless of the problems encountered, Project 1-37A was completed in 2004 and has entered the implementation process. The Project 1-37A M-E design process presents a significant improvement over earlier M-E processes such as those developed by the Portland Cement Association, Asphalt Institute, University of Minnesota, and Illinois Department of Transportation. The 1-37A procedure is the first to use incremental damage accumulation. The incremental damage process provides the capacity to accumulate damage on a monthly or semi-monthly basis over the design period. The integrated climatic model is used to estimate climatic conditions for each increment of the time that is used in accumulating damage. These estimates of climatic conditions are used to estimate changes in materials properties for each increment. In addition, the 1-37A procedure includes models for estimating the changes in materials properties due to aging. Traffic loadings are estimated for each increment, and where appropriate, differences in daytime and nighttime traffic can be considered

The 1-37A process is iterative in that it evaluates trial design sections and materials to determine if they meet the threshold distresses, set by the agency, over the design period.

The following factors and actions impact the acceptance and schedule for adoption of the M-E guide developed in Project 1-37A (*18*):

- The analysis process presented in the M-E guide represents a radical change in the way pavements are analyzed and designed.
- Implementation will require a significant commitment of resources to be successful.
- Implementation will require a change in the way highway agencies approach pavement engineering.
- The M-E guide is not a cookbook that can be applied by inexperienced engineers.
- The user will need an understanding of pavement performance.
- Agencies will need to develop engineers with a specialization in pavement engineering.
- Implementation will require close coordination between materials, construction traffic data collection, and design.

ACTION PLAN FOR THE FUTURE

The implementation and continued development of a M-E pavement design procedures is going to require a significant commitment of resources by highway program managers. When considering benefits of adopting improved pavement engineering practices it is important to consider the amount of money being spent on highway pavements. In 2002, highway agencies had capital outlays totaling $46.3 billion (*19*). The distribution of capital outlays is shown in Figure 3. As indicated in the figure, 55%, or approximately $25 billion, annually go to pavement-related work on 3R, major widening, reconstruction, and new construction projects. Prudent management requires that sound pavement design processes be used in the development of these projects to insure the most efficient use of the limited resources available.

It should be pointed out that the cost of capital improvements does not include the cost to the highway user for delays incurred during the reconstruction and maintenance of highway pavements. In 1997 FHWA estimated congestion delay cost ranging between $181.6 and $16.3 billion with a midrange estimate of highway congestion of $61.7 billion (*20*). FHWA estimates that 10% of highway congestion is caused by highway construction (*21*).

BENEFITS OF THE M-E DESIGN METHOD: IS IT WORTH THE INVESTMENT?

A hypothetical presentation of the observed performance of current pavement designs could be expected to follow the solid line shown in Figure 4. One of the prime reasons for the premature failures is the inability to incorporate variations in materials and construction into the design procedure. In the current flexible design procedure, the only material property incorporated is the loosely defined coefficient *a*. This problem results from the fact that variation in material quality was not a primary experimental variable included in the AASHO Road Test. Another observed problem with current designs is the large variation in performance life in relation to design life. This variation is to be expected because of the large extrapolation of the road test data.

The M-E design procedures will provide the tools for the designer to evaluate the effect of variations in materials on pavement performance. M-E procedures will provide a rational relationship between construction and materials specification and the design of the pavement structure. It will also provide the tools required to evaluate the effects of changes that occur during construction. Since the mechanistic procedure will be able to account better for climate, aging, today's materials, and today's vehicle loadings, variation in performance in relation to design life should be reduced and thus allow the agency manager to make better decisions based on life-cycle cost and cash flow.

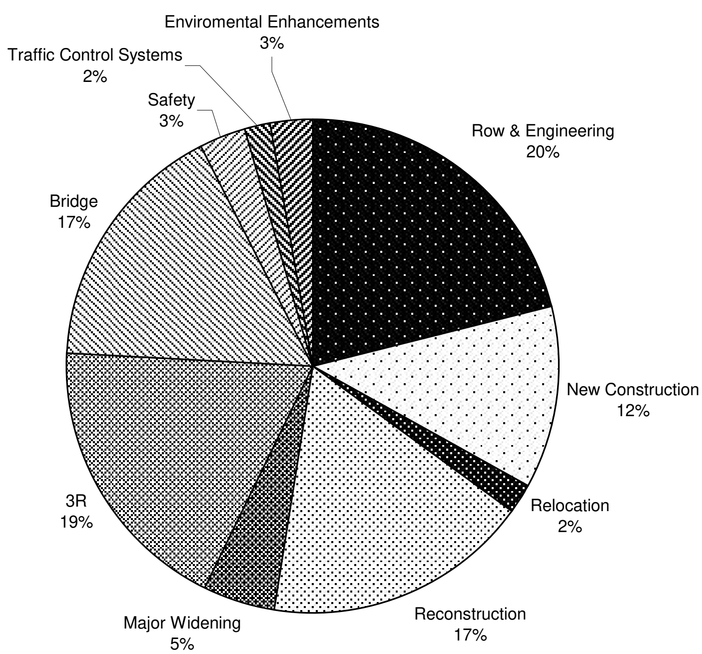

FIGURE 3 Distribution of capital outlays by state highway agencies, 2002.

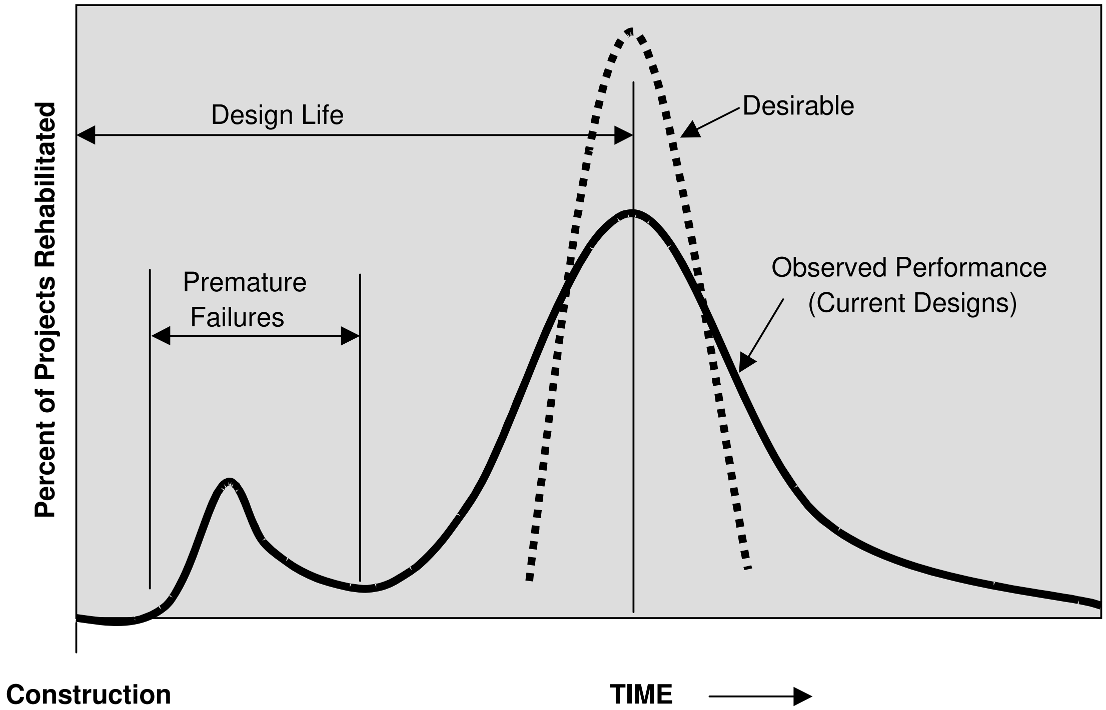

FIGURE 4 Performance of existing pavement designs

ORGANIZATIONAL DEVELOPMENT: STRUCTURE TO MEET FUTURE GOALS

To utilize advanced M-E design procedures effectively, highway agencies will need to incorporate pavement engineering divisions within the department that are staffed by career pavement engineering specialists. These specialists will need to be developed through a combination of on-the-job experience and post-graduate education. This will be a long-term commitment by the agency, since development of experienced and competent staff will take time. Bridge engineering and geotechnical staff development provide good models to follow in the development of pavement engineering staff.

To facilitate the continued development and implementation of improved M-E pavement design and analysis tools, it is recommended that the states work with AASHTO on the creation of a subcommittee for pavements. Pavements represent the largest single capital expenditure by state highway agencies. It is important that all of the states participate in the development and implementation of pavement engineering tools. Although the Lead State Program created by FHWA is a step toward implementation and adoption, it is only the initial step in the process. Further, it is important that each state pavement engineer have the opportunity to meet with peers at annual meetings of the subcommittee. The current AASHTO organization using the joint task force approach does not allow participation of all states in the process.

PERFORMANCE DATA NEEDS: MONITORING AND UNDERSTANDING PAVEMENT PERFORMANCE

Historical pavement performance data are a critical need in the development and implementation of M-E pavement analysis and design procedures. In the M-E process, modeling is used to predict pavement distresses on the basis of climatic, materials, vehicle loading, and pavement layer thickness. Therefore, it is essential that the types of distress occurring, where they originate, and how the progress is identified.

Performance data to confirm the accuracy of the design methods can come from three data sources: roadway test sections using actual truck traffic (LTPP, MnRoads, etc.), advanced passenger train (APT) facilities using full-scale test tracks (WesTrack, NCAT), and APT facilities using simulated truck loadings (FHWA, Florida, and Louisana APTs).

Over the years, much progress has been made in the identification and cataloging of visible pavement distress. The LTPP is an excellent example of detailed monitoring of the manifestation of pavement distress over time. However, as detailed as the LTPP distress identification and monitoring program was, significant problems arose when using the data to develop the M-E procedures under NCHRP Project 1-37A. The greatest problems were encountered during the development of the flexible pavement analysis and design procedures. While the LTPP database provided good data on the extent and severity of pavement cracking, there was no indication as to whether observed cracking originated at the top or bottom of the pavement layer. At the beginning of the project, the generally accepted assumption in classical M-E analysis was that most load-related cracking originated at the bottom of the pavement. However, detailed studies where trenching and coring of cracks was performed were beginning to show that a significant amount of wheelpath cracking was surface initiated (*22*). Since the LTPP program did not call for or allow any destructive testing within the section, it was impossible from the data to determine the origination of the cracks.

Similarly, it was the desire of the research team to predict permanent deformation (rutting) in each of the pavement layers. However, only the surface measurement of pavement deformation was available. As a result, many months of trial and error work was required to develop a procedure that would predict deformation each layer whose sum equaled the total measured on the surface.

These examples are provided not as a criticism of the LTPP program, but rather to highlight the importance of understanding failure mechanisms before embarking on a pavement distress measurement program.

An important aspect of the implementation of an M-E procedure is the calibration to local conditions. State highway agencies have been undertaking the development and implementation of pavement management systems (PMS) for more than 30 years. These systems generally include the gathering of pavement distress data on a routine basis. It was hoped that the PMS distress data could be used in the local calibration efforts.

FHWA undertook a project in 2001 (*23*) to examine how existing pavement management data and materials construction data in various state highway agencies could be used to evaluate the performance of new materials and concepts and to validate new design methods. One of the most significant findings of the study was “at the present time, most of the states visited don’t have an appropriate electronic format for other required materials and construction data needed for the evaluation analysis.”

It can be expected that significant resources will be required to develop the pavement monitoring systems required to calibrate and implement M-E procedures.

FUTURE RESEARCH REQUIREMENTS

The implementation of M-E analysis and design procedures will require a continuing long-term commitment to research. It would be reasonable to expect a level of effort and time frame equivalent to that required to implement the Superpave® asphalt mixture design system will be required to implement M-E design. This research can be placed in three categories: improved computational procedures, improved materials testing and characterization, and development of improved performance models.

Improved Computational Procedures

The rapid evolution of the computation capability of PCs is one of the primary factors that make M-E procedures possible today. To illustrate the advances in PC technology, at the first meeting of the 1-37A research team and the project panel in February 1998, a principle item of discussion was the need for a manual solution of the M-E procedure for those agencies that did not have access to a computer.

We are now reaching another turning point in the application of complex and specialized software. With the advent of the connectivity provided by high-speed broadband Internet connections, there may not be a need to develop software that will be operated on individual PCs. Rather, the software could be installed on a central computer that is specifically configured to provide optimum computational speed. Registered users supply the input data and receive the output via the Internet. A further advantage of having the software on a central computer is the ease of installing updates and corrections to the code.

An important computational process that must ultimately be addressed is the development of a finite element (FE) procedure that can be easily applied to flexible design and back-calculation analysis. While there is an FE process included in the 1-37A procedure, the power of the PC is not adequate to make the procedure applicable to routine use. The use of FE procedures is necessary to evaluate fully stress sensitivity in unbound layers, to evaluate the effects of damage on material response, and to couple the different distress mechanisms such as fracture and distortion.

In a November 1997 General Accounting Office (GAO) report (*24*) to the U.S. Secretary of Transportation included the following recommendation:

> To better assist states in designing safer, longer lasting, and more cost-effective new, reconstructed, and overlay highway pavement structures, we recommend that the Secretary of Transportation direct the Administrator, FHWA, to ensure that nonlinear 3D-FEM is considered in the current update of the pavement design guide.

FHWA made a commitment to pursue 3D FE analysis in the 1-37A project. The research team and project panel both agreed that in incorporation of 3D FE analysis was not feasible under the scope of the project. However, as computation power of PCs increase, honoring FHWA's commitment to the GAO is a worthy long-term objective.

Improved Materials Testing and Characterization

M-E structural response model require materials inputs that use test procedures that were not routinely performed by transportation agencies in the past. In addition the material characterizations must reflect changes in material properties due to both aging and climatic events and seasons. These material properties include

- Poisson's ratio for each layer,
- Elastic modulus for each layer,
- Creep compliance of hot-mix asphalt (HMA) mixtures,
- Tensile strength of HMA mixtures,
- Coefficient of thermal contraction and expansion,
- Temperature gradient in the pavement section, and
- Moisture gradient of in-place portland cement concrete (PCC) slabs.

SHRP and subsequent implementation projects at FHWA and NCHRP have made major improvements in the area of materials testing. Areas identified where additional improvements are needed in measurement of HMA properties include

- Dynamic modulus for asphalt concrete (AC) mixtures,
- Resilient modulus for granular materials,
- Permanent deformation properties of HMA and granular materials, and
- Fracture properties of HMA mixtures.

One of the recommendations contained in the final report for Project NCHRP 1-37A (*25*) was the need to improve methods for estimating key PCC materials and construction factors, for inclusion in the analysis process. These include

- Measures of strength and its gain over time,
- Elastic modulus and its gain over time,
- Cement content and type,
- Thermal coefficient of expansion,
- Relative drying shrinkage over time,
- Zero-stress temperature after placement, and
- Permanent curl/warp equivalent temperature difference.

Development of Improved Performance Models

The final report for Project NCHRP 1-37A (*25*) listed the following areas where improvements in models were needed:

- Climatic modeling (particularly variations in subgrade moisture),

- Design reliability,
- Smoothness models,
- Top-down cracking prediction model,
- Permanent deformation (rutting) model for HMA pavements,
- Reflective cracking model for HMA overlays, and
- Crack deterioration model for continuously reinforced concrete pavement (CRCP).

CONCLUSION

The AASHO Road Test provided the needed technology for the original construction of the Interstate system. Today there is a far greater understanding of the performance of pavement systems. This coupled with increased highway loadings, improvements in highway materials, and a shift in focus from new construction to rehabilitation shows the need for adoption of M-E pavement analysis and design procedures. Several key points conveyed in this paper are (*a*) pavements consume a significant portion of capital outlays made by highway transportation, (*b*) agencies should provide resources needed for design of pavements that are reflective of their large capital costs, and (*c*) the development of rational pavement design procedures is still in its infancy and will require significant resources (and patience) to complete.

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AASHO Road Test Effect on Pavement Design and Evaluation After 50 Years

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The AASHO Road Test, possibly the largest and most successful controlled civil engineering experiment ever undertaken, was conducted about 50 years ago. The results of the study are still widely used across the world. There were several things that made the Road Test successful, primarily the vision of Bill Carey, then associate director of the Highway Research Board (HRB); Walt McKendrick, chief of the Road Test; and Paul Irick, the project statistician, as well as the support of Fred Burgraaf, then director of HRB; Ted Holmes, director of research and planning for the Bureau of Public Roads (BPR); Alf Johnson, executive secretary of AASHO; and Frank Turner, chief engineer of the BPR. Carey and McKendrick carefully selected a good staff and allowed that staff to do good work.

Significant results from the Road Test still govern pavement design worldwide including (*a*) equivalent single-axle loads (ESALs), (*b*) the serviceability–performance concept, (*c*) effects of layer thickness and strength, and (*d*) effectiveness of dowels and joint spacing. The Road Test results are the basis for pavement design still widely applicable and currently used. It also changed the way that people conduct pavement research by illustrating the power of factorial experiments, high-quality data, and statistical analysis.

EVALUATION AFTER 50 YEARS

The AASHO Road Test was conducted about 50 years ago (1956–1961) in Ottawa, Illinois. It is possibly the largest and most successful controlled civil engineering experiment ever undertaken. The results of the study are still widely used across the world.

Several things made the Road Test successful, and nearly all of them were related to the people in charge. The original planning was done by Bill Carey, then associate director of the HRB; Fred Burgraaf, then director of the HRB; and Ted Holmes, director of research and planning for BPR. They were supported by Alf Johnson, executive secretary of AASHO (later to become AASHTO), and Frank Turner, chief engineer of the BPR. This group laid out the questions they needed to answer and then turned the work over to Bill Carey, Walt McKendrick,, and a carefully selected Road Test staff. They then allowed that staff to do their work.

Some 40 years later an attempt was made by pavement researchers to extend the work of the AASHO Road Test through the SHRP. The field conditions, data collection methods and program goals were not the same. As such, the results from SHRP and the ensuing LTPP cannot really be considered an extension of the AASHO Road Test results. A different set of results were achieved, including the Superpave design mix. In a sense, this makes the results of the AASHO Road Test even more historically relevant.

PEOPLE WHO MADE IT HAPPEN

By any standard, the Road Test was a remarkable project and its lasting effect is even more remarkable. Most of what has been written about the AASHO Road Test has and continues to relate to its contributions to the body of knowledge for design and performance of pavements. What was there about the AASHO Road Test that caused it to be of such wide interest, nationally and internationally, with results accepted and applied worldwide? In the 1950s, Congress had approved funds for a massive construction program referred to as the Interstate Defense Highway System. After World War II engineers were looking for better pavement design and performance information in the face of ever-increasing size and number of trucks to transport freight, and administrators were willing to support research to solve those problems.

Engineers realized that pavement design needed to be upgraded, time was of the essence, and administrators agreed to fund field trials that could produce results rapidly and test roads seemed one answer. While a series of test roads were originally planned, and one was completed using conventional asphalt concrete (AC) surfacing (WASHO Test Road in Idaho), a decision was made to proceed with a single large project including both rigid and flexible surfacing as well as bridges. A great deal of the credit for the project rests with the top administrators of AASHO, BPR (FHWA), U. S. Department of Defense, and HRB who were responsible for organizing and planning the project and with those groups that were represented on the various advisory committees and panels, including the American Petroleum Industry, American Trucking Industry, Automobile Manufacturers Association, academia, private consultants, and the tire industry—virtually the entire transportation industry. A complete list of participants can be found in seven Special Reports issued by HRB (*Special Report 62*). An important group of advisors were included in the technical advisory committees and panels created to provide input on everything from construction, maintenance, finance, instrumentation, statistics, data analysis, and performance ratings. The inclusion of this extremely wide representation helped to give the results credibility when they were published in 1962. Added to this credibility factor was the AASHO Road Test Conference in May 1962, at which time an additional report was issued by the Road Test staff and participating groups to provide potential applications of the results and an opportunity for critical comments and discussion of the staff analysis from potential users of the Road Test analysis and reports (*Special Report 73*).

An extremely important part of the equation leading to the credibility of findings and acceptance and application of results was the exceptional staff assembled by the HRB to do the hands-on work necessary to achieve assigned goals. The following sections present some personal comments and impressions to help readers appreciate "who were those guys that helped turn a dream into reality and finish an important job."

Project Staff

Walter "Walt" McKendrick, Jr.

As project director, McKendrick was responsible for the overall operation of the project. McKendrick had years of experience with the Delaware State Highway Department rising to the position of chief engineer before taking on the lead managerial role on the Road Test. He served as the primary representative to the "outside world" during the testing phase. One of the stipulations specified by AASHO was not to release data or analysis from the project until the traffic testing had been completed and the results analyzed by the project staff. One very important goal established by AASHO was to achieve at least 1 million load applications on each of the test sections. This goal became so important that the number of test vehicles (trucks) was increased in January 1960 and operations were increased from 6 days a week to 7. Traffic operated 18 h a day, rotating around the clock; thus pavement maintenance, the huge amount of routine measurements and special studies needed to be completed during that 6-h rest period. Special studies, in particular, became a problem at times on both the flexible and rigid test sections as they involved installing and calibrating various transducers needed for measurements or special equipment to make the measurements and the field staff would complain that 6 h was not sufficient time. Walt was sympathetic and would simply say, "Take all the time you want but in 6 h the trucks will be back on their way." His effort to make everyone on the project feel valuable created an atmosphere of trust and success. Walt loved to hunt and play cards and was good at both. Walt went on to work for the Portland Cement Association (PCA) and finished his career with FHWA as director of international programs. He died in 2003.

William "Bill" Carey, Jr.

William "Bill" Carey, Jr., as chief engineer for research, was primarily responsible for the research effort associated with all phases of the project. Carey was a person with a great imagination and willingness to listen to the staff ideas and solutions. He was one of the main reasons that there was a spirit of cooperation among the working staff. Bill's efforts are reflected in the seven volumes of a Special Report issued by HRB summarizing findings and possibilities. Bill had been the project director on the WASHO Test Road and brought that experience to the AASHO Road Test. Well liked and respected by the staff, Bill helped to bring the working staff together as a team, sometimes under very difficult working conditions. One legacy he will be remembered for was the development of a relatively simple profilometer to measure pavement profile; named the CHLOE profilometer after its developers: Carey, Huckins, Leathers, and "other engineers." Bill played bridge and was a better-than-average golfer, but research was his hobby. He went on to become the director of HRB later renamed the Transportation Research Board (TRB). He died in 1994.

The next tier of staff was remarkable because individually they were so different in personality and backgrounds but worked together as a team, a trademark of the AASHO Road Test staff. Yet there was a certain amount of friendly competition between the two groups; e.g., the flexible pavement staff complained (jokingly) that all asphalt sections were on the north-side tangents where the freeze–thaw conditions were more extreme to the disadvantage of the flexible sections. Conversely the rigid pavement staff complained there was more rain on the south tangents. The tangents were only 100 ft apart.

A. C. "Benk" Benkelman

A. C. "Benk" Benkelman, flexible pavement research engineer, was a former employee of the Minnesota Highway Department and BPR. Benk was the senior member of the Senior Staff in both age and experience. He had been a member of the research staff on the Hybla Valley Test Tract in Virginia (asphalt) and the Road Test 1-MD (concrete) in Maryland in 1950 and a key member of the research staff on the WASHO Test Road (asphalt) in Idaho, where he helped develop the Benkelman Beam. Benk was of the old school of researchers who relied on empirical and anecdotal personal observations. He was a man of the pavement and liked to spend a considerable amount of time walking the pavements and mentally analyzing what was going on. At meetings he would often introduce himself as the engineer who had previously been on Hybla Valley, Maryland, and Idaho Test Roads and if he didn't get it right "this time," they were going to "fire" him.

Benk was somewhat overwhelmed by the extensive use of statistics and the huge amount of data being generated from the 468 asphalt sections as were some of the staff at all levels. Benk could do more with an 8x10 sheet of graph paper and deflection measurements or cracking data than most of us could with a computer. He had a "feel" for the pavements. He loved to play bridge and play golf, using a 3 wood from tee to green. All the field staff recognized Benk as he was rather short and loved to have him on the project. He died in 1980.

Frank Scrivner, Rigid Pavement Engineer

Frank Scrivner, rigid pavement engineer, spent many years in pavement research for the Texas Highway Department. In 1954 he went to England to work on pavement evaluation for the U.S. Air Force. He was enticed back from that work by Bill Carey to become the research engineer for rigid pavements at the Road Test. Frank was a highly respected mathematician with a broad knowledge of portland cement concrete (PCC) pavements and was ideal to lead the rigid pavement group. Rigid pavements tended to be more amenable to calculations of stress and strain; and there were a number of theories available to be tested and Frank knew them all. Before going to England, Frank and a group of 10 technicians operating Marchant hand calculators had calculated the first comprehensive set of stress coefficients for three layers systems to be applied to flexible and rigid pavements. Frank did most of the planning for the technical details of the rigid pavement experiments and the design of those experiments with Paul Irick. He left most of the field operations to his assistant chief, W. R. Hudson, who is a coauthor of this paper. Fred Finn, another coauthor, served as assistant chief to A. C. Benkleman. Frank did not have many hobbies. He played a little bridge, but no golf. Mathematics and physics were his hobbies, and he indulged them 16 h a day. Following the Road Test, he joined the staff at Texas A&M University and continued his research effort there with the Texas DOT. He died in 1988.

Paul Irick, Chief, Data Processing and Analysis

Paul Irick, a professor of applied statistics at Purdue University became, in a manner of speaking, the heart of the project. After the initial processing, all data collected on the project came to Paul and his staff for further processing, storing, and analysis. Actually, along with Bill Carey and the Statistical Panel, Paul played a key role in the design of the factorial experiments in such a way as to eliminate any possibility of confounding the interpretation and to allow measurement of the degree of experimental error. The members of the staff at all levels soon found that it would be essential to become familiar with a new language, the language of the statistician. Terms like sums of squares, linear regression, randomization, replication, standard error, variance, correlations coefficients, confounding, confidence levels, accept once and rejection criteria, risk, and probability were all part of that new language.

Paul was a great listener and was always willing to take the time to help staff members understand analysis of the data. Certainly he was one of the most admired professionals on the project as a patient and caring person. Paul, like other members of the Senior Staff, played bridge but not always with the same level of passion. His legacy can be found in the fact that pavement research had never been the same since the Road Test, on the basis of an expanded use of statistics. Paul, like Bill Carey, went on to senior level positions at TRB, where he continued to apply his talent for 20 more years. He died in 1996.

Rex Leathers, Engineer of Special Assignments

Rex Leathers, engineer of special assignments, an engineer with the BPR and a carryover from the WASHO Road Test, was the perfect person for special assignments. As with any project of this size the unexpected was the norm of the day, and Rex was the person to handle the unexpected. Intelligent, resourceful and fearless, Rex became one of the gears that kept the machines running. Rex moved on through the ranks to become assistant director of engineering for FHWA. He was good at everything he did, including bridge (he played with a passion but remembered it was only a game), hunting, and golf were other hobbies. He died in 1998.

James F. Shook, Materials Engineer

Jim Shook, materials engineer, came to the project from the National Sand and Gravel Association Laboratories. During the major construction and traffic phases, he was responsible for the testing and reporting of material properties from the field and in the laboratory, which included all testing needed to control the uniformity of construction from the subgrade materials to surfacing. Rapid test results were important to accept or reject each construction unit according to criteria established by Dr. Irick and his staff and minimize construction delays. Jim efficiently managed the materials staff under considerable pressure of time. He lives near Washington, D.C.

Ivan Viest, Bridge Enginner

Most people don't know that the AASHO Road Test also contained a special experiment on the fatigue evaluation of 16 short-span bridges. Ivan was bridge research engineer for the study supported by John Fisher as his assistant. Ivan came from the University of Illinois and was a dynamo in setting up and operating bridge research under the repeated traffic loadings available at the AASHO Road Test. Ivan was active in professional affairs, including the American Society of Civil Engineers. After the Road Test, he joined Bethlehem Steel Corporation where he retired. John Fisher, his assistant, went on to get his PhD and joined the faculty of Lehigh University. Both live in Bethlehem, Pennsylvania.

The second level of staff at the AASHO Road Test was equally well qualified. In addition to John Fisher in Bridges, there were Lloyd Dixon, a staff member of the Illinois DOT, and Fred Finn, as previously mentioned. Ronald Hudson was assistant rigid pavement engineer, working closely with Frank Scrivner, supported by Bob Little and Bud Wright of the Illinois DOT. During the construction phase and early testing phase of the Road Test, there were some 30 engineers from Illinois DOT supervised by Emmett Chastain and Art Tosetti, who were indispensable in getting the roads constructed to the high-quality standards needed for the test. Bob Hain served as assistant to Paul Irick in the data-processing and analysis section. Howard Boswell served as chief maintenance engineer. Howard was a BPR engineer who had also worked on the WASHO Road Test with Rex Leathers.

During the Road Test the BPR rotated through approximately 50 trainees for a 6-month period each. Many of these went on to important jobs in the Federal Highway Administration, particularly including Les Lamm, Dick Morgan, and Dean Carlson, all of whom served as chief engineer of FHWA before their death or retirement. We apologize to the many other contributors to the Road Test whom we've omitted from this report. None of these omissions are intentional; all played a very strong role in the study.

WHERE WAS PAVEMENT TECHNOLOGY BEFORE THE AASHO ROAD TEST?

It is hard for modern engineers to believe that highly technical pavement design is relatively new. At the time of the AASHO Road Test (1958), the California bearing ratio (CBR) was the standard method of flexible pavement design. Three main variables were considered: (*a*) load, (*b*) subgrade strength, and (*c*) total pavement thickness. The WASHO Road Tests, a $3-million study (1954), had suggested that asphalt surface thickness was related to pavement life. A 2-in. thick pavement surface in that test carried approximately twice as many heavy load applications to failure as a 1-in. pavement surface subjected to the same traffic. The Interstate highway program was just beginning and this was a strong emphasis for learning more about pavements. We must remember that available computer power at that time did not permit rapid calculation of layer stresses.

At that same time, PCC design methods used primarily the PCA design method based on modified Westergaard theory for designing PCC pavements. Subgrade and subbase pumping under heavy load was a major problem. Corner cracking was a major failure mode, and pavement faulting was a serious problem, since load transfer using steel dowels was not yet an accepted practice. Little was known at that time about the value of stronger subgrades and subbases and the benefits of stabilized materials.

INTERSTATE DEFENSE HIGHWAY SYSTEM IS AUTHORIZED

BPR was charged in the Interstate Highway Act with determining the proper allocation of cost that vehicles should pay to use the new Interstate system. This was called the 210 Study. Frank Turner, the chief engineer, and Ted Holmes, director of highway research and planning of the BPR, went to HRB to seek assistance. They met with William Carey, Jr., and Fred Burgraaf, and after several discussions a plan was developed to conduct a major Road Test, which later became the AASHO Road Test. Alf Johnson, then the executive secretary of AASHO, was brought into the picture and helped sell the idea to the 48 states to gain funding support. This early planning was largely successful because Bill Carey had gained great experience in pavement research at the WASHO Road Test and the Maryland Road Test. Ted Holmes was director of both planning and research. The planning covered the cost allocation portion, and his research duties covered the pavement research portion. Thus, a small group of about five people were able to make the AASHO Road Test happen.

Bill Carey approached the Asphalt Institute and PCA, which both became strong supporters and contributors to the study. However, they strongly requested and succeeded in ensuring that the study did not compare the two pavements against each other. The experiments were totally independent. Unfortunately this did not keep the general public and unscrupulous salesmen from comparing the results after the Road Test was over. Thus 20 years of bickering between the two pavement types started; this in some ways continues to this day.

The Truck Manufacturers Association and the American Trucking Association were approached, and both made strong contributions, as did many state DOTs, including Illinois, where the test was conducted on a portion of the right-of-way on I-80. As history shows, the Teamsters Union was strong in the United States during that period. It would have been practically impossible to conduct the Road Test, which required around-the-clock operation of heavy trucks, if we had been required to deal with union labor truck drivers. This problem was solved when the U.S. Army agreed to provide truck drivers, and a company of Transportation Corps truck drivers were assigned to live in barracks constructed on the AASHO Road Test site and drive in three shifts around the clock.

TECHNOLOGY DEVELOPED AND LESSONS LEARNED OR DEVELOPED AT THE AASHO ROAD TEST

Many things were developed and learned at the AASHO Road Test. Some for the first time, others were reinforcements of existing theories and ideas developed at the WASHO Road Test and presented by theory. We do not have time in this paper to cover all these items. Please refer to the special reports from the Road Test [*Special Report 62* (7 volumes) and *Special Report 73*] for more details. We've also included a partial bibliography. We have not had time to codify all the details of individual references. But any student or practicing engineer interested can find these provided a useful road map to his studies. The results of the AASHO Road Test were directly used in 1960 to 1962 to develop the first-ever *AASHO Pavement Design Guide*. The first guide was produced in 1962 in interim form and remained interim for more than 10 years before the first version was actually adopted.

MAJOR TECHNICAL FINDINGS OF THE AASHO ROAD TEST

Surface Thickness

The AASHO Road Test gave quantitative value to the importance of pavement surface thickness in increasing the number of load repetitions that can be carried to pavement failure. It tied pavement surface thickness to pavement performance, where "performance" is defined as the service provided by the pavement or the number of load repetitions that can be carried to an unserviceable level.

Load Equivalency

Pavement engineers had long had trouble dealing with various axle loads in pavement design. Some methods used only the heaviest load (CBR), and others including the Texas Design Method used the average of the 10 heaviest loads that were expected to be carried on the pavement. The AASHO Road Test provided quantitative information about the relative damaging effect of heavy loads, and immediately after the Road Test, Paul Irick and Frank Scrivner used the Road Test equations to generate load equivalencies called ESALs. Francis Hveem of the California DOT had earlier hypothesized a load equivalency concept tied to 10-kip axles. The Road Test equivalencies validated and extended the Hveem hypothesis statistically.

The load equivalency concept (ESAL) is by far the most widely used pavement concept in the world. We as authors have collectively visited more than 50 countries and all 50 states in the United States. All of these agencies use the ESAL concept in pavement design.

PSI: Performance Concept

Before the AASHO Road Test there was no good definition of pavement failure. This seems hard to believe but please check the literature; you will find it to be true. After the WASHO Road Test, Paul Irick and Bill Carey developed the Present Serviceability Index (PSI) concept and defined "performance" as "accumulated traffic to a fixed level of PSI." The selected level of PSI was "failure." While many agencies adopted this concept, some have continued to refer to "roughness." Therefore, a defined level of roughness is sometimes accepted as failure in the form of an International Roughness Index (IRI) level. The technical literature shows that IRI and PSI are inversely related to each other.

The present serviceability concept (PSI) relates pavement failure directly to riding quality and the acceptance or satisfaction of the riding public. It is indeed more definitive of true performance than roughness alone and strong consideration should be given to resurrecting it in pavement studies and designs.

Layer Equivalencies: Material Properties

The AASHO Road Test included four types of base under asphalt pavements: (*a*) river gravel, (*b*) cement stabilized, (*c*) asphalt stabilized, and (*d*) crushed stone. These were compared to define the levels of performance that resulted from improving the quality of the base layer. Francis Hveem had also hypothesized such relative benefit of stronger layers as part of a "gravel equivalency concept" and he was instrumental in getting the wedge-shaped base sections added to the Road Test to validate that concept. The structural number concept, developed based on layer equivalencies, is widely used around the world and is the basis for layer selection in all AASHTO Pavement Design Guides up to 2002.

The Road Test of course was not perfect because it was impossible to make it large enough to solve all possible factors. We don't know if these layer equivalencies would be the same with different subgrades and in a different environment. These questions have been the subject of considerable research in the past 50 years.

Value of Subbase to Reduce Pumping in Rigid Pavements

At the Road Test those PCC pavement sections that had a gravel subbase under the slab performed much better than those that were placed directly on the clay subgrade. This occurred regardless of the thickness of the gravel subbase layer. However, there were no stabilized subbases used on the rigid pavements and we can only hypothesize what improvement would have resulted.

Pumping of Subbase and Subgrade Materials

Before the Road Test the PCC paving industry had strongly hypothesized that the problem of pumping of subgrade material from beneath pavements could be solved by placing a granular subbase beneath the slab. This was proved to be incorrect at the Road Test, where under heavy loads and high rainfall, even the gravel subbase layer pumped and caused early slab failure.

Effectiveness of Dowels for Load Transfer

Before 1960 most people were of the opinion that it was necessary to put some form of positive load transfer across joints and cracks in PCC pavements. Yet the concrete industry continued to claim that thicker pavements would solve the problem. The Road Test used load transfer dowels in all pavement sections. There was no faulting at the AASHO Road Test at cracks or joints, thus validating the effectiveness of dowels for load transfer under extremely heavy loads up to 30,000 pounds on a single axle.

Joint Spacing

Two joint spacings were used at the AASHO Road Test: 15-ft joint spacing with no reinforcement steel and 40-ft joint spacing with mild reinforcement. Both of these joint spacings performed well under heavy loads up to 30-kip single axle and both contained dowels across the joints. The 40-ft slabs cracked at approximately 12- to 15-ft spacing, and no faulting occurred at those cracks during the test. However, 15 years later, field studies of some of these same sections left in service on IH 80 did show faulting as the mild reinforcement steel rusted and lost its effectiveness.

Limitations of the AASHO Road Test Findings

Nothing is perfect, and there are several limitations of the AASHO Road Test findings. They are as follows:

1. One subgrade only—the Road Test was carried out on a lean clay subgrade and therefore no direct inference to other subgrades can be made.
2. Only 2 years long—the Road Test was conducted during the period October 1958 to December 1960 and related primarily to this time period and the climatic conditions existing during that period.
3. One environment only—the Road Test took place in Central Illinois, which is a freeze–thaw, wet environment. Information is needed to extend it to other environments.

4. One AC mix only—a single high-quality AC was used in the AASHO Road Test. Information related to other qualities of asphalt concrete surfacing must be inferred in other ways.
5. One PCC mix only—a single high-quality PCC was used in the AASHO Road Test. Information about the performance of other strengths of PCC must be inferred from other sources.

It is important to point out however that these factors limit only the "inference space" for the results, not the "validity" of the results themselves. The AASHO Road Test results are the most complete and valid experiment every conducted on pavements. There are a number of theoretical and empirical methods of extending the inference space to other subgrades, other environments, other concrete mixes, and other AC mixes.

Extending the AASHO Road Test Results

It was well understood at the Road Test and subsequently that the Road Test applied only to the environment and the soil conditions in central Illinois when the test was conducted. The test was a $500-million project (in 2005 dollars). It was impossible to extend the study to all regions although that would have been a desirable goal for a second phase. To adapt these results, nearly every state DOT had a project funded with FHWA research funds, entitled the Application of AASHO Road Test Results to XYZ State (fill in the blank with any of the 40 states). These results were used in implementing the AASHTO Pavement Design Guides nationwide.

GENERAL CONCEPTS IN PAVEMENT ENGINEERING THAT RESULTED OR WERE GREATLY IMPROVED BY THE AASHO ROAD TEST

Renewed Interest in Pavement Engineering

The AASHO Road Test sparked a renewed interest in pavement engineering and research worldwide. Even though the AASHO Road Test itself was originally done to prove the relative damage and cost of various axle loads (the famous 210 Study), it really produced more information and more interest in pavement engineering per se than it did in cost allocation. It did, however, produce the necessary information needed for cost allocation, and that information was subsequently used in setting fuel tax levels in the United States.

Modeling Pavement Performance

The AASHO Road Test sparked interest in the modeling of pavement performance, and the AASHO Road Test equations themselves subsequently became the equations for the Pavement Design Guides. The Design Guides from 1962 up until the present date were primarily based on the performance equations developed originally by Paul Irick and then expanded and continued by the Asphalt Institute, by Frank Scrivner at the Texas Transportation Institute, and by many others.

The Importance of Quality Data

The AASHO Road Test showed that it was important to have high-quality complete data if meaningful answers were to be produced from costly pavement research. The test showed the need for well-thought-out experimental designs and the necessity of collecting complete data that fulfilled the factorial of those designs for valid analysis. The study also showed the necessity to follow through on "all" data collection and the requirement to develop "complete" data sets for valid analysis

Factorial Experimental Design and Testing

The AASHO Road Test showed the value of statistically designed experiments that were large enough to cover the inference space effectively. This plan has been followed by many pavement researchers over the years since the Road Test.

Good Statistical Analysis of Data

This concept goes along with good factorial design, but it should be remembered that good statistical analysis can be and is used in other types of research also. More pavement research engineers now use statistical analysis in all aspects of their work than would be using it had not the AASHO Road Test and Paul Irick pointed the direction.

Pavement Evaluation

The AASHO Road Test sparked renewed interest in evaluating pavements, primarily using roughness and the PSI but also expanding the use of distress surveys and deflection measurements. The availability of deflection to evaluate pavement behavior, distress measurements to evaluate pavement condition and roughness measurements to calculate serviceability and evaluate pavement performance has permitted the development of the pavement management concept.

Implementation Conferences for the AASHO Road Test

Following the St. Louis Conference, which provided information on early extension and implementation of Road Test results, the Asphalt Institute and the University of Michigan sponsored significant international asphalt pavement conferences that focused attention not only on the results of the AASHO Road Test asphalt pavements but also on mechanistic pavement design. These conferences have continued every 4 or 5 years since 1962. Clearly the AASHO Road Test was the spur for these conferences and to these improvements in pavement design.

SUMMARY

Many engineers who attended this seminar or read this document are aware of many more details that could have been added here. Our hope in this paper is to encourage you to read many of the documents that relate to the details touched on herein. Start with TRB Special Reports 62 and 73.

They will lead you to a far broader understanding of the AASHO Road Test and its impact. Follow that by delving into the early AASHO Pavement Design Guides and many of the references presented there.

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What Pavement Research Was Done Following the AASHO Road Test and What Else Could Have Been Done but Was Not

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The AASHO Road Test provided significant results that led to improved pavement design in the near term following the Road Test (*1*) and to an expanded research effort by the pavement engineering community worldwide. In particular, it resulted in the development of what is termed today M-E pavement design. Results of the Road Test also contributed to the development of nondestructive pavement evaluation, now an important part of pavement M&R considerations, including overlay pavement design, and to the development of pavement management concepts embodied in PMSs currently in use. These developments are briefly summarized in this paper.

Many people and organizations have been involved; references are made to the contributions of some of the members of the pavement engineering community, both in North America and elsewhere, particularly Europe. It is important to state that the engineers involved have shared freely their knowledge through the venues of two major international conferences, one for flexible (asphalt) pavements and the other for rigid (concrete) pavements. The resulting information has contributed significantly to the state of pavement engineering today. Both conferences were introduced in response to the renewed interest in pavement engineering sparked by the AASHO Road Test.

Some important pavement related research was not initiated following the Road Test, some of which was delayed for almost 40 years. A brief summary and a few examples of this not-included research are also included.

INITIAL DISSEMINATION OF AASHO ROAD TEST RESULTS

Results of the Road Test were presented at a meeting sponsored by the Highway Research Board (HRB) in St. Louis, Missouri, May 16 to 18, 1962. Information from this meeting was subsequently published in *Special Report 73* (*2*). Fred Burggraf was executive director of HRB at the time, and A. E. Johnson was director of AASHO. A national advisory committee was appointed by HRB. K. B. Woods of Purdue University served as chairman of this committee and W. A. Bugge, then director of the Washington Department of Highways, was vice chairman. HRB appointed staff to conduct the Road Test, as described in a paper presented at the 85th Annual Meeting of the Transportation Research Board in Washington, D.C. (*1*). Many distinguished pavement engineers served on the advisory committee. The 36 members included representatives from AASHO, the Bureau of Public Roads, state highway departments, the asphalt and concrete industries, academia, the tire industry, and automobile and truck associations. Appendix A contains a listing of the complete membership.

Sessions at the 1962 conference included the following:

1. Background and History, F. Burggraf, A. E. Johnson, K. B. Woods, and W. B. McKendrick, Jr.;
2. Bridge Research, I. Viest;
3. Selected Special Studies;
4. Pavement Performance, A. C. Benkelman (Flexible) and F. Scrivener (Rigid);
5. Pavement Research Forum; and
6. Use of Road Test Research Findings.

In the paper presented at the 85th Annual Meeting of the Transportation Research Board (*1*), the role of W. N. “Bill” Carey, Jr., as chief engineer for the Road Test was summarized. Much of the success of the program must be attributed to him in that he provided an environment that allowed for innovation. In his capacity as chief engineer, he carefully weighed the comments of the advisory group as well as others. While often accepting their advice, he shielded the research staff from costly and nonproductive suggestions, allowing them the freedom to explore innovative ideas. Figure 1 contains photos of Burggraf, Carey, and Woods.

In the period between the completion of traffic applications on November 30, 1960, and the preparation of the research reports by the Road Test staff and their presentation at the St. Louis Conference in May 1962, pavement groups in both the asphalt and concrete industries were evaluating the results as well.

Significant pavement research and development programs were instituted. Also, planning for two pavement conferences was initiated to provide venues for widespread dissemination of the information being developed to the world pavement engineering community. These venues have become known as the International Conferences on Asphalt and on Concrete Pavement Design, respectively. The 10th Conference for Asphalt Pavement was held in Quebec, Canada, in August 2006, while the 8th Concrete Pavement Conference was held in Colorado Springs, Colorado, in August 2005. Developments in pavement design and rehabilitation resulting from these activities will be briefly described in the following sections.

FLEXIBLE (ASPHALT CONCRETE) PAVEMENT DESIGN

At the time of the AASHO Road Test, the Asphalt Institute was the primary representative for the asphalt pavement industry in the United States. The first International Conference an Asphalt Pavement (termed the International Conference on the Structural Design of Asphalt Pavements)

(a) (b) (c)

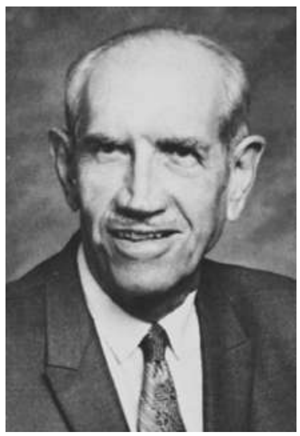
FIGURE 1 Some key engineers associated with the AASHTO Road Test: (*a*) Fred Burggraf, Highway Research Board; (*b*) K. B. Woods, Purdue University; and (*c*) W. N. "Bill" Carey, Jr., Highway Research Board.

was initiated by J. E. Buchanan (institute president) with input from F. N. Finn (the Institute's representative on the Road Test) to provide a technical venue for discussion of the Road Test results as well as for worldwide developments in asphalt pavement design at the time. The University of Michigan (UM), because of its excellent reputation in the pavements area in the United States, was selected as the conference site. W. S. Housel and W. K. Parr of the UM Civil Engineering Department, working with Asphalt Institute staff and key U.S. and international members of the paving community, developed a most successful conference in August 1962. Figure 2 contains photos of Buchanan, Finn, Housel, and Parr.

These conferences have had extensive international as well as U.S. representation. Some of the key participants at the first conference included A. C. Benkelman (AASHO Road Test flexible pavement research engineer); D. M. Burmister (Columbia University; developer of the first solutions for multilayer elastic systems subjected to surface loads); N. W. McLeod (consultant, Imperial Oil Company, Canada); W. H. Goetz and E. J. Yoder (Purdue University, professors); and F. N. Hveem (materials and research engineer, California Division of Highways). Subsequent conferences included well-known representatives from many countries and organizations—a few examples are William Glanville (director, Road Research Laboratory, United Kingdom); P. J. Rigden (director, National Institute for Road Research, South Africa); K. Wester (manager, Road Research Center, Holland); E. Nakkel (Federal Ministry of Transport, West Germany); J. M. Kirk (Danish Asphalt Industries, Road Research Laboratory, Denmark); and J. Bonitzer and R. Sauterey (chief engineers, LCPC, France) (Figures 3 and 4). Other participants will be referred to in connection with technical developments emerging from the conferences and described in the following paragraph.

While many of the elements of M-E pavement design were being worked on before the first conference, the framework for this approach "gelled" there, particularly through the efforts of Shell, the Asphalt Institute, and the UK Road Research Laboratory investigators.

(a) (b)

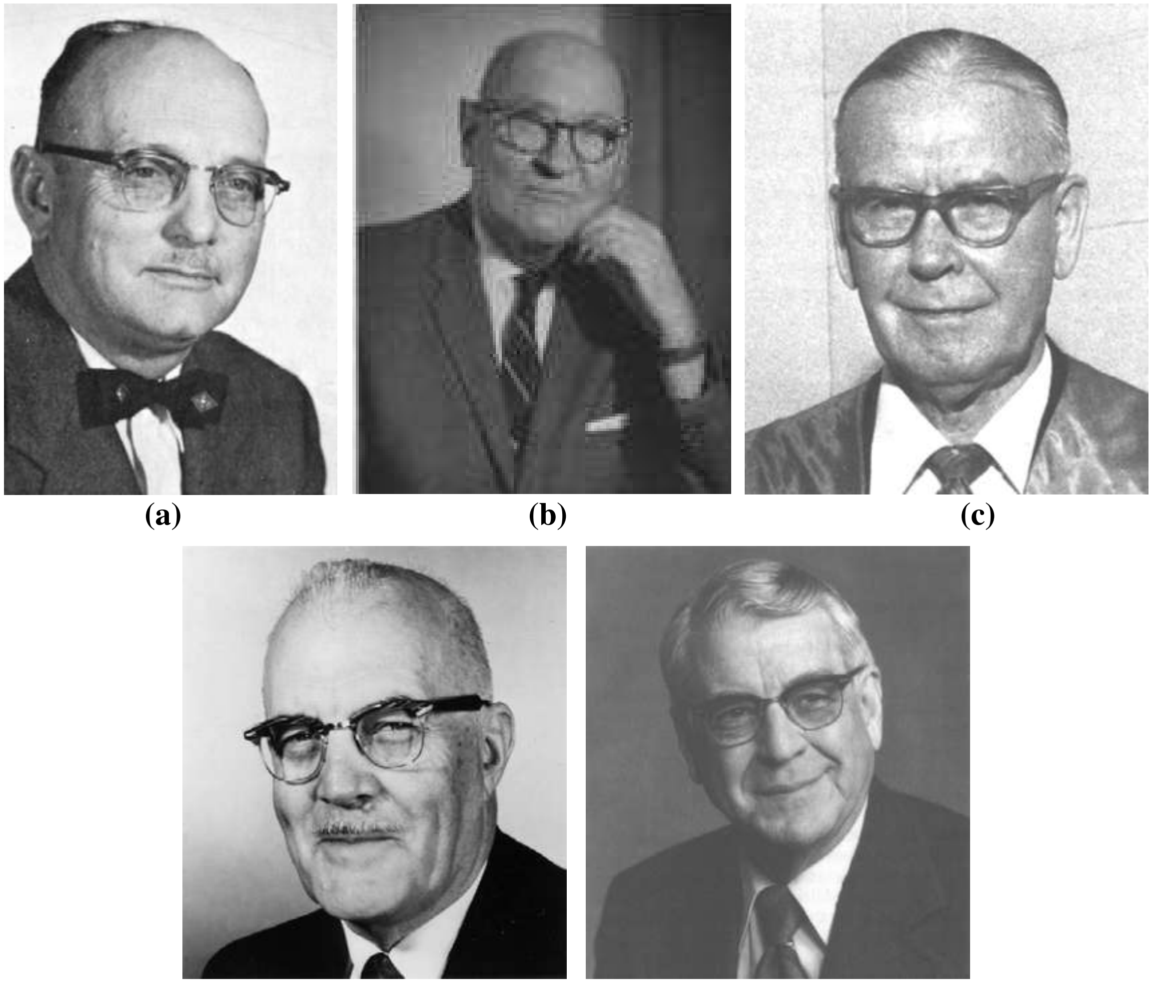


(c) (d)

FIGURE 2 Individuals responsible for First International Conference on the Structural Design of Asphalt Pavements, University of Michigan, August 1962: (*a*) J. E. Buchanan and (*b*) F. N. Finn, Asphalt Institute; and (*c*) W. S. Housel and (*d*) W. K. Parr, UM Civil Engineering Department.

Before the conference, the Asphalt Institute had provided funds to support research at the University of California at Berkeley (UC Berkeley)for repeated load testing of the AASHO Road Test subgrade soil (H. B. Seed, Figure 5), determination of viscoelastic (VE) behavior of asphalt concrete (AC) (K. E. Secor and C. L. Monismith), and analysis of VE multilayer systems (R. Westmann, and K. S. Pister). The institute also provided support to the University of Idaho to evaluate low temperature cracking of AC (R. Lottman). At the conference, on the basis of the research on repeated load testing of the AASHO subgrade soils, Seed introduced the measure of soil stiffness termed *resilient modulus*, which is in use today (*3*).

During the period following the completion of the Road Test, Shell engineers used the results of the Road Test to establish concepts for pavement design that were introduced at the conference. These included use of the computed elastic vertical shear strain at the surface of the subgrade to mitigate surface rutting contributed by unbound materials in the pavement structure



(a) (b) (c)

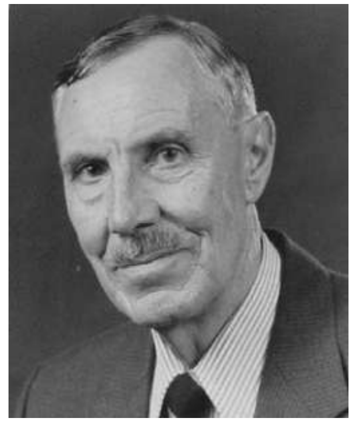
(d) (e)

FIGURE 3 Some participants in the First International Asphalt Pavement Conference, 1962: (*a*) A. C. Benkelman, Highway Research Board; (*b*) D. M. Burmister, Columbia University; (*c*) N. W. McLeod, Imperial Oil Company, Canada; (*d*) F. N. Hveem, California Division of Highways; and (*e*) W. H. Goetz, Purdue University.

by Dormon and use of the computed elastic tensile strain at the underside of the AC in contact with untreated base as the determinant of fatigue cracking in this layer by Peattie and Dormon. Both concepts were later to become key elements of a number of M-E design methodologies.

The use of tensile strain as the determinant for fatigue cracking was reinforced by extensive fatigue test data on both asphalts and mixes presented by Pell, whose research at Nottingham University had been supported by Shell. Subsequent to the conference, Peattie suggested the use of the linear summation of cycle ratios cumulative damage hypothesis to analyze the contributions of traffic loads of different magnitudes to hot-mix asphalt fatigue cracking. This concept is now used in many of the M-E design methods.


(a)


(b)


(c)

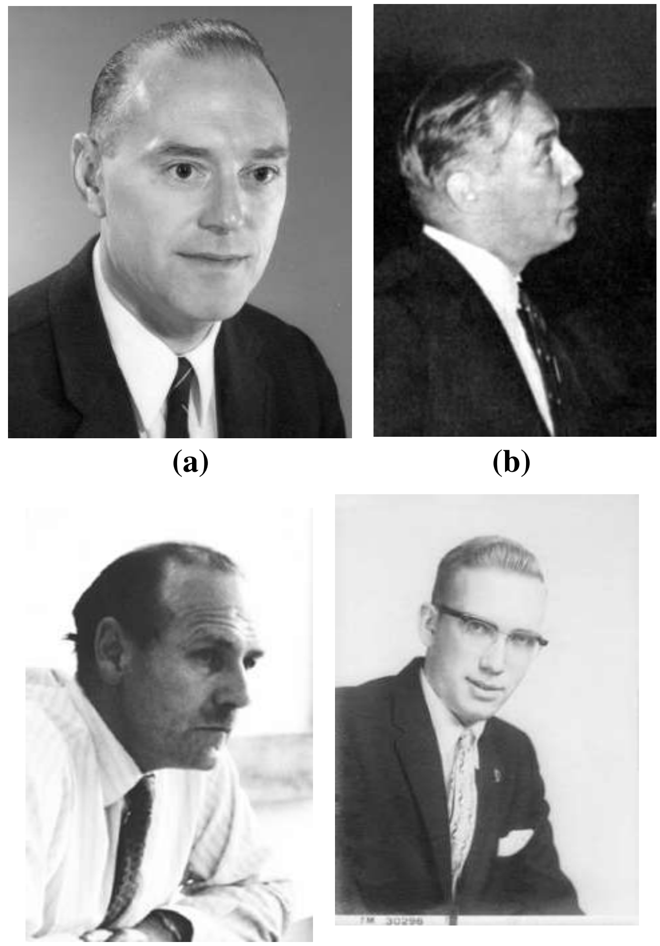

(d)


(e)


(f)


(g)

FIGURE 4 Some participants in subsequent International Asphalt Pavement Conferences: (*a*) William Glanville, Road Research Laboratory, United Kingdom; (*b*) P. J. Rigden, National Institute for Road Research, South Africa; (*c*) K. Wester, Road Research Center, Netherlands; (*d*) J. M. Kirk, Asphalt Industries, Road Research Laboratory, Denmark; (*e*) J. Bonitzer, LCPC, France; (*f*) R. Sauteray, LCPC, France; and (*g*) E. Nakkel, Federal Ministry of Transport, West Germany.



(a) (b)

(c) (d)

FIGURE 5 Contributors to First International Asphalt Conference: (*a*) H. B. Seed, UC Berkeley; (*b*) G. M. Dormon, Shell, United Kingdom; (*c*) P. S. Pell, University of Nottingham; and (*d*) E. L. Skok, Jr., Asphalt Institute.

At the Royal Dutch Shell Laboratory at Amsterdam, research was also under way on the use of nondestructive dynamic pavement testing by L. W. Nijboer, W. Heukelom (Figure 6), and A. Klomp to determine stiffness moduli of the various pavement components. This will be discussed subsequently.

The conference also accelerated the development of solutions for the behavior of multilayer elastic systems. With the introduction of the electronic computers, the methodology developed by Burmister in the 1940s for analysis of two- and three-layer elastic systems became tractable and thus became a part of the design systems.

At subsequent conferences, particularly the 1977 and 1982 conferences, complete and user-friendly M-E design systems were prepared, e.g., the Shell procedure (1977) and the Asphalt Institute procedure (MS-1) (1982). Eventually a number of systems following the above methodology were introduced worldwide (*5*). The most current AASHTO pavement design guide (proposed for balloting), developed using NCHRP, e.g., Projects 1-37 and 1-40, incorporates a number of the concepts presented at the international conferences (*4*).

RIGID (PORTLAND CEMENT CONCRETE) PAVEMENT DESIGN

In 1966 the Portland Cement Association (PCA) presented a new method for plain jointed-concrete pavement design using the concepts of cumulative damage noted above to select slab thicknesses (*7*). Analyses developed by Westergaard (*6*) (plate on a dense liquid subgrade) and presented in the form of influence charts by Pickett and Ray (Figure 7) (*8*) were used

(a) **(b)**

FIGURE 6 Royal Dutch Shell researchers and conference participants: (*a*) L. W. Nijboer and (*b*) W. Heukelom.

(a) **(b)**

FIGURE 7 Early contributors to concrete pavement analysis and design: (*a*) H. M. Westergaard, Harvard University, and (*b*) Gordon Ray, PCA.

to calculate slab thicknesses to mitigate fatigue cracking. Results of the AASHO Road Test permitted calibration of the design methodology.

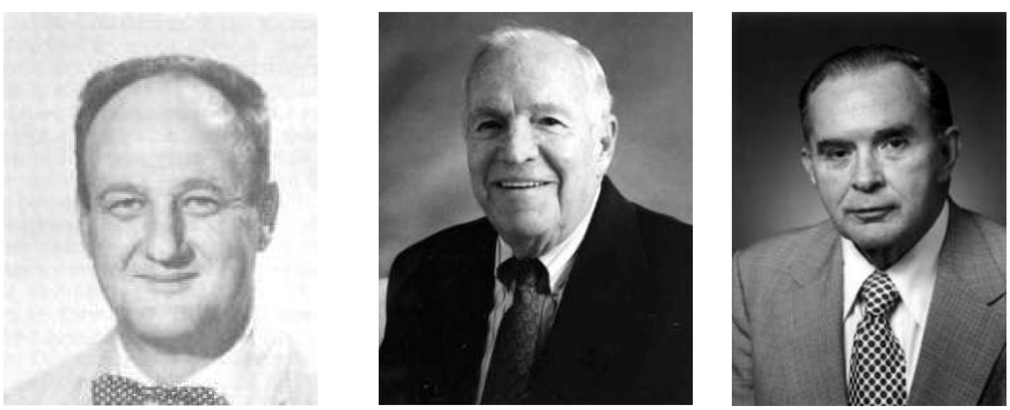
Like the asphalt research pavement design developments, research in concrete pavements was stimulated by the Road Test. Much of this research was presented at the First International Concrete Pavement Conference held at Purdue University in 1977. Key individuals in making this conference a success were E. J. Yoder of Purdue and G. Ray, R. Packard, and B. Colley of the PCA (Figure 8).

Subsequent conferences contained the results of improved methodology (*9*). With advent of the computer and the development of the finite element methodology, the design procedures were expanded to include the influence of dowel bars and tied concrete shoulders on thickness design, e.g., the procedures developed by Darter and Barenberg (Figure 9), in 1977 for the FHWA (*10*) and the development by Tayabji (Figure 9) and Colley and Packard of the PCA in 1984, the latter also considering the effects of pumping.

MECHANISTIC PAVEMENT ANALYSIS

The use of multilayered elastic analysis to represent asphalt pavement response, although developed by Burmister in the 1940s (*11*), did not receive widespread attention until the 1962 Asphalt Pavement Conference. At this conference, important contributions were made by Whiffen and Lister (*12*), Skok and Finn (*13*), Peattie (*14*) and Dormon (*15*). Both Whiffen and Lister and Skok and Finn illustrated how layered elastic analysis could be used to analyze pavement distress. As noted earlier, Peattie and Dorman presented several concepts based on such analyses that would later become a part of the Shell pavement design methodology.

A number of general solutions for determination of stresses and deformations in multilayered elastic solids also were presented at the 1962 conference. Additional related work was published in 1967 at the second international conference. These general solutions, coupled



(a) **(b)** **(c)**

FIGURE 8 Individuals responsible for First International Concrete Pavement Conference, Purdue University, 1977: (*a*) E. J. Yoder, Purdue University; (*b*) R. G. Packard, PCA; and (*c*) B. E. Colley, PCA.

(a) **(b)**

FIGURE 9 Contributors to concrete pavement analysis and design: (*a*) E. J. Barenberg, University of Illinois, and (*b*) S. Tayabji, PCA.

with rapidly advancing computer technology, fostered the development of the current generation of multilayered elastic and viscoelastic analysis computer programs. Table 1 contains a listing of some of the most commonly used programs. The ELSYM program, developed at UC Berkeley, by G. Ahlborn (*16*) and widely used[1], directly benefited from the 1962 and 1968 conference papers.

Computer solutions for layered systems in which the properties of each of the layers could be represented as linear viscoelastic materials were subsequently introduced; two available solutions, VESYS and VEROAD, are listed in Table 1.

In the late 1960s, finite element analyses to represent pavement response were developed by a number of researchers [e.g., Duncan et al. (*28*)]. Increasingly, the finite element method (FEM) has been used since that time to model pavement response, particularly to describe the nonlinear response characteristics of pavement materials. Examples of this approach include ILLIPAVE and FENLAP (*24*, *25*).

Recently, solutions for the dynamic analysis of AC pavement under moving, fluctuating loads have been developed. The SAPSI-M program (*27*), Table 1, is one such example. In this program, moving loads are modeled as a series of pulses with durations equal to the time requires for a load to pass a specific location.

In terms of current analytically based pavement design procedures, layered elastic analysis is the primary method for defining pavement response to load. Use of the FEM has had limited application to date (e.g., ILLIPAVE), possibly because of computational time constraints. Hence, it has been used primarily in special applications. However, improvements both in computer capabilities and in formulating finite element representations should allow it to become an integral part of routine pavement analysis of asphalt pavements. As will be seen in the following material, finite element analysis is an integral part of the design procedure for PCC pavement in the *AASHTO Mechanistic–Empirical Pavement Design Guide* (MEPGD), which is balloted for acceptance and implementation in the summer of 2007.

TABLE 1 Summary of Some Available Computer-Based Analytical Solutions for AC Pavements

Program	Theoretical Basis	Number of Layers (max.)	Number of Loads (max.)	Program Source	Remarks
BISAR (*18*)	MLE	5	10	Shell International	The program BISTRO was a forerunner of this program
ELSYM (*16*)	MLW	5	10	FHWA (UCB)	Widely used MLE analysis program
PDMAP (PSAD) (*19*)	MLW	5	2	NCHRP Project I-10	Includes provisions for iteration to reflect non-linear response in untreated aggregate layers
JULEA (*20*)	MLE	5	4+	USACE WES	Used in Program LEDFAA
CIRCLY (*21*)	MLE	5+	100	MINCAD, Australia	Includes provisions for horizontal loads and frictionless as well as full-friction interfaces
VESYS (*22*)	MLE or MLVE	5	2	FHWA	Can be operated using elastic or viscoelastic materials response
VEROAD (*23*)	MLVE	15 (resulting in half-space)		Delft Technical University	Viscoelastic response in shear; elastic response for volume change
ILLIPAVE (*24*)	FE		1	University of Illinois	
FENLAP (*25*)	FE		1	University of Nottingham	Specifically developed to accommodate non-linear resilient materials properties
SAPSI-M (*26,27*)	Layered, damped elastic medium	N layers resting on elastic half-space or rigid base	Multiple	Michigan State University/ UC Berkeley	Complex response method of transient analysis—continuum solution in horizontal direction and finite element solution in vertical direction

MLE—multilayer elastic
MLVE—multilayer viscoelastic
FE—finite element

In the concrete pavement area, the use of elastic analysis was first introduced by Westergaard in 1925 (*6*). He represented the jointed concrete slab as a plate on a dense liquid subgrade and presented closed form solutions for single circular loads at midslab, a corner, and the edge. Pickett and Ray (*8*) made the Westergaard solutions much more useful for pavement design and analysis with their development of influence charts. These charts also permitted the use of multiple wheel loads rather than just one circular loaded area and were used as a part of both PCA and U.S. Army Corp of Engineers methods for concrete pavement thickness determination.

With the advent of electronic computers, discrete element analysis was used by W. R. Hudson to represent the behavior of concrete slabs as discontinuous orthotropic plates (*29*). Subsequently. finite element solutions for the Westergaard pavement idealization were introduced by Tabatabaie and Barenberg (ILLISLAB) in 1977 (*30*) and by Tayabji and Colley (J-Slab) in 1981 (*31*). Variations are in use today; e.g., an updated version of ILLISLAB is included in the AASHTO MEPDG (*4*).

MATERIALS CHARACTERIZATION

An important aspect of the development of M-E design procedures has been the evolution of procedures to define requisite properties of materials. The international conferences referred to earlier have expedited the development of improved materials characterization for all materials used in the pavement structures, including untreated fine-grained soils and granular materials, treated materials (with lime, portland cement, and asphalt as admixture), portland cement concrete (PCC), and AC.

A number of the analysis procedures summarized in Table 1 for asphalt pavements as well as those noted for concrete pavements are based on the assumption of linear response (either elastic or viscoelastic). The majority of materials used in pavement structures do not satisfy such an assumption. Accordingly, ad hoc simplifications of response have been used for many of these materials. Terms such as stiffness, stiffness modulus, resistant modulus as well as modulus of elasticity. These moduli are used to determine stresses, strains, and deflections within the pavement structure. Results of such computations then permit determination of estimates of the various forms of distress which influence pavement performance.

AC Mixes

The stiffness characteristics of asphalt mixes was defined before the AASHO Road Test by the Shell Investigations, e.g., van der Poel (*32*) and Heukelom and Klomp (*33*), as a function of both time of loading and time temperature, that is,

$$S_{\text{mix}} = \frac{\sigma}{\varepsilon}(t,T) \tag{1}$$

where

S_{mix} = mix stiffness;
σ, ε = axial stress and strain, respectively;

t = time of loading; and
T = temperature.

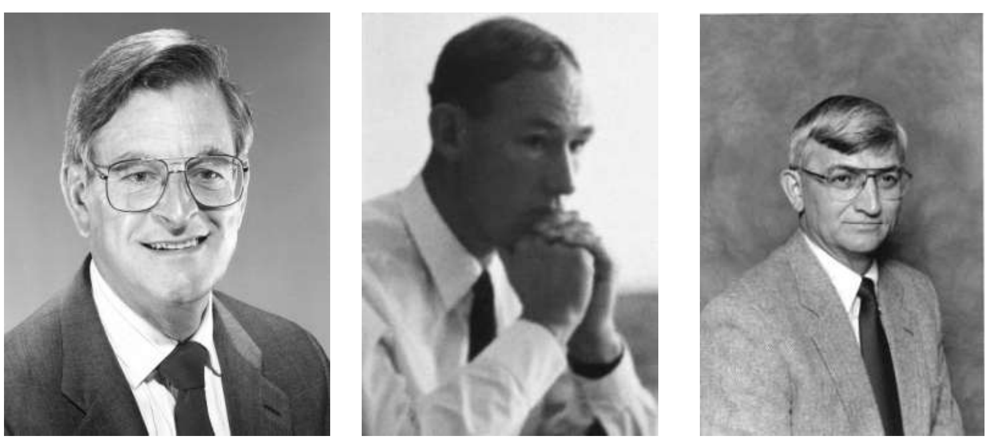
The use of viscoelastic concepts to define AC stiffness was described at both the 1962 and the 1967 conferences in research reported from Ohio State University [e.g., H. Papazian (*34*) and R. Baker (*35*)], UC Berkeley (*36*) as noted earlier, and the Asphalt Institute (AI) [B. Kallas (*37*)]. The use of the complex modulus and the interchangeability of time and temperature to represent AC stiffness was documented. Complex modulus as the measure of AC stiffness was utilized in 1982 by the AI in its M-E design procedure and is used in the AASHTO MEPDG (*4*).

Fine-Grained Soils

The stiffness characteristics of fine-grained soils are dependent on water content, dry density, soil structure, freeze–thaw action, applied stress, and soil moisture suction. Seed et al. (*3*), as noted earlier, presented a detailed study of the factors affecting the resilient (elastic) characteristics of compacted fine-grained soils. Others contributing to our current understanding of soil stiffness are J. K. Mitchell for soil structure (*38*), S. F. Brown for considerations of effective stress (*39*), and K. Sauer (*40*) and M. R. Thompson (*41*) for freeze–thaw effects. Photos of Mitchell, Brown, and Thompson are shown in Figure 10.

Untreated Granular Materials

Stiffness characteristics of untreated granular materials are dependent on stress state, dry density, degree of saturation, and aggregate gradation [particularly the proportion passing the No. 200 sieve (P_{200})]. Early contributors included Mitry in NCHRP-sponsored research (*42*), Barksdale (*43*), Dehlen (*44*), Hicks (*45*), and Thompson (*46*). Photos of Dehlen, Barksdale, and Hicks are shown in Figure 11. This work generally involved repeated load tests on triaxial test specimens with a range in confining and deviator stresses. These results contributed to the development of



(a) (b) (c)

FIGURE 10 Contributors to the stiffness (elastic) characterization of compacted, fine-grained soils: (*a*) K. Mitchell, UC Berkeley; (*b*) S. F. Brown, University of Nottingham; and (*c*) M. R. Thompson, University of Illinois.

(a) (b) (c)

FIGURE 11 Contributors to the stiffness (elastic) characterization of compacted untreated granular materials: (a) G. L. Dehlen, National Institute for Road Research, South Africa; (b) R. D. Barksdale, Georgia Institute of Technology; and (c) R. G. Hicks, UC Berkeley.

the current long-term pavement performance (LTPP) procedure (LTPP P46) for measuring the resilient moduli of granular materials.

Treated Soils and Granular Materials

Following the Road Test, considerable effort was devoted to defining the properties of treated soils and granular materials. These studies have included measures of the stiffness characteristics of PCC, asphalt emulsion, lime, and lime–fly ash-stabilized materials.

Mitchell (*47*) prepared a detailed summary of much of the work on PCC-stabilized bases completed up to 1976. This summary included results of studies by the PCA as well as other U.S., Australian, and the UK researchers. The 1982 International Asphalt Conference also included results of a number of studies from Belgium, France, the Netherlands, and South Africa (*48*). The 1977 and 1982 International Concrete Conferences (*9*) included considerable information on PCC-stabilized materials as well. All of these studies have provided excellent data for M-E design using PE-stabilized materials in flexible, composite, and rigid pavement structures.

Considerable data on the stiffness characteristics of asphalt emulsion bases were presented at both the 1967 and 1977 (*5*) International Asphalt Conferences. The work of Santucci (*49*) (Figure 12*a*), PE included in the 1977 conference was incorporated in the AI M-E design procedure introduced in 1982 (*50*).

The stiffness characteristics of lime and lime–fly ash–stabilized soils have been evaluated by a number of investigators (*51*). Thompson and his colleagues (*52*), for example, have presented useful stiffness data for pavement design and evaluation purposes that include the influence of curing and freeze–thaw cycles.

Fatigue and Fracture Characteristics

Asphalt Concretes

Considerable research has been devoted to the definition of the fatigue characteristics of AC mixes. Notable studies in this area, as stated earlier, include the work of Pell (*53*) presented at the First International Asphalt Conference. Also, Monismith and his colleagues (*54*), Coffman et al. (*55*), and others (*56–63*) presented extensive data in the period 1958 to 1972.

Test results from these efforts have been defined by relationships of the form:

$$N = A\left(\frac{1}{\varepsilon_t}\right)^b$$

$$N = C\left(\frac{1}{\sigma_t}\right)^d$$

$$WD = F\,N^z \qquad (2)$$

where

N = number of repetitions to failure,
ε_t = magnitude of the tensile strain repeatedly applied,
σ_t = magnitude of the tensile stress repeatedly applied,
WD = total dissipated energy to fatigue failure, and
A, b, C, d, F, z = experimentally determined coefficients.

Mix factors influencing fatigue response include AC stiffness, binder content, and degree of compaction (*64*). One general form of the equation representing strain as the damage determinant and including the influence of mix factors is as follows:

$$N \sim \left(\frac{1}{\varepsilon_t}\right)^a \left(\frac{1}{S_{\text{mix}}}\right)^b \left(\frac{V_{\text{asp}}}{V_{\text{asp}} + V_{\text{air}}}\right) \qquad (3)$$

where

S_{mix} = mix stiffness, dependent on time of loading and temperature and
V_{asp}, V_{air} = volume of asphalt and air respectively in compacted mix.

For cumulative loading with loads of different magnitudes (*65*) and loads applied at different temperatures (*66*), the linear summation of cycle ratios cumulative damage hypothesis appears suitable and is incorporated in the M-E design procedures that have evolved. Application of this concept to AC was first suggested by Peattie in 1961. This hypothesis is expressed as follows:

$$\sum_{i=1}^{n} \frac{n_i}{N_i} \le 1 \qquad (4)$$

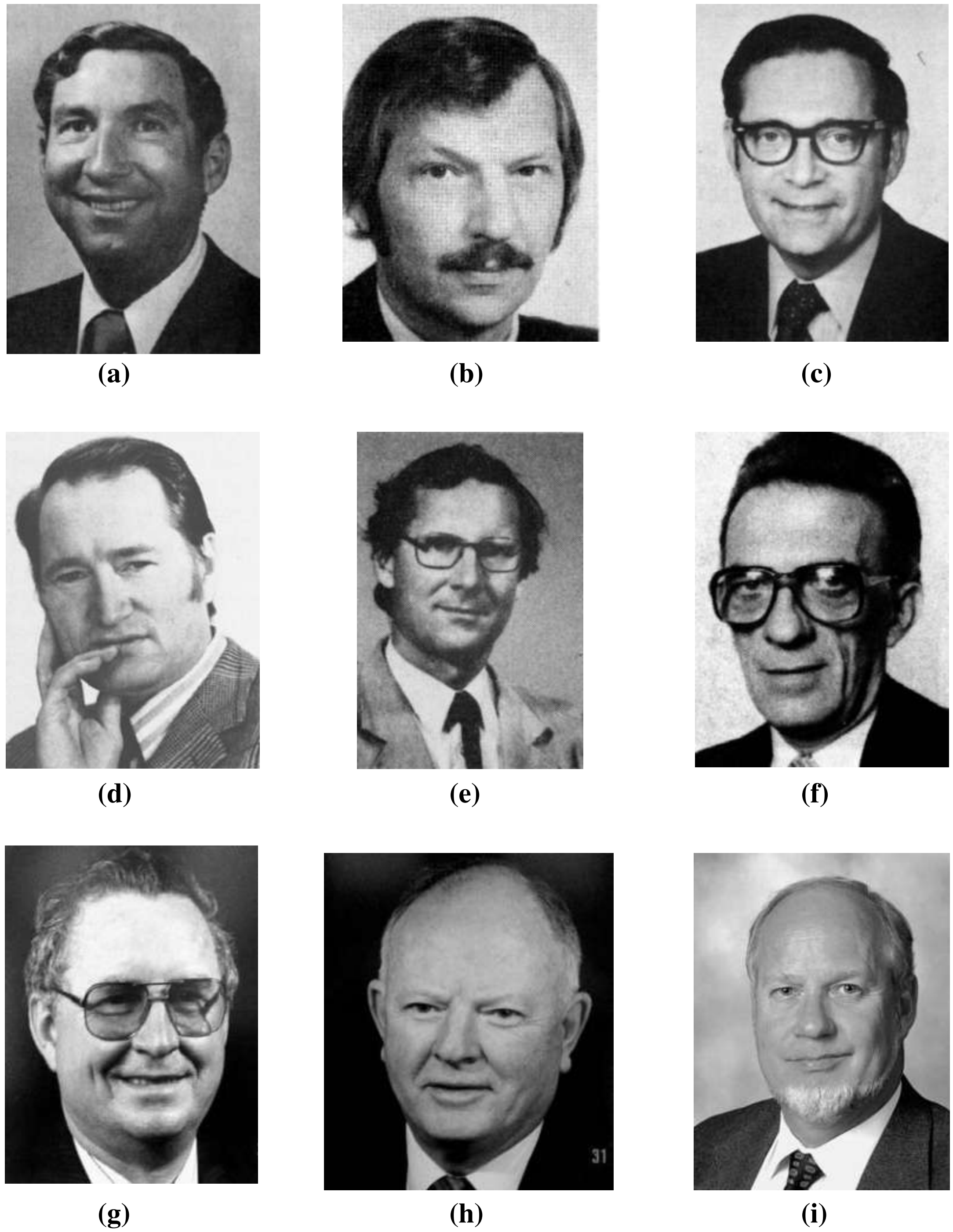


FIGURE 12 Engineers associated with development of some M-E design procedures for flexible pavements: (*a*) J. F. Shook, AI, AI procedure; (*b*) M. W. Witczak, AI, University of Maryland, AI procedure; (*c*) L. E. Santucci, Chevron Research, AI procedure; (*d*) W. "Pim" Visser, Shell procedure; (*e*) J. Bonnot, LCPC procedure, France; (*f*) J. Verstraeten, CRR, Belgium; (*g*) C. R. Freeme, National Institute for Transportation and Road Research (NITRR), South Africa; (*h*) N. Walker, NITRR, South Africa; and (*i*) H. Maree, NITRR, South Africa.

where

n_i = number of actual load repetitions of at strain level ε_i and
N_i = number of load repetitions to failure at strain level ε_i.

Because of concerns with fracture of AC due to development of high stresses at low temperatures as well as braking and sliding stresses resulting from tire–pavement contact forces, considerable attention has been devoted to AC fracture behavior (*67*). Like fatigue response, mix fracture characteristics are dependent on time of loading and temperature and mix volumetric characteristics. At low temperatures the fracture characteristics of the aggregate and asphalt stiffness have significant influence on mix fracture. These response characteristics have been well documented in SHRP studies (*68*).

Portland Cement Concrete

While data on the fatigue characteristics of PCC had been developed before the AASHO Road Test (*69*), studies of the Road Test performance data resulted in additional relationships (*70*). These relationships generally are expressed in terms of the ratio of the applied stress to the modulus of rupture of the concrete at some prescribed curing time, e.g., 28 days:

$$N \sim \left(\frac{\sigma_{appl}}{\sigma_{frac}}\right) \qquad (5)$$

where

σ_{appl} = applied stress and
σ_{frac} = modulus of rupture, e.g., 28 days.

Other Treated Pavement Materials

Materials in this category include cement- and lime-treated as well as asphalt emulsion-treated materials. In addition to the factors listed above for AC, e.g., $N \sim \varepsilon_t$ or σ_t, curing effects must be considered. Summaries of the results of research on these materials are included in the following references: cement-treated materials (*47*), lime-treated materials (*51*), and asphalt emulsion-treated materials (*49*).

MECHANISTIC–EMPRICAL PAVEMENT DESIGN AND ANALYSIS

As stated earlier, the Road Test provided a major impetus for the formation of the international conferences on asphalt and concrete pavements. These conferences, in turn, accelerated the development of M-E pavement design and analysis systems.

AC M-E Pavement Design

Currently, many M-E (analytically based) design procedures have been developed. Some, while not used, have served as the basis for other procedures. Several such procedures are briefly summarized in Table 2. All procedures idealize the pavement structure as a multilayer elastic or viscoelastic system using programs like those described in Table 1. These procedures received impetus from the 1962 conference (as noted earlier).

The procedures listed in Table 2 all consider the fatigue and rutting modes of distress in establishing pavement structures. Fatigue estimates are based on relationships of the form shown by Equation 3 and on subgrade strain or stress criteria. The linear sum of cycle ratios cumulative damage hypothesis is used in the majority of the methods to assess the effects of mixed traffic and environmental influences on fatigue cracking. Those procedures using a subgrade strain procedure incorporate a form of the linear sum of cycle ratios (based on compressive strain) for the same purpose. Photos of some people associated with the development of these methods are shown in Figure 12.

PCC Pavement Design

In the United States, PCA and FHWA supported the development of M-E design procedures for plain, jointed-concrete pavements. The results of the Road Test concrete pavement tests contributed to these developments as had the tests on the asphalt pavements, as noted earlier.

In 1966 the first M-E procedure was introduced by the PCA (*7*). This was developed by P. Fordyce (Figure 13) and R. Packard for plain, jointed, undoweled pavements. Design charts for stress determinations were developed for single- and tandem-axle loads using the influence charts developed by Pickett and Ray (*8*). The results permitted thickness selection based on considerations of cumulative damage in fatigue (on the basis of stress as compared with the use of strain for asphalt pavements as described earlier) using the linear sum of cycle ratios concept.

The FHWA supported the development of an M-E design procedure using the finite element analysis developed by Tabatabie and Barenberg. In this design methodology, developed by M. Darter (Figure 13) and E. Barenberg (*10*), both undoweled and doweled joints were considered.

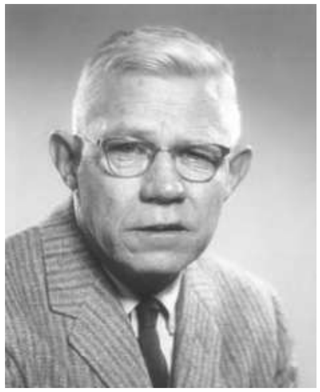
Subsequently the PCA introduced a new design methodology in 1984 (*84*), also based on a finite element analysis procedure. The method not only considered fatigue but also included provision for considerations of pumping. Also, the methodology permitted consideration for the use of either doweled or undoweled joints and tied concrete or conventional AC shoulders.

NONDESTRUCTIVE TESTING

While the Benkelman beam was developed for use at the AASHO Road Test (1952), the experience at the AASHO Road Test led to its widespread use subsequently for overlay design purposes for both asphalt and concrete pavements. The International Asphalt Paving Conferences of 1967 and 1972, for example, continued development in this area. The Benkelman beam spurred the development of mechanized versions like the LaCroix deflectograph (France) and traveling deflectometer (Hveem, California). The devices are shown in Figure 14.

TABLE 2 Examples of Analytically Based Design Procedures for Asphalt Pavements

Organization	**Pavement Representation**	**Distress Modes**	**Environmental Effects**	**Pavement Materials**	**Design Format**
Shell International Petroleum Co., Ltd., London, England (*71*)	Multilayer elastic solid	Fatigue in treated layers; rutting: − Subgrade strain and − Estimate in asphalt bound layer	Temperature	AC, untreated aggregate, cement-stabilized aggregate	Design charts; computer program BISAR for analysis
NCHRP Project 1-10B Procedure (AASHTO) (*19*)	Multilayer elastic solid	Fatigue in treated layers; rutting	Temperature	AC, asphalt-stabilized bases, untreated aggregates	Design charts; computer program (MTC093)
AI, Lexington, Ky. (MS-1, MS-11, MS-23) (*73*,*74*)	Multilayer elastic solid	Fatigue in asphalt treated layers; rutting: subgrade strain	Temperature, freezing and thawing	AC, asphalt emulsion, treated bases, untreated aggregate	Design charts; computer program DAMA
Laboratoire Central de Ponts et Chaussées (LCPC) (*75,76*)	Multilayer elastic solid	Fatigue in treated layers; rutting	Temperature	AC, asphalt-treated bases, cement-stabilized aggregates, untreated aggregates	Catalogue of designs; computer program (ELIZE) for analysis
Centre de Recherches Routieres, Belgium (*77*)	Multilayer elastic solid	Fatigue in treated layers; rutting	Temperature	AC, asphalt-stabilized bases, untreated aggregates	Design charts; computer program (MTC093)
NITRR, South Africa (*78–80*)	Multilayer elastic solid	Fatigue in treated layers; rutting: − Subgrade strain and − Shear in granular layers	Temperature	Gap-graded asphalt mix, AC, cement-stabilized aggregate, untreated aggregate	Catalogue of designs; computer program
NCHRP Project 1-26 Procedure (AASHTO) (*81*)	Finite element idealization; multilayer elastic solid	Fatigue in treated layers; rutting: subgrade strain	Temperature	AC, untreated aggregates	ILLI-PAVE; elastic layer programs (ELSYM)
FHWA, U.S. DOT (*82*)	Multilayer elastic or viscoelastic solid	Fatigue in treated layers; rutting: − Estimate at surface and − Serviceability (as measured by PSI)	Temperature	AC, cement-stabilized aggregate, untreated aggregate, sulphur-treated materials	Computer program: VESYS
University of Nottingham, U.K. (*83*)	Multilayer elastic solid	Fatigue in treated layers; rutting: subgrade strain	Temperature	Continuous or gap-graded asphalt mixes of known volumetrics on standard UK materials	Design charts; computer program (ANPAD) for analysis and design



(a)

(b)

FIGURE 13 Engineers associated with development of M-E design procedures for rigid pavements: (*a*) P. Fordyce, PCA, and (*b*) M. I. Darter, University of Illinois.

The Shell researchers used vibratory testing equipment at the Road Test (Nijboer) to measure in situ material response (e.g., moduli). Following the Road Test equipment such as the Dynafleet (Texas, F. Scrivener), vibrators to measure wave propagation (Transport and Road Research Laboratory) and eventually the falling weight deflector (FWD) (France, Denmark) were developed.

Pavement profile equipment was introduced for use at the Road Test (CHLOE Profilometer) and served as an important measure to quantify the present serviceability index (PSI) (Carey and Eirich) to define pavement performance. This equipment stimulated research on pavement smoothness and roughness that resulted in equipment such as the GMR Profilometer (1962 International Asphalt Conference) as well as other equipment worldwide.

OVERLAY PAVEMENT DESIGN

Nondestructive deflection test equipment played a major role in the development of overlay design methods for asphalt pavements. Notable among the methods (*86*) are those developed by the Asphalt Institute (Benkelman beam measured deflections) (*87*), the TRRL procedure developed by N. W. Lister (Figure 15) (Benkelman beam and TRRL deflectograph deflections) (*88*), and the State of California procedure (Benkelman beam and traveling deflectometer deflections) (*89*). The work of Lister is particularly noteworthy in this regard in that he introduced the concept of probability of achieving a given life in the overlay with thickness requirements based on probabilities of 50% and 90% of achieving the design life.

The vibratory equipment introduced by the Shell investigators (van der Poel and Nijboer) (*90*) to measure dynamic pavement deflections was later extended to wave propagation measurements by Heukelomp and Klomp (*91*) and Jones and Thrower of the TRL (*92*). The work by Jones and Thrower is particularly important in that they used different vibratory equipment over a range in frequencies to measure waves of different types (compression, shear, Rayleigh, and Love waves) and developed methodology to determine which type of wave was being measured. This, in turn, permitted estimates of both shear (*G*) and elastic (*E*) moduli of the

(a)

(b)

(c)

FIGURE 14 Nondestructive pavement test equipment: (a) Shell heavy vibrator; (b) LeCroix deflectograph, France; and (c) traveling deflectometer, California.

various layers of a pavement system. With such developments, the groundwork for M-E overlay design as a follow on to the procedures developed for new pavements as described earlier was established.

PAVEMENT MANAGEMENT SYSTEMS

In the period 1966 to 1970 Fred Finn and W. R. Hudson served as investigators for NCHRP Project 1-10, Translate AASHO Road Test Findings—Basic Properties of Pavement Components. During the conduct of the project, the concept of treating pavements as systems emerged and led, in turn, to the development of pavement management systems. Fred Finn worked with Roger LeClerc of the Washington State Highway Department to develop the first pavement management system for a highway department in the United States in 1972 (*93*).

At about the same, Ralph Haas began the development of the pavement management concept in Canada. Collectively these three, Fred Finn, W. R. Hudson, and R. Haas (*95*), are credited with the start of PMSs in North America, while R. LeClerc should be recognized for his foresight in bringing forth the first state highway PMS (*94*). Photos of Hudson, Haas, and LeClerc are shown in Figure 16. It is important to reiterate, however, that the AASHO Road Test, followed by research on the NCHRP, provided a major impetus to pavement management system development. Moreover, FHWA, recognizing the importance of this research, commissioned TRB to conduct two workshops in 1980 (*96*) to hasten PMS implementation at the state level.

PAVEMENT RESEARCH NOT DONE

At the conclusion of the AASHO Road Test, two NCHRP Reports, Numbers 2 and 2A, *Guidelines for Satellite Studies,* were prepared P. Eirich and W. R. Hudson. The purpose of the reports was to establish a series of test roads around the United States to study the effects of different environments, soil types, and a range of pavement materials on pavement performance. Regrettably, this program was not instituted until the start of LTPP about 40 years later.

(a) **(b)**

FIGURE 15 Engineers associated with overlay design: (*a*) N. W. Lister, TRRL, United Kingdom, and (*b*) A. J. G. Klomp, Shell, Netherlands.

(a) (b) (c)

FIGURE 16 Contributors to the development of PMSs: (*a*) W. R. Hudson, University of Texas, Austin; (*b*) R. C. G. Haas, University of Waterloo, Canada; and (*c*) R. LeClerc, Washington State Highway Deptartment.

A second area of research not conducted concerned road roughness and its impact on pavement performance. Following the Road Test, roughness measurements concentrated on ride quality from the driver stand point (PSI, as noted earlier). While this is an extremely important development, road damage based on smoothness–roughness measurements was not considered. Moreover, truck–pavement interaction studies to minimize pavement, truck, and goods damage was delayed for about 40 years, as well.

Following the Road Test, rutting was considered in flexible pavement design through the control, initially, of elastic vertical compressive strain at the subgrade surface with the intention of controlling contributions of unbound materials to surface rutting. Moreover, the M-E design procedures concentrated on AC fatigue. Until the Shell researchers introduced a procedure for rutting estimations in the AC layer(s) in 1977, research in this area was limited. While the Shell approach suggested that different mixes could be evaluated for their contribution to pavement surface rutting, it was not until the start of the first SHRP program almost 40 years after the Road Test, that the concept of relating AC mix design (based on rutting and fatigue considerations) and pavement design was introduced.

NOTE

1. Although the work of the Chevron researches never appeared in the published literature, it is important to recognize their significant contribution since they presented the first computer solution for a five-layer system (CHEV5L) in 1963 (*17*).

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APPENDIX A: NATIONAL ADVISORY COMMITTEE

This committee was appointed by HRB to advise the board and its project staff in relation to administrative and technical matters concerning the AASHO Road Test.

K. B. Woods, *Chairman,* Head, School of Civil Engineering, and Director, Joint Highway Research Project, Purdue University
W. A. Bugge, *Vice Chairman,* Director, Washington Department of Highways

W. F. Abercrombie,[1] Engineer of Materials and Tests, Georgia State Highway Department
R. R. Bartelsmeyer, Chairman, AASHO Committee on Highway Transport, and Chief Highway Engineer, Illinois Division of Highways; Chairman, Highway Research Board[2]
W. G. Burket, tire industry; Chairman, Technical Advisory Committee, Rubber Manufacturers Association;[3] and Manager, Truck Tire Engineering, Goodyear Tire and Rubber Company
D. K. Chacey, Director of Transportation Engineering, Office of the Chief of Transportation, Department of the Army Transportation Corps
W. E. Chastain, Sr., Engineer of Physical Research, Illinois Division of Highways
T. F. Creedon,*[8] Highway Engineering Advisor, Automobile Manufacturers Association
George Egan,* Chief Engineer, Western Highway Institute
R. E. Fadum, Head, Civil Engineering Department, North Carolina State College
E. A. Finney, Director, Research Laboratory, Michigan State Highway Department
C. E. Fritts, Vice President for Engineering, Automotive Safety Foundation
Sidney Goldin, Petroleum Industry; General Manager, Head Office Marketing, Shell Oil Company
W. D. Hart,[5] Transportation Economist, National Highway Users Conference
E. H. Holmes, Assistant Commissioner for Research, Bureau of Public Roads
J. B. Hulse, Managing Director, Truck Trailer Manufacturers Association
F. N. Hveem, Materials and Research Engineer, California Division of Highways
J. O. Izatt,* petroleum Industry; Asphalt Paving Technologist, Products Application Department, Shell Oil Company
A. E. Johnson, Executive Secretary, American Association of State Highway Officials
M. S. Kersten, Professor of Civil Engineering, University of Minnesota
John H. King,* Manager, Motor Truck Division, Automobile Manufacturers Association
George Langsner, Chairman, AASHO Committee on Design,[6] and Assistant State Highway Engineer, California Division of Highways
R. A. Lill,[7] Chief, Highway Engineering, American Trucking Associations
R. E. Livingston, Planning and Research Engineer, Colorado Department of Highways
L. C. Lundstrom, Former Chairman, Automobile Manufacturers Association Committee for Cooperation with AASHO Road Test, and Director, General Motors Proving Ground
B. W. Marsh, Director, Traffic Engineering and Safety Department, American Automobile Association
G. W. McAlpin,[9] Assistant Deputy Chief Engineer (Research), New York State Department of Public Works
R. A. Moyer, Professor of Highway Transportation Engineering and Research Engineer, Institute of Transportation and Traffic Engineering, University of California
R. L. Peyton, Assistant State Highway Engineer, Kansas Sate Highway Commission
K. M. Richards, Manager, Field Services Department, Automobile Manufacturers Association
C. F. Rogers,* Special Assistant, Office of Research, Bureau of Public Roads
T. E. Shelburne, Director, Highway Investigation and Research, Virginia Department of Highways
H. M. Straub,[4] tire industry, Manager, Tire Construction and Design, B. F. Goodrich Company
H. O. Thompson, Testing Engineer, Mississippi State Highway Department
R. G. Winslow,* Highway Engineer, Automotive Safety Foundation
J. C. Womack, President, American Association of State Highway Officials,[10] and State Highway Engineer and Chief of Division of Highways, California Division of Highways

The following persons served on the National Advisory Committee during the years indicated in the same capacity as the current member bearing the same footnote indicator:

[1] J. L. Land, Chief Engineer, Bureau of Materials and Tests, Alabama State Highway Department (1956)
[2] C. H. Scholer (1958); H. E. Davis (1959); Pyke Johnson (1960); W. A. Bugge (1961)—Chairman, HRB
[3] G. M. Sprowls (1956); C. R. Case (1957); W. C. Johnson (1958); L. Marick (1959); H. M. Straub (1960)
[4] L. Marick (1960)
[5] R. E. Jorgensen, Engineering Counsel, National Highway Users Conference (1956–1961)
[6] J. C. Young (1956); C. A. Weber (1957–1959); J. C. Womack (1960)
[7] H. A. Mike Flanakin, Highway Engineer, American Trucking Associations (1956–1957)
[8] I. E. Johnson, Manager, Chrysler Corporation Proving Ground (1956–1960)
[9] L. K. Murphy, Construction Engineer, Primary Highways, Maine State Highway Commission (1956–1959)
[10] C. R. McMillan (1958); D. H. Stevens (1960); D. H. Bray (1961)

A. A. Anderson, Chief Highway Consultant, Portland Cement Association (1956–1960)
Hugh Barnes, Assistant Vice President, Portland Cement Association (Resigned March 1961)
Douglas McHenry,* Portland Cement Association (1956)
Earl J. Felt,* Portland Cement Association (1957–1960)
B. E. Colley,* Portland Cement Association (Resigned March 1961)
H. F. Clemmer, Consultant, District of Columbia Department of Highways and Traffic (1956–1961)
R. D. Johnson,* Assistant Engineering Counsel, National Highway Users Conference (1958–1961)
A. S. Wellborn, Chief Engineer, Asphalt Institute (1956—resigned March 1961)
J. M. Griffith,* Engineer of Research, Asphalt Institute (1956—resigned March 1961)
Rex M. Whitton, First Vice Chairman (1956–1961); Chief Engineer, Missouri State Highway Department, resigned March 1961 to become FHWA Administrator
W. C. Williams, State Highway Engineer, Oregon State Highway Commission (1956–1961)

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* Alternate

A Historical Look at Interstate Highway System Pavements in the North Central Region

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The construction of the Interstate Highway System (IHS), one of the largest single public works projects in history, represents a significant milestone in the development of highway transportation in the United States. Initiated in 1956 with the passage of the Federal-Aid Highway Act, the IHS serves as a critical link in the nation's economy and today is the workhorse of the nation's highway system.

As the nation commemorates the 50-year anniversary of the signing of the 1956 Federal-Aid Highway Act effectively launching the construction of the IHS, it is appropriate to take a look back at pavement design practices used in constructing the Interstate system and how those practices have evolved during that time period. This paper focuses on the pavements constructed on the Interstate system in the North Central region of the United States and first presents a general background on the development of the Interstate system and then reviews the evolution of the concrete pavement and the hot-mix asphalt (HMA) pavement design practices used in the North Central region during the construction of the Interstate system. This is followed by a brief summary of the AASHO Road Test, which was located in the North Central region and served as the basis for the design of much of the Interstate system.

INTRODUCTION

General Background

The construction of the IHS, one of the largest single public works projects in history, represents a significant milestone in the development of highway transportation in the United States. Initiated in 1956 with the passage of the Federal-Aid Highway Act, the IHS serves as a critical link in the nation's economy and today is the workhorse of the nation's highway system, representing a little more than 1% of the nation's road mileage but carrying more than 24% of the traffic, including 41% of the commercial trucks (*1*).

The IHS can actually trace its origins to the late 1930s, when the Bureau of Public Roads (BPR, forerunner to FHWA) produced a two-part report, *Toll Roads and Free Roads*, on the feasibility of constructing a series of interregional highways (*2*). Later, in 1944, Herbert Fairbank, BPR's Information Division chief, prepared a report, *Interregional Highways*, which recommended an interregional highway system of 39,000 mi that was designed to accommodate traffic 20 years from the date of construction (*2*). At that time, basic design standards were also prepared as a first step toward the development of uniform, high-type roadway facilities (*3*).

The 1944 Federal-Aid Highway Act provided the framework for highway improvements in the postwar years and specifically authorized the designation of a 40,000-mi National System of Interstate Highways. Although the 1944 act thus authorized an Interstate system, it included no special provisions to give the Interstate highways a priority based on their national importance, nor did it authorize any special funding for construction of the system (*2*).

Limited progress was made on the construction of the Interstate System until the passage of the Federal-Aid Highway Act of 1956. This landmark highway bill called for the construction of a 41,000-mi IHS to be completed by 1975 at an estimated cost of $27 billion. Major features of the highway bill included a new pay-as-you-go financing plan, 90/10 federal and state cost sharing on new construction, and the adoption of elevated design standards: fully controlled access; 12-ft travel lanes; 10-ft outer shoulders; design speeds of 50, 60, and 70 mph for mountainous, rolling, and flat terrain conditions, respectively; separated traffic lanes with variable median widths; and design adequacy for projected 1975 traffic (a requirement that was changed to a 20-year minimum design period by 1963 legislation) (*4*). An amendment in 1966 legislation mandated at least four lanes of traffic (*4*).

The National System of Interstate and Defense Highways that emerged from the 1956 Highway Act owes much to President Dwight D. Eisenhower, who fought for the 1956 legislation and is commonly recognized as a pioneering visionary of the modern highway facility. Eisenhower understood the importance of an effective highway network, having participated in the U.S. Army's transcontinental convoy from Washington, D.C., to San Francisco, California, in 1919 and having traveled over portions of Germany's Autobahn during and after World War II (*5*). In recognition of the contributions of Eisenhower, in 1990 the Interstate system was renamed the Dwight D. Eisenhower System of Interstate and Defense Highways.

Clearly, the construction of the IHS represents a significant achievement. Although its social impacts have been debated over the years, the Interstate system is lauded for its many benefits and contributions. These benefits run the gamut from economic and safety aspects to mobility and accessibility. In addition, it is estimated that the nation, as a whole, has reaped a gain of at least $6 for every $1 invested in the construction of the Interstate system (*6*). More information on the benefits of the IHS is provided elsewhere (*6, 7*).

Detailed background information on the evolution of the Interstate system is found in several FHWA reports (*4, 7*), in an American Public Works Association publication (*8*), and in a series of articles by Weingroff (*2*, *9*, *5*, *10*, *11*). In addition, state perspectives on the construction of the Interstate system are provided in several AASHTO publications (*12*, *13*).

Interstate Pavement Design

Little guidance was initially given to the design of the pavements that would comprise the IHS. Most of the early Interstate pavements were designed to accommodate 1975 traffic levels but, as previously mentioned, this changed to a fixed 20-year design period in 1963. One of the early requirements was that the pavement design must be "soundly and justifiably arrived at and be adequate to support anticipated traffic loads" (*4*). A significant majority of the original Interstate system was constructed with portland cement concrete (PCC) (*14*), although HMA pavements [including composite HMA–portland cement concrete (PCC) sections] would later assume a substantial share of the market as the original Interstate pavements required rehabilitation or reconstruction.

As the nation commemorates the 50-year anniversary of the signing of the 1956 Federal-Aid Highway Act effectively launching the construction of the IHS, it is appropriate to take a look back at pavement design practices used in constructing the IHS and how those practices have evolved during that time period. This paper focuses on the pavements constructed on the Interstate system in the North Central region of the United States, and first begins with a summary of the conditions in the North Central region, followed by a review of the concrete pavement and the HMA pavement design practices used during the construction of the Interstate system. Finally, a brief summary of the AASHO Road Test is provided, which was located in the North Central region and served as the basis for the design of much of the IHS.

NORTH CENTRAL REGION

As defined by the FHWA's Long-Term Pavement Performance program, a 20-year study of in-service pavements across North America, 13 states comprise the North Central region of the United States, as shown in Figure 1. These states are broadly classified in the wet–freeze climatic zone, and overall have over 13,000 centerline miles of Interstate pavement, roughly 30% of the entire U.S. Interstate mileage. Table 1 lists the Interstate mileage by states in the North Central region.

Table 2 summarizes the condition [in terms of the International Roughness Index (IRI) and the Present Serviceability Index (PSI)] of the Interstate pavements in the North Central region, as reported for 2003 (*15*). This table indicates that the overall performance of the current system is very good, with 89% of the Interstate highways having PSI values greater than 2.8 and 25% greater than 3.9. A PSI value of 2.8 is in the range commonly used to trigger pavement rehabilitation.

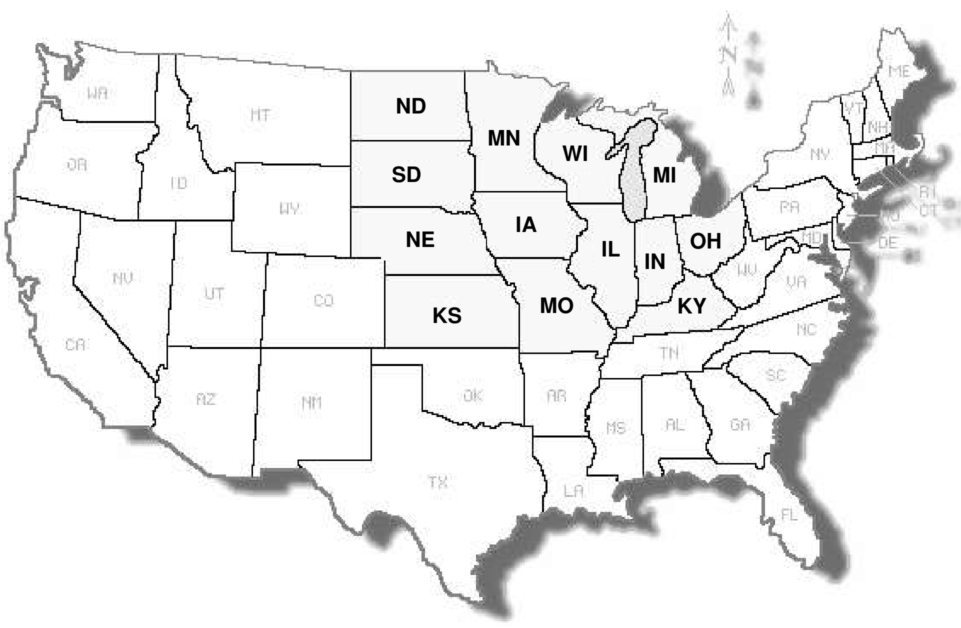


FIGURE 1 States in North Central region.

TABLE 1 Interstate System Mileage in the North Central Region

State	Mileage	State	Mileage
Illinois	2,170	Missouri	1,181
Indiana	1,169	Nebraska	482
Iowa	782	North Dakota	572
Kansas	874	Ohio	1,574
Kentucky	763	South Dakota	679
Michigan	1,243	Wisconsin	745
Minnesota	912		
Total mileage = 13,146			

TABLE 2 Condition of Interstate Pavements in North Central Region (*15*)

IRI Range	Approximate PSI	Cumulative % Mileage Better Than
< 60	3.9	24.8
60–94	3.2	69.4
95–119	2.8	88.8
120–144	2.4	96.6
145–170	2.0	99.1
> 171	1.6	100.0

The North Central region boasts two of the first Interstate segments that were associated with the 1956 Federal-Aid Highway Act (*9*). The first contract to be awarded under the new legislation (August 2, 1956) was a portion of U.S. Route 66 in LaClede County, Missouri, later incorporated into I-44 (see Figure 2). The first project to be completed under the 1956 legislation (November 14, 1956) was a portion of U.S. Route 40 in Kansas, later incorporated into I-70 (see Figure 3).

Most of the original interstate pavement network in the North Central region was constructed with PCC, more so than any other region of the country. In fact, nine of the 13 North Central region states (Illinois, Indiana, Iowa, Missouri, Nebraska, North Dakota, Ohio, South Dakota, and Wisconsin) paved more than 90% of their original Interstate pavements with PCC (*14*). Later, many of these PCC pavements were overlaid with HMA to create a composite pavement structure. Within the past decade or more, states in the North Central region have used both PCC and HMA for new pavement construction.

FIGURE 2 Missouri claims the first Interstate contract awarded after passage of Federal-Aid Highway Act of 1956 (*9*).

FIGURE 3 Kansas claims the first Interstate highway project completed under the provisions of the Federal-Aid Highway Act of 1956 (*9*).

The following sections describe the evolution of the design of both PCC and HMA pavements in the North Central region.

PCC PAVEMENTS IN THE NORTH CENTRAL REGION

PCC Pavement Type

Three PCC pavement types are commonly used in highway construction: jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), and continuously reinforced concrete pavement (CRCP). JPCP is short-jointed pavement (transverse joints generally spaced less than about 16 ft apart) that contains no reinforcing steel distributed throughout the slab. JRCP employs longer joint spacings (typically about 30 to 40 ft) and contains steel reinforcement (welded wire fabric or deformed steel bars) distributed throughout the slab that is intended to hold tightly together any transverse cracks that may develop. CRCP has no regularly spaced transverse joints but contains a significant amount of longitudinal steel reinforcement that both influences the development of transverse cracks within an acceptable spacing (about 3 to 8 ft) and serves to hold them tightly together.

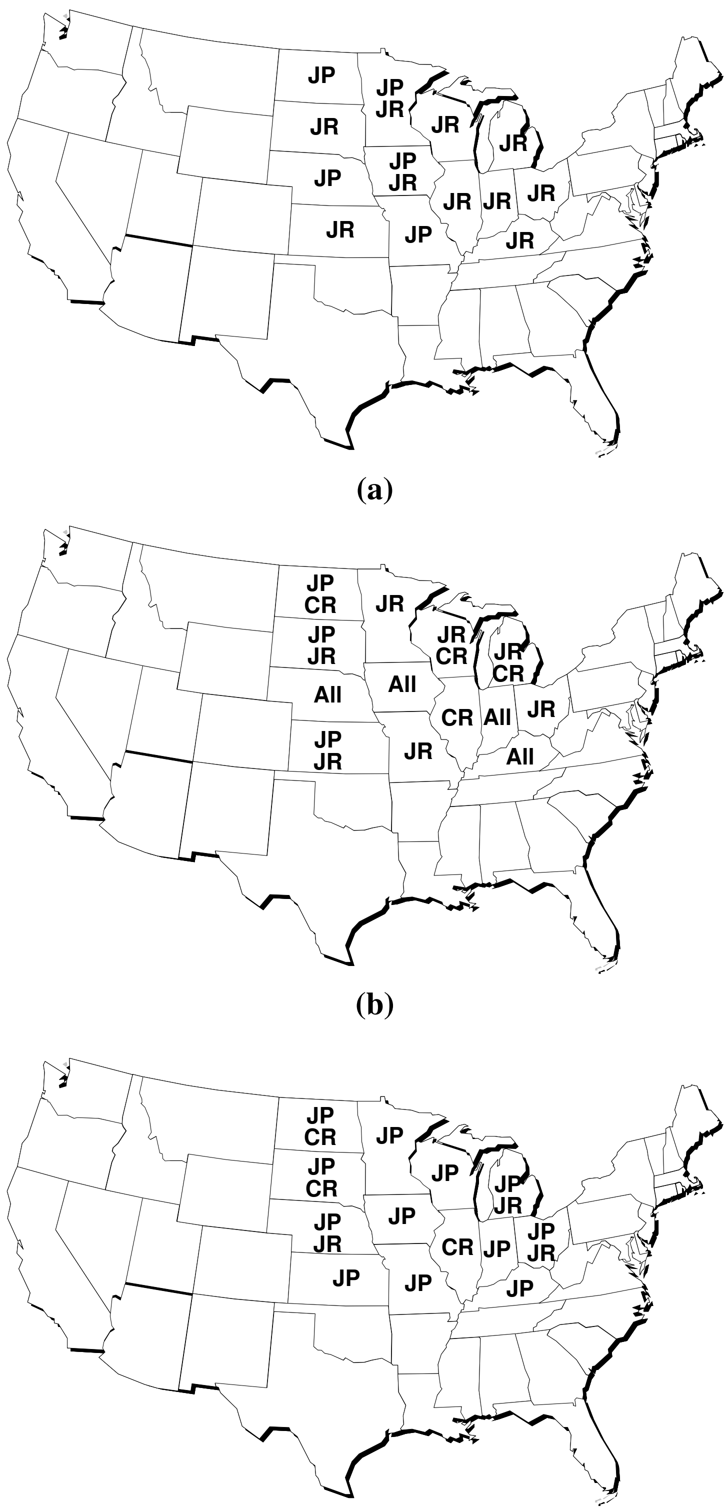
In the North Central region, all three PCC pavement types have been used. In the early days of the Interstate construction, JRCP designs were the pavement type of choice, as seen in Figure 4*a* (*16*). In 1975, many agencies had incorporated both JPCP and CRCP designs as PCC pavement design alternatives (Figure 4*b*) (*17*). However, by 1999, most agencies had moved to JPCP designs (see Figure 4*c*), which is by far the most common PCC pavement type currently being constructed (*18*). Three states (Illinois, North Dakota, and South Dakota) are shown in Figure 4 as currently constructing CRCP designs on their Interstate highways and have had excellent performance (*19*, *20*).

PCC Slab Thickness Design

In the early days of the Interstate program, PCC slab thickness "design" was often based on standard thicknesses that had been developed by each highway agency. These standard thicknesses were based on observed performance of PCC pavements within the states, and nominally accounted for traffic levels and subgrade support conditions. In 1958, slab thicknesses for PCC pavements constructed in the North Central region were commonly between 9 and 10 in. (Figure 5*a*).

Upon the completion of the AASHO Road Test in 1960, interim design guides were developed and issued by the AASHO Committee on Design in 1962 to be used by the states for a 1-year trial period (*21*). After the 1-year trial period, the AASHO Committee on Design did not consider it necessary to revise the interim guides (based on the largely positive feedback from the states), and the guides were retained as interim documents. As a reflection of the general satisfaction with the AASHO procedure, a review of design practices in 1975 indicated the following trends in the North Central region (*17*):

- Eight of the 13 states were using the 1972 AASHO Interim Design Guide,
- One state was using the Portland Cement Association (PCA) design procedure, and
- Four states were using their own procedures.

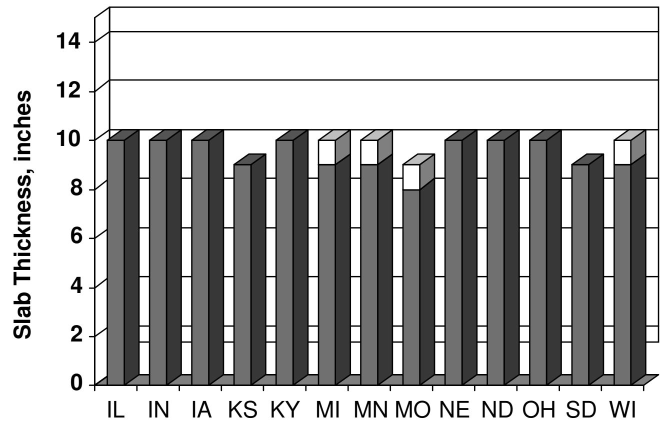


(a)

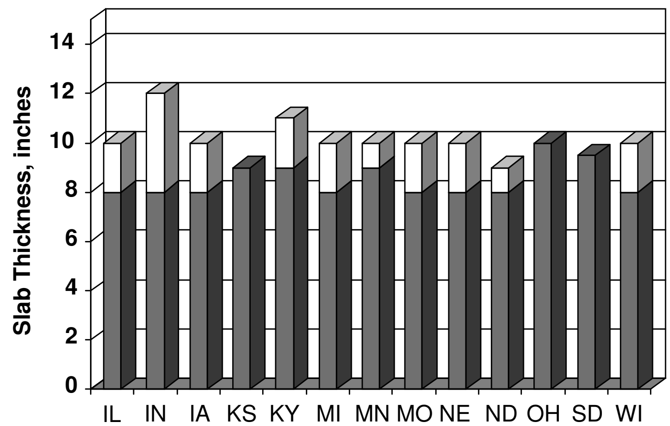
(b)

(c)

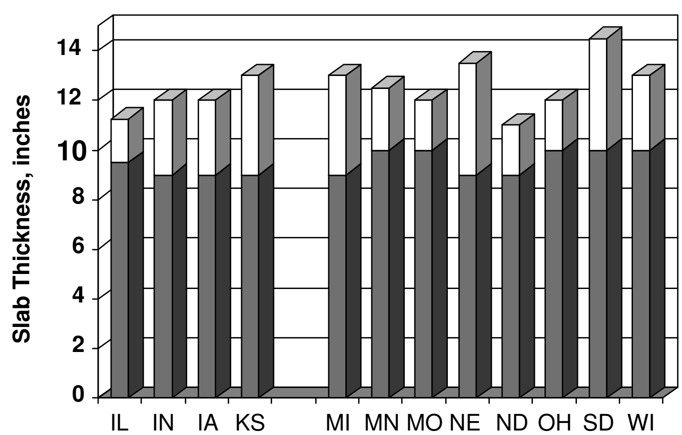
FIGURE 4 Primary PCC pavement type constructed in the North Central region in (*a*) 1958, (*b*) 1975, and (*c*) 1999 (JP = JPCP; JR = JRCP; CR = CRCP).



(a)



(b)



(c)

FIGURE 5 Typical range of PCC slab thicknesses on Interstate highways in North Central region in (*a*) 1958, (*b*) 1975, and (*c*) 1999.

As shown in Figure 5*b*, slab thicknesses in 1975 were observed to be in the 8- to 10-in. range, with the thinner 8-in. slabs associated with CRCP designs that had come into more widespread use during this period (many of the early CRCP designs built in the 1960s and 1970s were commonly constructed thinner than alternative jointed designs).

In 1986, AASHTO released a revised pavement design guide, still based on the results of the AASHO Road Test but containing a number of enhancements. The document was issued again in 1993 containing a revised overlay design procedure, but the design procedures for new pavements remained unchanged. According to a survey of pavement design practices in 1999, highway agencies in the North Central region were using the following design procedures (*18*):

- Eight states were using the 1986/1993 AASHTO Guide,
- Two states were using the 1972 AASHO Guide,
- One state was using the PCA procedure, and
- Two states were using their own (mechanistic-based) design procedures.

In 1999, typical slab thicknesses for PCC pavements constructed on the Interstate system in the North Central region had increased tremendously, now ranging from 9 in. to as much as 12 in. or more (see Figure 5*c*). Part of the reason for this increase is the inherent conservative nature of the AASHTO PCC pavement design procedure, which had come into more widespread use by the late 1990s. In addition, design traffic levels for many Interstate pavements had increased substantially, not only because of increased truck volumes but also because of longer initial design periods.

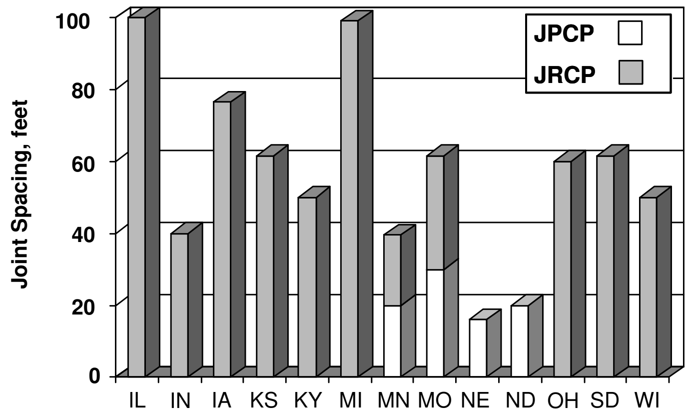
Many of the early PCC Interstate pavements in the North Central region achieved their design traffic levels after a fraction of their design life but continued to perform well. For example, many of the early PCC Interstate pavements in Illinois were designed for less than 5 million equivalent single-axle load (ESAL) applications but on average withstood about three times that designed traffic level before receiving rehabilitation (*22*).

Transverse Joint Spacing

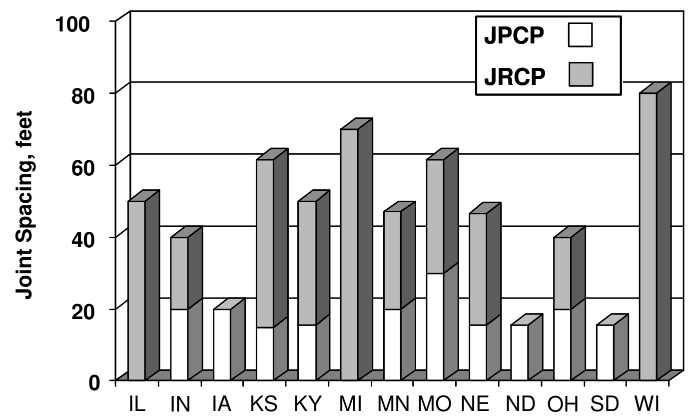
Transverse joint spacing is an important design feature that strongly affects jointed PCC pavement performance. Historical trends of transverse joint spacing in the North Central region are closely tied to the pavement type, as JRCP designs (which contain distributed steel intended to hold cracks tight) typically employed longer joint spacings (commonly 40 to 60 ft) whereas JPCP designs (which contain no distributed steel) employed shorter joint spacings (commonly 20 ft or less). These trends are illustrated in Figure 6, which shows the typical joint spacing for pavements constructed in 1958 (*16*), 1975 (*17*), and 1999 (*18*).

It is observed from Figure 6*a* that more states were constructing JRCP designs in 1958 and that many of these early JRCP designs used transverse joints spacings of 60 ft or more. The highway agencies constructing JPCP designs at that time were using joint spacings between about 15 and 30 ft. Although expansion joints had been used by many highway departments in the 1930s and 1940s, by the 1950s most agencies had adopted contraction joints and limited the use of expansion joints to bridges and other structures (*16*).

By 1975, highway agencies had reduced the joint spacings on their JRCP designs within the range of 40 to 60 ft (see Figure 6*b*). This was in recognition of the performance benefits of shorter joint spacings in terms of less slab movement and reduced joint openings. Figure 6*b* also



(a)



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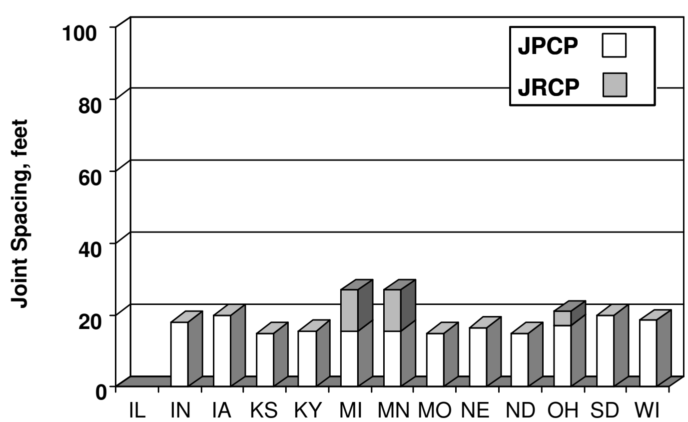

(c)

FIGURE 6 Transverse joint spacing used in North Central region: (*a*) 1958, (*b*) 1975, and (*c*) 1999.

shows the increased adoption of JPCP as a design alternative, with transverse joints commonly between 15 and 20 ft.

In 1999, most highway agencies had adopted JPCP designs and were using joint spacings between 15 and 20 ft (see Figure 6*c*). A few agencies continued to keep JRCP as a design alternative, but had adopted significantly shorter joint spacings (21 to 27 ft) than had previously been used.

Changes in Other PCC Design Features

There are a number of other design features that go into concrete pavement design, and it is interesting to trace how these have changed during the time of the Interstate pavement construction in the North Central region. Table 3 lists the general practices for each of these design features at various points in time during the Interstate pavement construction (based on information compiled from PCA (*16*), Nussbaum and Lokken (*17*), ERES (*23*), and ACPA (*18*).

Historically, most agencies have placed transverse contraction joints perpendicular to the centerline of the pavement and at uniform, fixed intervals (e.g., 15 ft apart). Beginning in the 1960s, some agencies began experimenting with joint orientation and layout, often placing the transverse joints at a counterclockwise skew to the centerline (typically offset by 1 to 2 ft per 12-ft wide lane) and at repeated variable spacings (such as 12, 15, 13, and 14 ft). By the 1970s and on into the 1980s and 1990s, this was a standard practice in many states in the North Central region. However, Table 3 shows that the highway agencies in the North Central region returned to the use of perpendicular and uniformly spaced joints for JPCP, based on several performance reports questioning the effectiveness of skewed and variable joint spacings (*24*, *25*).

At the beginning of the Interstate construction, JRCP designs in the North Central region were almost universally doweled, using either 1- or 1.25-in. diameter dowels. However, JPCP designs at this time were often undoweled, the belief being that aggregate interlock would provide effective load transfer because the shorter joint spacings would minimize joint opening.

TABLE 3 Historical Changes in Other PCC Pavement Design Features in North Central Region

Design Feature	1958 Practice	1975 Practice	1999 Practice
JPCP transverse joint orientation and layout	Perpendicular and uniform spacing	Skewed and variable spacing	Perpendicular and uniform spacing
Load transfer	1 to 1.25 in. dowels (JRCP designs only)	1.25 in. dowels (JRCP and JPCP)	1.5 in. dowels (JRCP and JPCP)
Joint sealing	Hot- and cold-poured sealants	Hot-poured sealants Unsealed joints (one state—experimental)	Hot-poured sealants Silicone salants Unsealed joints
Base type	Granular base	Granular and greater use of treated bases	Treated or permeable bases (medium- to high-traffic volumes)
Shoulder type	HMA	HMA and experimental PCC	PCC shoulders and/or widened PCC slabs

By the mid-1970s, the use of dowel bars in the North Central region was almost universally adopted for both JRCP and JPCP designs, and the use of 1.25-in. dowels was fairly standard for the 9- and 10-in. slabs commonly constructed during that era. The more recent trends have been to the use of larger diameter dowel bars (1.5-in. are now quite common). A number of research projects conducted in the period of the 1970s to the 1990s confirmed the need for dowel bars for most PCC pavements subjected to significant truck traffic (*24*, *25*, *26*, *27*).

Transverse joint sealing practices have evolved considerably in the North Central region. The early Interstate pavements were sealed after construction using either a hot- or cold-poured bituminous sealant. By the mid-1970s, agencies had adopted high-type hot-poured sealants almost exclusively, with one highway agency (Wisconsin) beginning to evaluate unsealed joints (*28*). In the late 1990s, the use of silicone sealants had become more common, and Wisconsin had adopted a no-seal policy for its new PCC pavements constructed on the Interstate. Moving into the 21st century, several highway agencies in the North Central region have constructed experimental pavements to assess the relative performance of sealed and unsealed joints.

Base types used in the early days of the Interstate pavements in the North Central region were almost universally granular. Performance studies conducted in the 1940s had clearly demonstrated the benefits of a granular base beneath PCC slabs to prevent pumping (*29*). Because of the experience of other highway agencies in the use of treated base courses, by the mid-1970s several highway agencies in the North Central region had either adopted or were evaluating the effectiveness of treated bases (e.g., aggregate or soil–aggregate mixtures treated with a small percentage of either cement or asphalt) in reducing joint faulting and improving overall performance [see, for example, studies by Minnesota (*30*), Michigan (*31*), and Ohio (*32*)]. In the past decade, many highway agencies in the North Central region have adopted the use of treated or permeable bases for Interstate pavements subjected to significant truck traffic volumes.

The 1956 Federal-Aid Highway Act called for a 10-ft paved outer shoulder, and the standard for the early Interstate pavements was one paved with either a HMA or a bituminous surface course. PCC shoulders were constructed as early as 1957 in Missouri and by several other North Central states (Illinois, Michigan, Nebraska, Iowa, Kentucky, Ohio, and North Dakota) on an experimental basis in the 1960s and 1970s (*33*, *34*). The use of widened PCC slabs (in which the outer slab is paved 1 to 3 ft wider than normal but the traffic lane is still striped at 12 ft) was first used in the late 1970s. Both of these design features are expected to provide performance enhancements by reducing critical edge stresses. Today, widened slabs and PCC shoulders are commonly used in the North Central region on Interstate pavements.

Other Significant Developments

Several other significant PCC pavement-related developments occurred within the North Central region, as described in the following sections.

Slipform Paving

The slipform paver completely revolutionized the PCC pavement construction industry and greatly increased overall paving productivity. The slipform paver traces its roots to the Iowa State Highway Commission, where highway engineers James Johnson and Bert Myers developed a prototype device in 1947 and first slipformed a roadway in 1949 (*35*). The first project was a county highway located near Primghar (in the northwest corner of the state) and was paved in two passes because the slipform paver was only 10 ft wide. From the 1950s into the 1960s, the slipform paver moved from an experimental process to a proven technology, even for the mesh-dowel pavements that were commonly constructed in the North Central states in the 1960s (*36*). By the early 1970s, virtually all highway agencies had adopted the slipform paver for Interstate pavement construction (*17*).

D-Cracking

D-cracking has been a nemesis of PCC pavements in the North Central region since it was first identified in Kansas in the 1930s. D-cracking is a form of deterioration in PCC pavements associated primarily with the use of coarse aggregates that disintegrate when they become saturated and subjected to repeated freeze–thaw cycles (*37*). D-cracking is visible as a series of fine cracks generally running parallel to joints, cracks, or free edges in the slab and may be accompanied by spalling, scaling, and staining.

The performance of many PCC Interstate pavements constructed in the North Central region has been hampered by the development of D-cracking. This led to major research efforts in the highway agencies to develop beneficiation techniques to upgrade coarse aggregates for use in pavement construction. One of the most effective methods found to reduce the susceptibility of coarse aggregates is to reduce the maximum size of the aggregate, and consequently many highway agencies adopted a maximum coarse aggregate size of 0.5 or 0.75 in. This practice, however, may lead to a significant increase in the paste requirement and may compromise the structural integrity of cracks and joints relying on aggregate interlock for load transfer (*38*). In recent years, highway agencies have developed extensive testing programs using a suite of tests to identify D-cracking susceptible aggregates.

Summary of PCC Pavement Design Trends in North Central Region

The following summarizes some of the key trends that have been observed in the design of PCC Interstate pavements in the North Central region:

- Use of longer designs lives (from a nominal 20-year design to 30 years or longer);
- Construction of thicker slabs (from 9- and 10-in. slabs to 11- to 14-in. slabs);
- Adoption of JPCP designs as the almost exclusive PCC pavement design type (with three states also constructing CRCP);
- Use of shorter JPCP joint spacings (typically 15-ft perpendicular joints that are uniformly spaced);
- Adoption of dowel bars for virtually all Interstate pavement construction;
- Increased use of treated and permeable bases for most Interstate pavements;
- Incorporation of edge support features, including tied PCC shoulders and widened slabs;
- Resurgence in production of durable PCC mixtures (including use of combined gradations, supplementary cementitious materials, and specialized admixtures);
- Adoption of initial PCC pavement smoothness specifications; and
- Reexamination of PCC pavement surface texturing techniques because of noise concerns (with some agencies adopting alternative methods, such as random transverse tining, longitudinal tining, and burlap drag).

HOT-MIX ASPHALT PAVEMENTS IN THE NORTH CENTRAL REGION

HMA pavements are layered systems that develop their load-carrying capacity through the load-distributing characteristics of the layered system. The conventional HMA pavement structure consists of an HMA surface course constructed on a granular base and granular subbase, but some agencies have also constructed full-depth designs consisting of one or more layers of HMA placed directly on the subgrade.

This section discusses the evolution of HMA pavement thickness design in the North Central region, and also describes the changes that have occurred in HMA mixture design and asphalt cement classification.

HMA Pavement Thickness Design

Two primary parameters used in the thickness design of HMA pavements are subgrade soil properties and design traffic characterization. Each of these parameters has changed significantly over the past 50 years.

Subgrade Characterization

The characteristics of the subgrade have long been recognized as a critical input to HMA pavement design. Nationwide, in the 1950s and 1960s, soil characteristics were evaluated using soil classification systems (gradation and Atterberg limits), California bearing ration (CBR), R-value, and the triaxial strength tests. In the North Central region, in the early 1950s, states were primarily using the CBR, with one state using the triaxial test, one state using a cone test, one using a shear test, and four relying upon soil classification methods (Figure 7). At this time, all states were routinely conducting soil classifications, using either the Highway Research Board (HRB) method, the Public Roads Administration method, or other pedological methods (*39*).

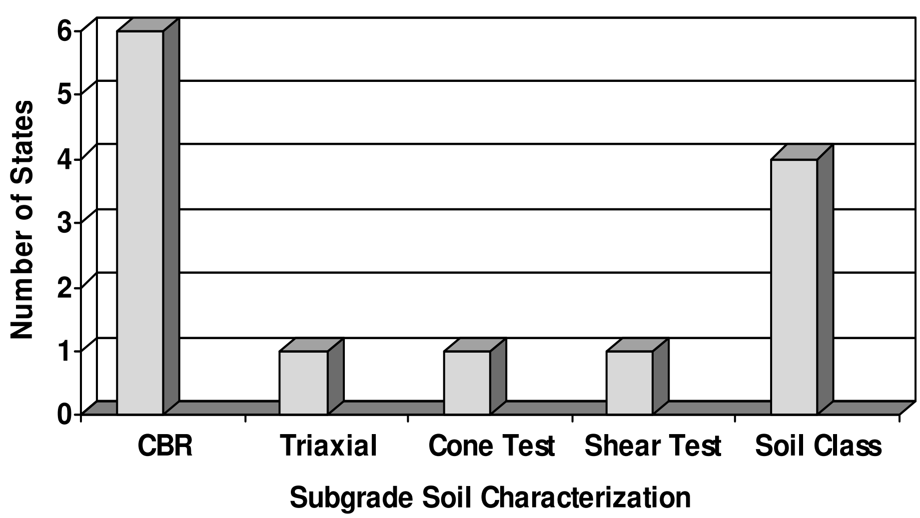

FIGURE 7 Subgrade soil characterization methods used by the North Central states in the mid-1950s (*39*).

In the early 1970s, little had changed in the way that these highway agencies characterized their subgrade soils. However, beginning with the release of the 1986 AASHTO Pavement Design Guide, many highway agencies began moving toward the use of resilient modulus to characterize their soils. Although resilient modulus had been used before this time, the inclusion of resilient modulus as an integral part of the 1986 AASHTO pavement design procedure strongly encouraged this move. Moreover, the resilient modulus was recognized as being an improved method of characterizing the subgrade soil.

Traffic

In the period of the late 1950s and early 1960s (before the results of the AASHO Road Test were published), state highway agencies evaluated traffic in a number of different ways, including

- Design, maximum, or equivalent wheel load;
- Traffic volume, in terms of either total traffic per day or number of commercial trucks per day; and
- Traffic index, an empirical approximation of the accumulated effect of wheel loads and repetitions.

With the release of the AASHO Interim Guide in 1962, a methodology was provided for converting mixed traffic into a single overall traffic loading indicator, the 18-kip ESAL. In this way, the damage caused by different axle types, configurations, and loadings are all expressed in terms of the equivalent damage caused by a standard 18-kip single-axle. The ESAL loadings are calculated using axle load equivalencies derived from the AASHO Road Test. By the early 1970s, eight of the 13 states in the North Central region had adopted the AASHO load equivalencies (*21*).

The ESAL factor continues to serve as the basis for many HMA pavement design procedures, but the recent movement toward M-E design procedures is advancing the use of load spectra data. Load spectra are the summation of axle load magnitudes and repetitions expected over the design life of the pavement; in the M-E design process, the load spectra data are used to compute the individual and accumulated effects of axle loads and types on pavement performance.

Design Procedures

A wide variety of design procedures have been used in the design of HMA pavements, including those based on theoretical and semitheoretical considerations, those based on empirical procedures, and those based on soil classifications. In the mid-1950s, a number of different HMA pavement design procedures were in use, as shown in Figure 8 (*40*). In the North Central region, CBR and Group Index (soil classification method) were commonly used.

After the release of the AASHO Interim Guide in 1962, highway agencies began using the new procedure and comparing and contrasting it with designs produced by their current design procedures. By the early 1970s, more than 60% of all state highway agencies had adopted the AASHO Interim Guide nationally, and within the North Central region nine of the 13 highway agencies were using it directly in their pavement design work (*21*). By the late 1990s,

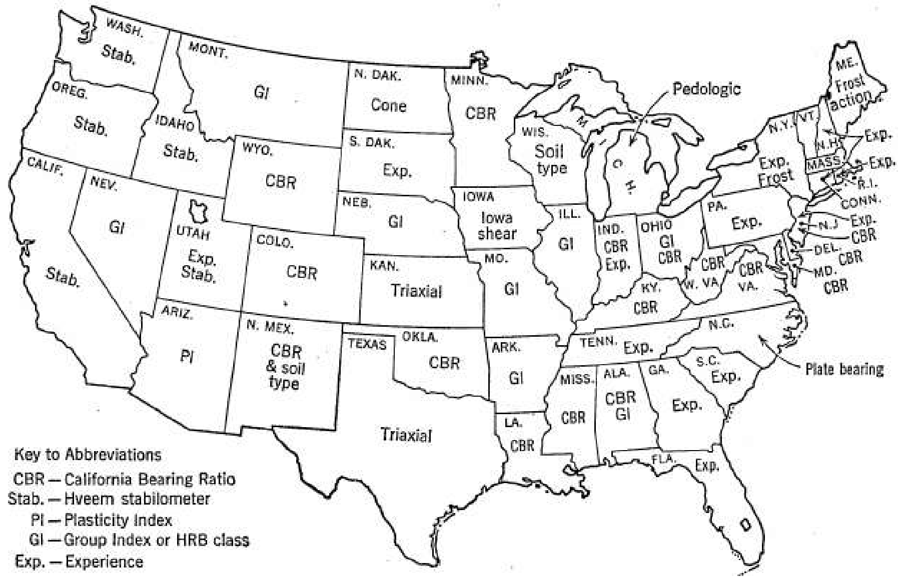

FIGURE 8 HMA pavement design procedures used in late 1950s (*40*).

more than half of the highway agencies were using the 1993 AASHTO Design Guide for HMA pavement design, including eight states in the North Central region (*23*).

HMA Mixture Design

The design of the HMA mixture is an important component of HMA pavement design. In the mid-1950s, a variety of mix design procedures were employed by the states in the North Central region, as shown in Figure 9. However, by the 1960s and into the 1970s, the Marshall mix design method was almost universally used in the North Central region (*41*, *42*).

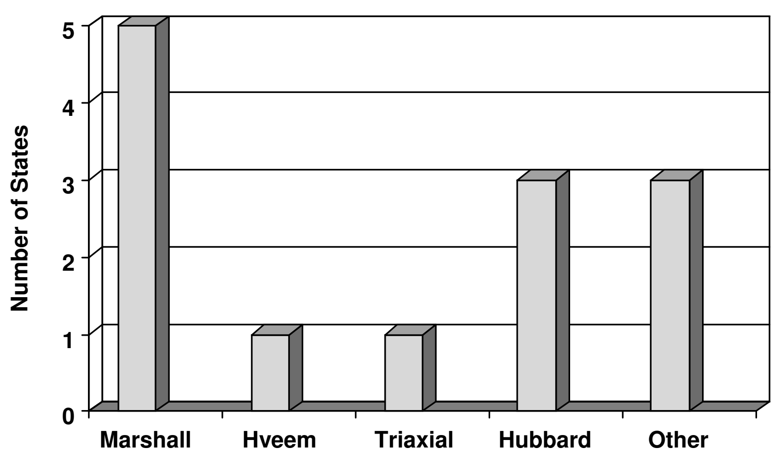

FIGURE 9 Method of mix design used by North Central region in the 1950s (*43*).

The earliest version of the Marshall mix design method was developed at the Mississippi Highway Department by Bruce Marshall about 1939. It was further developed by the U.S. Army Corps of Engineers Waterways Experiment Station in 1943 to design airfield pavements (*44*). Limiting criteria for items such as stability and flow were established, but these design criteria were later adjusted as tire pressures and weight of aircraft increased.

A goal of the laboratory compaction was to develop a simple preparation procedure that would help determine optimum asphalt content. The apparatus also needed to be portable enough to be taken into the field for quality control purposes. A 50-blow compaction effort on each side of the specimen was originally used. As heavier wheel loads and higher tire pressures were encountered the compaction effort was increased to 75 blows per side (*42*).

With the Marshall method, the optimum asphalt content was determined by maximum stability, a design air void content of 4%, and the maximum density. Many of the mixture designs developed by using the Marshall method were relatively fine gradations.

A survey of state highway practices in the 1984 indicated that the Marshall mix design was used by all but one of the states in the North Central Region (*45*). However, a standard test method was not universally used by all agencies.

Asphalt Cement Classification

In the 1950s and 1960s, asphalt cements were classified by the penetration test (ASTM D5). In the penetration test, a sample of asphalt cement at a temperature of 77°F is placed under a standard needle. The needle is loaded with a 100-g weight and is allowed to penetrate the asphalt cement sample for 5 s (*46*). A lower penetration value represents harder asphalt, whereas a higher penetration value represents softer asphalt. Into the mid-1980s, two states in the North Central Region continued to use the penetration test as the prime method of asphalt grading (*47*).

Along with the penetration test, the softening point test was conducted to characterize the temperature susceptibility of the asphalt cement. The softening point (ASTM D36) is simply the temperature at which asphalt cement cannot support the weight of a steel ball and starts flowing (*46*). The standard thin film oven test (ASTM D1754) and the rolling thin film oven (RTFO) test (ASTM D2872) were used to control the aging characteristics of the asphalt cement. In addition, a standard ductility test (ASTM D113) was used to measure that property of the asphalt cement. Example asphalt cement specifications are presented in elsewhere (*48*).

A major change in asphalt cement grading specifications was initiated in the early 1960s with the introduction of the viscosity grading system (*46*). The viscosity at 140°F was used as the standard value for grading, as this was considered the maximum pavement temperature for most of the United States. (*47*). Viscosity grading has the following advantages (*46*):

- Viscosity is a fundamental property;
- Viscosity is suitable for a wide range of environments (from 77°F to 140°F);
- There is no overlap in the grading;
- A number of test instruments are available;
- It is based on a maximum temperature appropriate for most of the United States; and
- Temperature susceptibility can be measured directly with the viscosity at three standard temperatures.

Into the mid-1980s, all but two of the states in the North Central region were using viscosity grading (*47*). ASTM D 3381 lists typical specifications for viscosity-graded asphalts.

Superpave System

In the late 1980s and early 1990s, SHRP designated a significant amount of research to the development of a more rational system to classify asphalt cements. The Superpave system includes various physical tests (*46*):

- RTFO (ASTM D2872 and AASHTO T240) to simulate hardening of the asphalt during production and construction.
- Pressure-aging vessel (ASTM D454 and ASTM D574) to simulate the asphalt binder aging that occurs during 5 to 10 years of in-service pavements.
- Dynamic shear rheometer (DSR) (AASHTO TP5) to characterize the viscous and elastic characteristics of asphalt binders at high- and intermediate-service temperatures. The DSR measures the complex shear modulus (G*) and the phase angle (δ) of asphalt binders at the desired temperature and frequency of loading, which are indicators of the stiffness and material elasticity, respectively. For a rut-resistant asphalt, the G* should be high and the phase angle low so that the asphalt cement is stiff and elastic. For a fatigue-resistant asphalt, the G*/ (sin δ) should be a minimum of 1.0 kPa for unaged asphalt cement and 2.2 kPa for aged material.
- Bending beam rheometer testing (ASTM D790) to evaluate the low-temperature properties of the asphalt cement. The test uses a transient creep load applied in the bending mode to load an asphalt beam specimen held at a constant low temperature.

The tests determine the upper and lower temperature performance grading classification for the asphalt cement. The high temperature indicates the temperature at which the asphalt cement will be stiff enough to withstand permanent deformation, whereas the low temperature indicates the low-temperature flexibility of the asphalt cement. The selection of the high-temperature grade is based on the high pavement temperature predicted over a 7-day period and the low temperature is selected on the basis of a low temperature recorded over a 20-year period (*49*). Typical asphalt binder specifications are presented by Roberts et al. (*46*).

Given the features of the Superpave system, it is now possible to design an asphalt cement for more specific conditions such as slow traffic and parking lots. By 1997, all states in the North Central region had adopted performance-graded asphalts.

Superpave mixture samples are prepared with the gyratory compactor and for weight–volume criteria developed by each agency. For gyratory mixture design, criteria have been developed for initial number of gyrations ($N_{initial}$) before it reaches 89% of maximum theoretical density (i.e., 11% air voids). The design number of gyrations is the number of gyrations (N_{design}) to compact the sample to 96% of maximum theoretical density (4% air voids). At the maximum number of gyrations ($N_{maximum}$), the minimum air voids should be 2% percent (*50*, *51*).

Air voids, voids in mineral aggregate, or film thickness are criteria used to design asphalt mixtures for both the Marshall and Superpave procedures. All HMA mixtures used on the Interstate pavements in the North Central region are now designed with the Superpave system.

Summary of HMA Pavement Design and Construction Trends in North Central Region

The following summarizes some of the key trends that have been observed in the design and construction of HMA Interstate pavements in the North Central region:

- The development and use of the Superpave mix design system including the performance-grading system for asphalt cements.
- The development of tests to measure the resilient modulus of the subgrade, granular base courses, and HMA surface courses for a flexible pavement. These values are used for input for mechanistic pavement designs.
- The development and calibration of mechanistic design procedures.
- The increased use of HMA pavement recycling. Mixtures with up to 40% recycled materials can be designed with the same criteria as for virgin mixtures using either the Marshall or Superpave procedures. HMA plants were modified so that the recycled materials are not subjected to a direct flame (*46*).
- The development and use of
  - Open-graded friction courses (*52*, *53*),
  - Stone matrix asphalt mixtures (*54*), and
  - Dense-graded large stone mixtures.
- In the late 1970s, the drum mixer was perfected and is used on most large projects.
- The use of storage silos to improve logistics at asphalt plants (*55*).
- The use of dust collectors to essentially eliminate particulate matter from plants:
  - Primary dry collector,
  - Wet scrubbers, and
  - Fabric filter (baghouse) system to remove dust and return it to aggregate input (*55*).
- The use of belt weighing systems for aggregate.
- On-paver developments include
  - Self-leveling and automatic ski controls and
  - Grade reference systems.
- Development of end result specifications using quality control/quality assurance concepts and test strips (*56*).
- Additives and modifiers for asphalt cements and HMA. These can be classified as
  - Extenders,
  - Modifiers,
  - Polymers,
  - Rubbers,
  - Plastics,
  - Fibers,
  - Oxidants,
  - Antioxidants,
  - Recycling agents (softening, rejuvenating), and
  - Antistripping agents [liquid, lime additives (*57*)].

THE AASHO ROAD TEST

One of the most significant pavement research projects conducted in the past 50 years was the AASHO Road Test, which was constructed in the North Central region. Conducted between 1958 and 1960 on the future alignment of I-80 near Ottawa, Illinois (Figure 10), the project not only evaluated the performance and behavior of pavement structures under a variety of axle loadings, but also investigated the performance of highway bridge structures under known loading conditions (*58*, *59*, *60*, *61*, *62*, *63*, *64*).

The pavement construction, pavement monitoring, and data analysis activities conducted at the Road Test established many standards and protocols and helped define design, construction, and evaluation practices. Even now, nearly five decades after the completion of the Road Test, the AASHO (and now AASHTO) design procedures continue to serve as the cornerstone for both PCC and HMA pavement design in the United States, as well as in other countries. Additionally, data from the Road Test are often used by researchers in a variety of ways unforeseen by the original AASHO Road Test developers (*65*).

Six pavement loops comprised the AASHO Road Test; five of them were exposed to traffic loadings (Figure 11). Each traffic loop consisted of two straight tangent sections and a superelevated turnaround connecting them at each end. Tangent lengths were 6,800 ft for Loops 3 through 6, 4,400 ft for Loop 2, and 2,000 ft in Loop 1. Two traffic lanes were constructed throughout the large loops, and each lane in the traffic loops were subjected to a specific truck of known axle type and axle load (*65*).

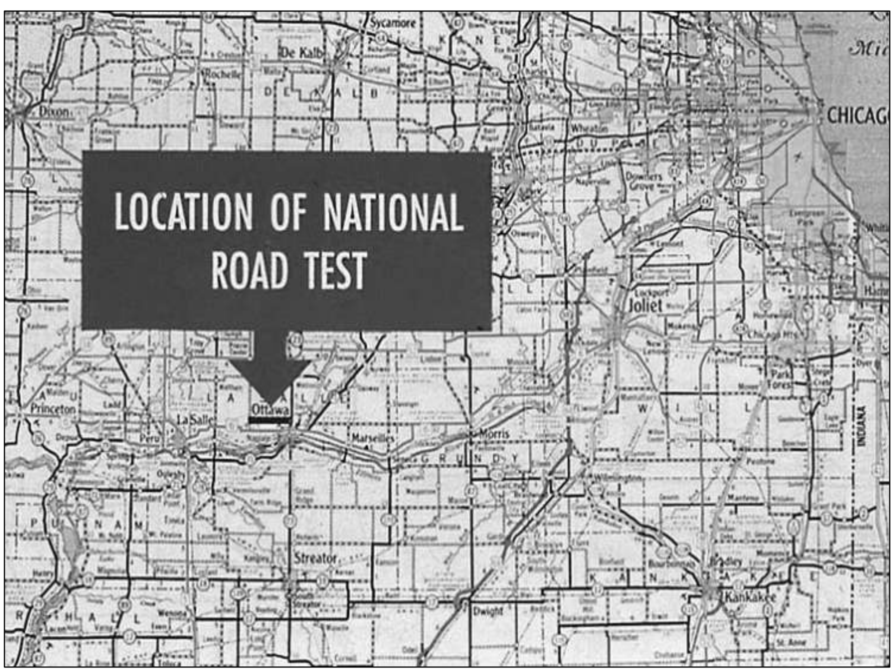

FIGURE 10 Location of AASHO Road Test.

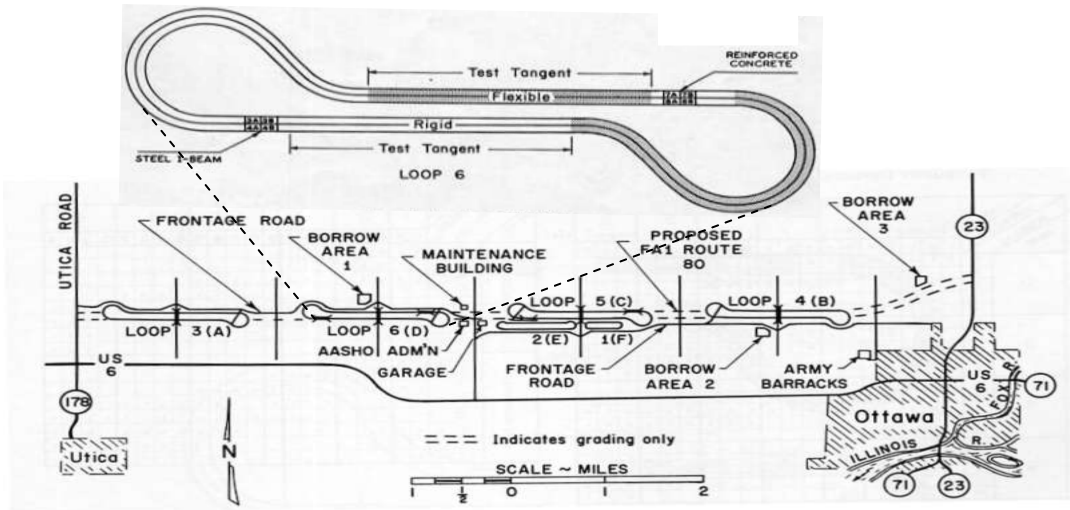

FIGURE 11 Layout of test loops at AASHO Road Test.

The northern tangent section and the eastern turnaround of each loop consisted of HMA test sections, whereas the southern tangent section and the western turnaround of each loop consisted of PCC test sections. A series of short test sections were constructed within each tangent (representing a factorial set of pavement thicknesses), separated by a short transition. Each test section was separated into two identical pavement sections by the centerline of the pavement (as specific truck axle type/axle load combinations operated in each lane). In total, there were 836 test sections constructed in all loops at the AASHO Road Test: 468 HMA pavement test sections and 368 PCC pavement test sections.

The performance of the pavement sections were closely monitored over the 2-year period. In general, measurements were made on those variables that had been demonstrated in previous research to be related to pavement performance. Routine performance measures included

- HMA pavements:
  - Longitudinal profile,
  - Roughness,
  - Cracking,
  - Patching, and
  - Rut depths and
- PCC pavements:
  - Longitudinal profile,
  - Cracking,
  - Patching,
  - Spalling, and
  - Joint/crack faulting.

The roughness and profile testing were conducted at 2-week intervals, whereas the other pavement condition data were collected as part of weekly surveys. Other measurements, such as surface deflections, surface strains, vertical subgrade pressures, and pavement temperature distributions, were obtained at various times during the test period.

Because one of the principal objectives of the Road Test was to determine relationships between pavement performance and design variables, it became necessary to develop a rational method of determining and expressing as a single number the performance of each pavement section at periodic intervals (*62*). To address this need, Bill Carey (chief engineer for research) and Paul Irick (chief, data processing and analysis) developed the concept of "pavement serviceability," which is founded on the principle that the primary function of a pavement is to serve the traveling public. As such, users of a pavement facility can provide their subjective opinions as to how well the pavement is meeting their needs, and it is assumed that the serviceability of a given highway may be expressed as the mean evaluation of all highway users (*66*). Serviceability was expressed on a scale of 0 to 5, with 0 representing a pavement that was impassible and 5 representing a perfectly smooth riding pavement.

Since it was impractical to convene a panel of raters regularly to provide their opinion of road serviceability on the more than 800 pavement sections at the Road Test, an alternate means of assessing pavement serviceability based on objective pavement condition measurements (roughness and distress) taken on each test section, was developed. With these objective measures and models developed from a panel review of pavements in Illinois, Minnesota, and Indiana, a PSI was computed for each section that served as an estimate of the mean panel serviceability rating. Figure 12 shows typical serviceability histories for selected pavement sections included in the Road Test.

Equations were developed to define the serviceability trends predicted from the number, type, and weight of axle loads relative to either the thickness index (now called the structural number) for HMA pavements or the slab thickness for PCC pavements. Figure 13 presents the graphical relationship that was developed for HMA pavements.

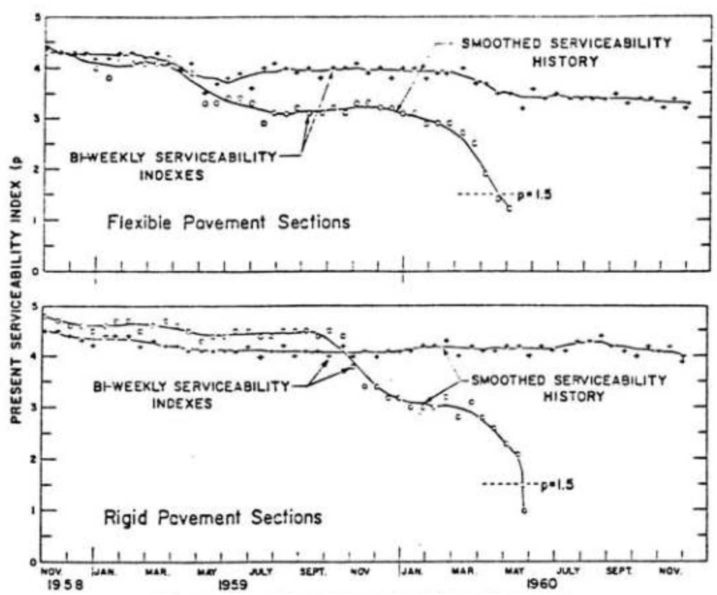

FIGURE 12 Typical serviceability histories for selected pavement test sections (*62*).

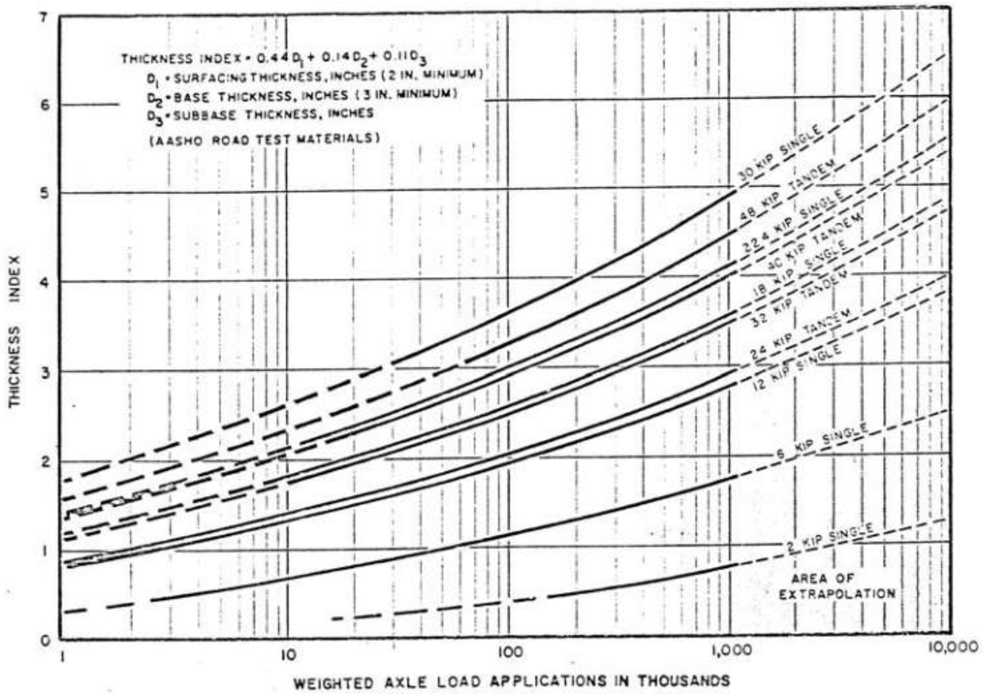

FIGURE 13 HMA pavement "thickness index" as a function of axle loadings for PSI of 2.5 (*60*).

As mentioned earlier, the results of the AASHO Road Test were used in the development of the interim pavement design procedures that were released to the states in 1962 for a 1-year trial usage. The design procedures were widely accepted by many highway agencies and were used to design many of the pavements constructed during the Interstate era. A portion of the original AASHO Road Test sections (Loop 1, the untrafficked loop) still remains today in the right-of-way of I-80 as a visible reminder of the significant contributions of the Road Test (see Figure 14).

SUMMARY

This paper describes the development of the IHS and the changes in the pavement design practices of the states in the North Central region, namely, Illinois, Indiana, Iowa, Kansas, Kentucky, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Ohio, South Dakota, and Wisconsin.

Most of the original Interstate pavement network in the North Central region was constructed with PCC, more so than any other region of the country. In fact, nine of the 13 North Central region states paved more than 90% of their original Interstate pavements with PCC (*14*). Later, many of these PCC pavements were rehabilitated with HMA overlays to create a composite pavement structure. Within the past decade or more, states in the North Central region have used both PCC and HMA for new pavement construction.

(a) (b)

FIGURE 14 Loop 1 of the AASHO Road Test in September 2005: (*a*) PCC pavement and (*b*) HMA pavement.

The PCC pavements in the North Central region have evolved from 9- to 10-in. JRCP designs in the 1950s to JPCP designs that today are constructed 11 in. thick or greater. CRCP designs are commonly constructed in three states. Dowel bars, treated and permeable bases, and PCC shoulders and widened slabs comprise other key elements of the modern JPCP design in the North Central region.

The thickness design of HMA pavements was originally based on soil classification or empirical soil tests. Traffic was evaluated with the maximum axle load, and now ESALs is used. Equivalent load factors developed from the results of the AASHO Road Test are used to calculate design ESALs for specific locations. Mix design procedures progressed from the Marshall procedures in the 1950s and 1960s to Superpave procedures by the late 1990s. The development of the Superpave performance-grading system for characterizing asphalt cements has been significant and makes it possible to specify asphalt cements for specific environmental and loading conditions. Various modifiers are also available to improve the characteristics of asphalt cement. Thickness design procedures are now based on mechanistic representation of the pavement layer system, and elastic-layered systems are used to simulate HMA pavement behavior. Tests are available to measure resilient modulus for specific conditions.

The AASHO Road Test was conducted in the North Central region and played an important role in the design of Interstate pavements. It was conducted between 1958 and 1960 on the future alignment of I-80 near Ottawa, Illinois, and contained 468 HMA and 368 PCC pavement test sections. The pavement construction, pavement monitoring, and data analysis activities conducted at the Road Test established many standards and protocols and helped define many pavement design, construction, and evaluation practices.

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Pavement Lessons from the 50-Year-Old Interstate Highway System

California, Oregon, and Washington

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Pavements constructed for the Interstate Highway System (IHS) in the West Coast states of California, Oregon, and Washington were examined to see what lessons could be learned about design and performance during the past 50 years. The focus was on new or reconstructed pavement structures.

The origin for the information in this paper was originally developed for and presented at the 85th Annual Meeting of the Transportation Research Board in Session 353, Pavement Lessons from 50-Year-Old Interstate System. Presentations were made by four regional groups:

- Western region,
- Southern region,
- North Central region, and
- North Atlantic region.

This paper is an attempt to summarize the information presented for the western regional states of California, Oregon, and Washington.

INTRODUCTION

Given the time since the start of initial construction of the Interstate highway pavements nearly 50 years ago until today, much can be learned about past practices. This paper attempts to summarize some of the available information spanning the 50-year period for three states in the Western United States. These three states were chosen simply to limit the amount of data gathering along with the assumption that any lessons learned would be largely reflective of a larger number of states.

The three West Coast states of California, Oregon, and Washington have a combined population of about 45 million, or 15% of the U.S. population (Table 1). The combined lane miles of Interstate system pavement for the three states are about 22,000 which represent approximately 10% of the total Interstate lane miles in the United States. The major Interstate routes in the three states are California I-5, I-8, I-10, I-15, I-40, and I-80; Oregon, I-5 and I-84; and Washington, I-5, I-82, and I-90.

Information provided in this paper spans the time from the 1950s to today. However, much of the Interstate system was complete by the 1960s. For example, Oregon reported that 77% of its Interstate system was complete by 1965 (Kramer, 2004). The percentage complete was likely not as high for California and Washington in 1965 but suggests that design and construction techniques largely developed in the 1940s and 1950s were applied to much of the Interstate system.

As an initial examination of these pavements, Table 2 shows the current International Roughness Index (IRI) for the three states and the United States (FHWA, 2004). The urban portions of the Interstates are in the poorest condition as defined by IRIs associated with the mediocre and poor categories (California, 65%, Oregon 36%, and Washington 22%). The mediocre and poor categories are those pavements that have IRIs greater than 1.9 m/km. [The IRI levels of mediocre and poor were, in part, chosen based on a prior user survey conducted in the Seattle area (Shafizadeh, 2002).] The U.S. average urban IRI reported for all states has about 27% falling into the mediocre and poor categories. The picture for rural Interstates is better (California 28%, Oregon 8%, and Washington 17%) with the U.S. average at 12%. Overall, the IRI of Interstate pavements for the three states span that of the U.S. averages. Later in this paper, more specific IRI will be presented that examines flexible and rigid pavements separately.

TABLE 1 General Statistics for California, Oregon, and Washington

State	Size (km^2)	Population	Interstate Lane Miles (lane km)
California	430,000	36.0 million	14,700 (23,800)
Oregon	258,000	3.6 million	3,100 (5,000)
Washington	187,000	6.2 million	3,900 (6,400)

Source: U.S. Census Bureau, July 1, 2005, and FHWA, *Highway Statistics*, 2004.

TABLE 2 Interstate IRI Percentages for California, Oregon, Washington, and the United States

State	Interstate IRI (m/km)				
	Very Good (<1.0)	Good (1.0–1.5)	Fair (1.5–1.9)	Mediocre (1.9–2.7)	Poor (>2.7)
California					
Urban	5	16	14	34	31
Rural	23	29	20	20	8
Oregon					
Urban	3	21	40	33	3
Rural	5	38	49	8	0
Washington					
Urban	28	37	13	14	8
Rural	24	44	15	16	1
United States					
Rural	18	36	19	19	8
Urban	29	44	15	10	2

Source: FHWA, *Highway Statistics*, 2004.

TRAFFIC

An examination the performance of Interstate pavements today is complicated by numerous factors such as the original designs, construction quality, maintenance, traffic, and climate—with much of this information not available or difficult to characterize. However, we will examine from a broad perspective the available information for some of these factors. Regarding traffic there is no evidence that the traffic levels in these three states are significantly different from the United States as a whole or between the three states. Table 3 shows for a sample of Interstate routes that the number of trucks per day varies widely but is largely reflective of urban versus rural locations. The number of trucks estimated per day result in a range of annual equivalent single-axle loads (ESALs) of about 3 million ESALs per year for the sample of rural locations to about 4 to 11 million ESALs per year for urban areas. This is a significant difference between locations but suggests that all of these Interstate routes have experienced significant truck traffic. Further, they have experienced far more ESALs than originally designed for since (*a*) the original design period was 20 years with many of these pavements still in service after 40 to 50 years, and (*b*) the number of trucks and their loadings were fewer 50 years ago. This suggests that the pavement design procedures used in the 1950s and 1960s were more conservative than originally intended.

PAST AND PRESENT DESIGN PRACTICES

Tables 4 through 13 are used to overview past and current design practices for flexible and rigid pavements.

TABLE 3 Snapshot of Traffic Levels for the Three States (2004 Data)

State	Route	Location (urban or rural)	ADT	Percent Trucks	Trucks per Day
California	I-5	Los Angeles (U)	226,000	13%	29,000
	I-80	Berkeley (U)	159,000	6%	10,000
Oregon	I-5	Portland (U)	136,000	9%	12,000
	I-84	Hood River (R)	26,000	21%	5,000
Washington	I-5	Seattle (U)	242,000	5%	12,000
	I-90	Snoqualmie Pass (R)	30,000	18%	5,000

ADT= average daily traffic

TABLE 4 Comparison of Current West Coast Flexible Design Practices

Features	California[1]	Oregon	Washington[2]
Thickness design	Gravel equivalent	AASHTO 93	AASHTO 93
Design process	Hveem equations	DARWin	DARWin/State computer software
Design period	20 or 40 years (40 years used if AADT >150,000)	20 years (30 years in grade-constrained areas)	50 years
Traffic	Traffic index = 9.0 $(ESAL/10^6)^{0.119}$	ESALs	ESALs
Subgrade strength design parameter	*R*-value	M_R (mostly from FWD deflections)	M_P (mostly from FWD deflections)

[1] Caltrans (2006); [2] WSDOT (2005); FWD = falling weight deflectometer

TABLE 5 Comparison of 1960s West Coast Flexible Design Practices

Feature	California	Oregon	Washington
Thickness design	Gravel equivalent	Gravel equivalent	Gravel equivalent
Design process	Hveem equations	Charts	Charts
Design period	20 years	20 years	20 years
Traffic	Traffic index = 9.0 $(ESAL/10^6)^{0.119}$	Traffic index	Traffic index
Subgrade strength design parameter	*R*-value	*R*-value	*R*-value

TABLE 6 Comparison of Current West Coast Rigid Design Practices

Feature	California[1]	Oregon	Washington[2]
Thickness design	State procedure but adopting M-E procedures	AASHTO 93	AASHTO 93
Design process	Design tables (Chapter 600 series)	DARWin	DARWin
Design period	20 yrs (if 20-year AADTT <150,000) 40 years (otherwise)	30 years	50 years
Type of PCC pavement	Primarily JPCP and CRCP to a lesser extent[1]	CRCP	CRCP
Transverse joints	For JPCP, straight joints either 12 or 15 ft with avg = 13.5 ft Use dowels	Not applicable	Straight joints with 15-ft spacing Use dowels
Traffic	Traffic index =9.0 $(ESAL/10^6)^{0.119}$	ESALs	ESALs
Base types	Mostly LCB and HMA	Stabilized base (generally HMA)	HMA
Subgrade strength design parameter	*R*-value	*k*-value (estimate from FWD data)	*k*-value

[1] Caltrans (2006); [2] WSDOT (2005)
AADT = annual average daily traffic; PCC = portland cement concrete; M-E = mechanistic–empirical; HMA = hot-mix asphalt; LCB = lean concrete base

TABLE 7 Comparison of 1960s West Coast Rigid Design Practices

Feature	California	Oregon	Washington
Thickness design	State process (modified PCA from 1967–1983)	State process	State process (combination of more than one)
Design process	Tables		
Design period	20 years	20 years	20 years
Type of PCC pavement	JPCP (No dowels)	JRCP (<1968) CRCP (>1963)	JPCP
Subgrade strength design parameter	*R*-value	*R*-value	*R*-value
Traffic	Traffic index = 9.0 $(ESAL/10^6)^{0.119}$	Traffic index	Traffic index
Base types	Cement-treated base	Crushed stone	HMA or crushed stone
Transverse joints	1950s: 15-ft joints, skew optional 1964: Skewed random spacing of 12, 13, 18, and 19 ft 1983: Skewed random spacing of 12, 15, 13, and 14 ft	Not applicable	1940s and 1950s: 90-degree 15-ft joints 1966: Random, skewed spacing ranging from 14 to 18 ft 1967–1992: Random spacing between 9 and 14 ft

TABLE 8 California Flexible Pavement As-Builts—1950s to Current

Feature	HMA Thickness (in.)	Subgrade
1950s		
Minimum HMA depth	Min. allowed: 1.2 Min. used: 3.0	Subgrade strength was characterized by *R*-value
Maximum HMA depth	Max. allowed: 24.0 Max. used: 12.0	
Current		
Minimum HMA depth	Min. allowed: 1.2 Min. used: 3.0	Subgrade strength was characterized by *R*-value
Maximum HMA depth	Max. allowed: 24.0 Max.used: 12.0	

TABLE 9 Oregon Flexible Pavement As-Builts—1950s to Current

Feature	HMA Thickness (in.)	Subgrade
1950s		
Minimum HMA depth	3.5	Characterized by *R*-value. 12–18 in. of select material was often specified over silty or clayey subgrades.
Maximum HMA depth	5.0	
Current		
Minimum HMA depth	12.0	6.0-in. aggregate base over 18.0-in. subbase
Maximum HMA depth	13.0	12.0-in. aggregate base

TABLE 10 Washington Flexible Pavement As-Builts—1950s to Current

Feature	HMA Thickness (in.)	Base/Subgrade
1950s		
Minimum HMA depth	3.0	Switched from CBR to *R*-value due to issues with clean sands and clayey gravel characterization about 1951 Some bases were CTB
Maximum HMA depth	6.0	
Current		
Minimum HMA depth	9.6	Use of *R*-value continued from early 1950s to 1990s. Then with AASHTO 86, switch to M_R was made.
Maximum HMA depth	10.8	

CTB = cement-treated base

TABLE 11 California Rigid Pavement As-Builts—1950s[1] to Current[2]

Traffic	PCC slab (in.)	Base (in.)	Other Factors
1950s			
Light traffic (design ESALs <2.5 million)	8.0	4.0 CTB	PCC not used if subgrade *R*<10; same for all climate regions
Heavy traffic (design ESALs >2.5 million)	9.0	4.0 CTB	
Current			
Lightest traffic (design ESALs <1,000,000 or TI<9)	8.4	4.2 HMA, LCB, or 12.0 granular base	Slab thicknesses for Inland Valley location, Type 2 subgrade (10≤*R*-value≤40)
Heaviest traffic (design ESALs >200 million or TI>17)	13.8	6.0 HMA or LCB	

[1] Harvey, et al. (2000); [2] Caltrans (2006)

TABLE 12 Oregon Rigid Pavement As-Builts—1950s to Current

Traffic	PCC slab (in.)	Base (in.)	Traffic
1950s			
Minimum rigid section	8.0	9.0 crushed stone	PCC primarily used for most new alignments starting in 1959
Maximum rigid section	8.0	12.0 crushed stone	
Current			
Minimum rigid section	11.0	4.0 HMA	LCCA used to determine pavement type
Maximum rigid section	13.0	4.0 HMA	

LCCA = life-cycle cost analysis

TABLE 13 Washington Rigid Pavement As-Builts—1950s to Current[1]

Traffic	PCC Slab (in.)	Base (in.)	Traffic
1950s			
Minimum rigid section	8.0	Crushed stone	PCC considered only when 10 year TI = 6.5 (or about 500,000 ESALs)
Maximum rigid section	9.0	Either HMA or crushed stone	
Current			
Minimum rigid section (design ESALs <25,000,000)	10.8	HMA	LCCA (mostly) used to determine pavement type Doweled transverse joints Reliability = 95% Thinner slabs allowed for ramps, rest areas, etc.
Maximum rigid section (design ESALs >50,000,000)	12.6	HMA	

[1] WSDOT (2005)

TABLE 14 Typical Rigid Pavement Designs [NCHRP 1-32 (1996)]

Feature	Slab Thickness (in.)		
Design ESALs	California	Oregon	Washington
2.5 million ESALs	8.4	8.0	8.0
7.5 million ESALs	9.0	8.5	8.5
30.0 million ESALs	10.2	11.0	10.5

FLEXIBLE DESIGN

Tables 4 and 5 provide a summary of current and past flexible pavement design practices for the three states. It is clear that the procedures developed by Hveem had a strong early influence on all three states. California, Oregon, and Washington, in effect, used the same (or similar) design methods (gravel equivalent), traffic characterization [traffic index (TI)], design period (20 years), and subgrade characterization (*R*-value). This changed over time for Oregon and Washington as they moved to AASHTO design procedures (the AASHTO 1986 Guide and subsequently the 1993 Guide). Further, Washington currently uses a structural design period of 50 years whereas California and Oregon use 30 to 40 years. The use of *R*-value has decreased with the advent of the FWD. Both Oregon and Washington currently make significant use of resilient modulus and backcalculation routines. California is moving in a similar direction (although that is not reflected in Table 4).

Hot-Mix Asphalt Thicknesses and Mixes

Tables 8 through 10 show typical ranges of HMA thicknesses used for Interstates in the 1950s to today. Although the range was large, it was common to have HMA thicknesses of about 3 to 6 in. Today, HMA thicknesses have approximately doubled—to about 10 to 12 in.

All three states use performance grade binders and Oregon and Washington have fully implemented the Superpave mix design process. Pierce et al (2004a) observed that HMA surfacing performance life for the Washington State Department of Transportation (WSDOT) increased about 14% (from 12.9 years to 14.7 years) over a 6-year period (1997 to 2003). This likely reflects changing construction practices more so than mix design procedures.

RIGID DESIGN

Tables 6 and 7 provide a summary of current and past rigid pavement design practices. Early on for Interstate design and construction, California and Washington adopted JPCP without the use of dowel bars. Oregon started with JRCP but began transitioning to CRCP by 1963 (Kramer, 2004). All three states made use of TI to quantify traffic for pavement design.

Portland Cement Concrete and Base Types

An early, major difference for the three states was in the selection of base types. California adopted CTB, and Oregon and Washington largely used either HMA or crushed stone bases. Washington did build a few sections of Interstate PCC pavements on CTB, but those were some of the earliest PCC pavements to fail and were subsequently removed and replaced or overlaid with HMA.

During the late 1930s and early 1940s, Francis Hveem of the California Highway Department (Caltrans) recognized that PCC faulting was a major contributor to PCC distress and recommended that CTB be used to combat this problem. The first CTB was placed on the California route system in 1937 with a HMA surfacing (Mahoney et al., 1991). The early CTBs performed well, except for shrinkage cracks in the HMA and eventually PCC surfaces. During the 1950s, the cement content of the CTBs was reduced to address the shrinkage cracking problem. However, over a longer span of time, the performance of CTB was not acceptable to Caltrans as the following quotation suggests (Caltrans, 1988): "A major change in that cement treated base, the traditional standard for PCCP (from 1950 to late 1970s), is no longer considered to be appropriate for portland cement concrete pavements because of its susceptibility to erosion."

Today, all three states have moved toward the use of HMA base for PPC pavements (PCCP). HMA base is exclusively used in Oregon and Washington. California allows either lean concrete base (LCB) or HMA. On the basis of recent construction trends, Caltrans uses HMA bases more than LCB largely because of speed of construction (communication with T. Hoover, Caltrans, October 5, 2006).

Transverse Joint Spacing

California and Washington both transitioned from straight (90-degree) transverse joints to a skewed, random spacing (skewed counter clockwise 2 ft in 12 ft). The difference between these two states was the spacing. Washington's joint spacing evolved over time. Starting in the 1940s and 1950s, WSDOT used a straight joint pattern with a 15-ft spacing without dowels. In 1966, this changed to the skewed pattern with a spacing ranging from 14 to 18 ft. This was quickly reduced in 1967 to a random spacing of 9, 10, 14, and 13 ft for an average spacing of 11.5 ft. By 1992, WSDOT began moving toward a straight 15-ft pattern with dowel bars (WSDOT, 1995 and WSDOT, 2005).

California also started with straight 15-ft transverse joints but changed to a skewed spacing. The initial skewed spacing was 13, 19, 18, and 12 ft for an average of 15.5 ft (Mahoney et al., 1991). Over time, many of the 19- and 18-ft slabs experienced midpanel transverse cracks. The spacing was eventually reduced to 12, 15, 13, and 14 ft for an average of 13.5 ft. Today, Caltrans continues to use a 12-, 15-, 13-, and 14-ft spacing but with a straight orientation and dowel bars (Caltrans, 2006).

PCC Slab Thicknesses

Tables 11 through 13 show typical ranges of PCC slab thicknesses used for Interstates in the 1950s to today. During the 1950s, all three states used slab thicknesses ranging from 8 to 9 in. Today, the range is more like 11 to 13 in. for higher ESAL levels. Based on slab designs obtained via Jiang (1996) and the NCHRP 1-32 report, Table 14 shows that there is little difference in slab thicknesses for the three states at the same ESAL levels; however, the data shown in the table were current as of 1996. State practices have continued to evolve since that time. Additionally, Washington has often added up to 1 in. of PCC to allow for studded tire wear.

Naturally, state practices evolved in ways that are difficult to capture in a short paper. For example, Caltrans rigid pavement thicknesses can be further described by at least four time periods (Harvey et al., 2000) plus current practice:

- 1952 to 1964: PCC slabs were 8 to 9 in. thick over a 4-in. CTB with increased strength. The transverse joints were straight with 15 ft spacing and no dowel bars.
- 1964 to 1967: PCC slabs remained 8 to 9 in. thick over a 4-in. CTB. The transverse joints were changed to the early, skewed pattern detailed above.
- 1967 to 1983: PCC slabs remained 8 to 9 in. thick but over a 6-in. CTB. The CTB strength was increased along with plant mixing.
- 1983 to 2005: PCC slabs ranged in thickness up to 14 in. CTB was eliminated but several base options were added including HMA, LCB, asphalt-treated permeable base, and cement-treated permeable base. The skewed, random transverse joint spacing was reduced.
- 2005 to 2006: PCC slabs range in thickness up to 13.8 in. Base types are mostly HMA or LCB (see Table 11). The thickest PCC slabs are designed for design ESAL levels exceeding 200 million. Transverse joints are straight, spaced 12 to 15 ft apart with dowel bars.
- Why did Caltrans change PCCP designs over time? A primary reason was faulting of the transverse joints and slab cracking. Changing the CTB specification, joint spacing, and slab thickness was tried in an attempt to improve PCCP performance. Hveem in 1949 concluded that 1.0-in. diameter dowels for transverse contraction joints were unacceptable because of poor performance due to corrosion, bent and broke bars, and “freezing” in the joint sockets (as reported by Harvey et al., 2000). He also noted that dowels did reduce faulting. That dowel experiment was done with PCC slab lengths that ranged from 20 to 60 ft—and Hveem observed that those slab lengths were excessively long and resulted in cracking. That work appears to have influenced various western states, including Washington, not to use dowels early on for Interstate pavement design and construction.

PERFORMANCE

Figures 1 through 6 are used to show basic pavement performance trends for Oregon and Washington. Similar data for California were not available at the time this paper was prepared.

Time Since Construction or Reconstruction

Figures 1 and 4 are used to overview the time since construction and reconstruction for Oregon and Washington.

Oregon data (Figure 1) show that the flexible pavements are a bit older than PCCP. Typically, the flexible pavements range in age from 30 to 50 years, with oldest being about 50 to 60 years with an average of about 40 years. The PCCP, on average, was constructed about 30 years ago, with the oldest being about 50 years. For both pavement types, few lane miles have been constructed during the past 10 years.

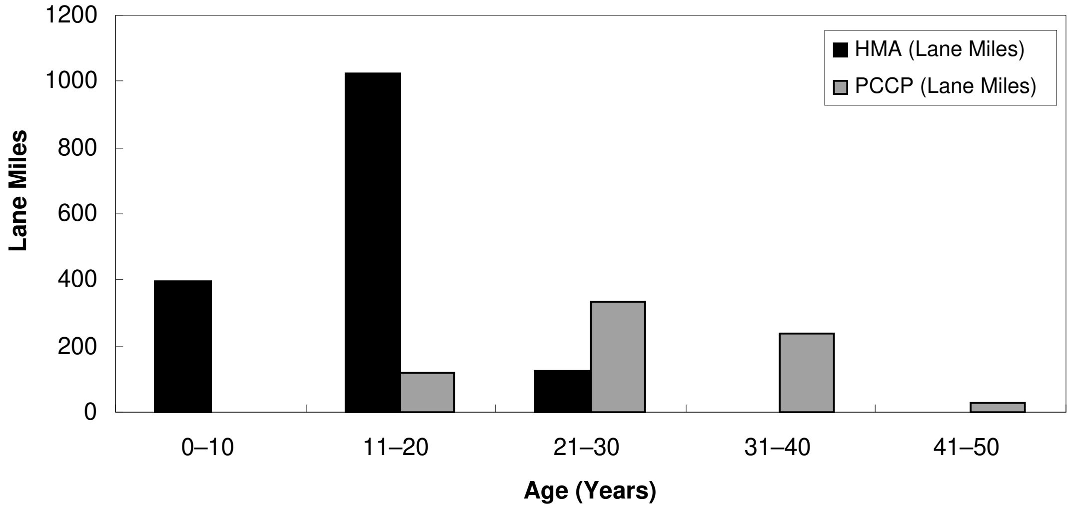

FIGURE 1 Oregon DOT Interstate pavements, time from initial construction or reconstruction.

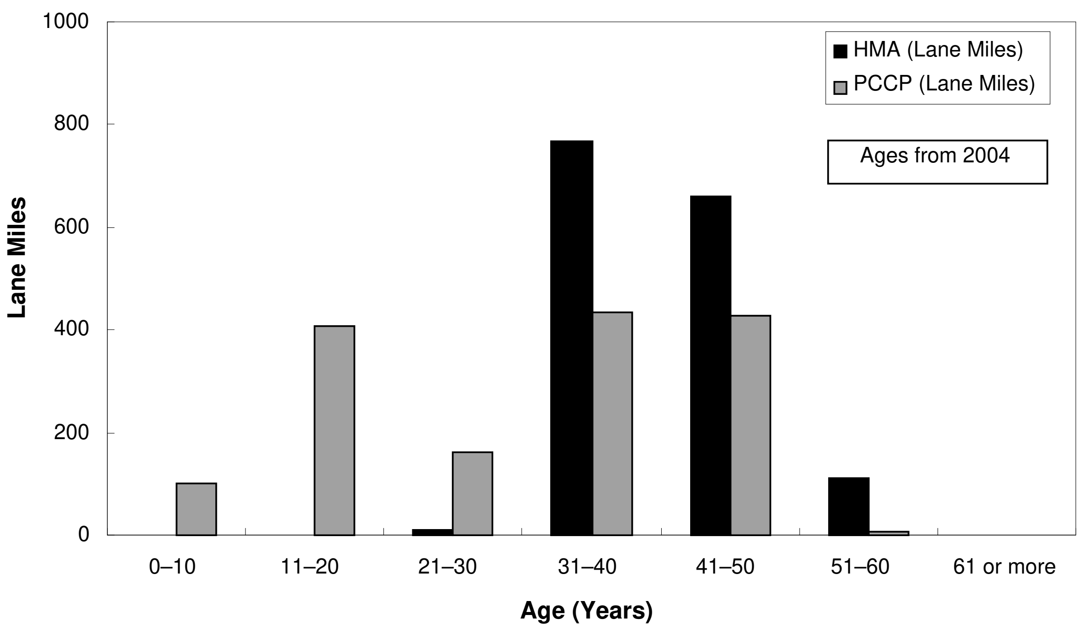

FIGURE 2 Oregon DOT Interstate pavements, time to first rehabilitation.

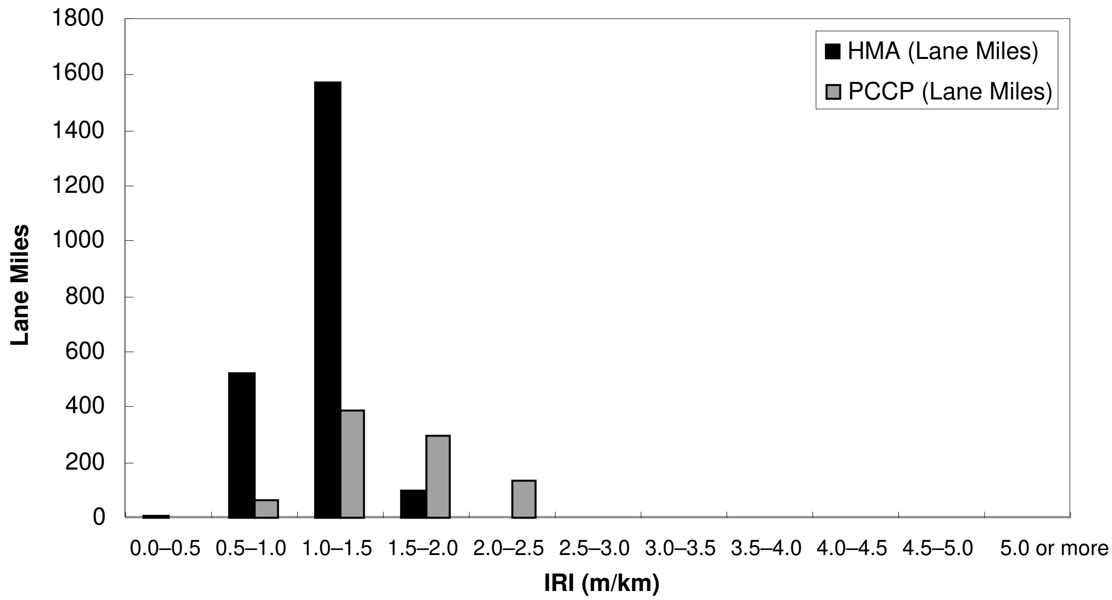

FIGURE 3 Oregon DOT Interstate pavements, IRI in 2004.

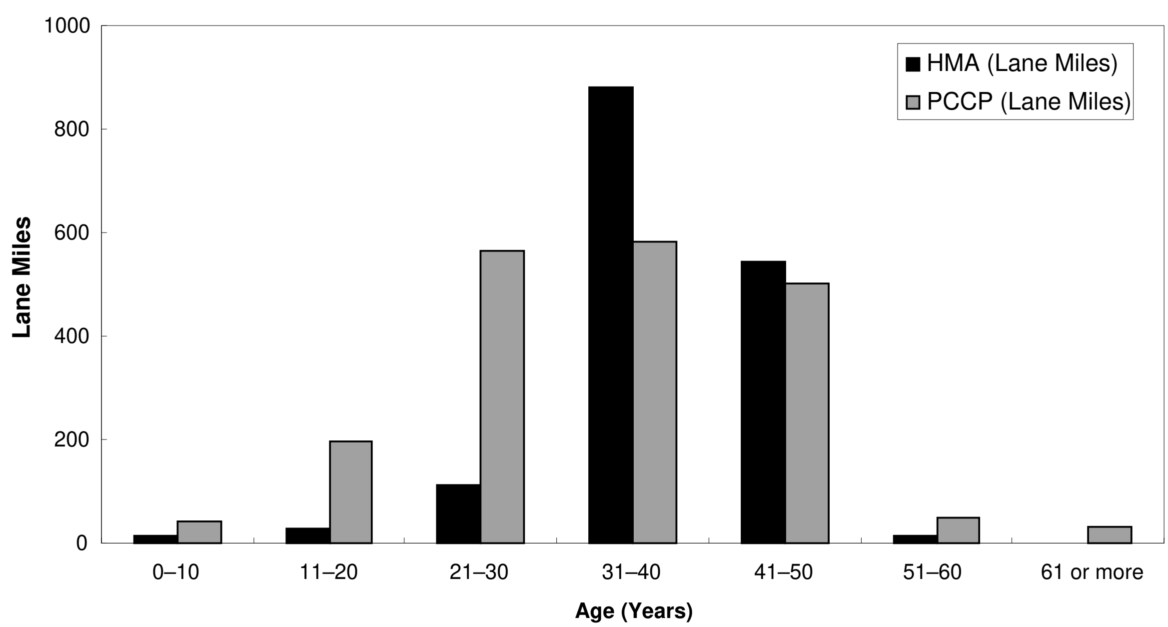

FIGURE 4 Washington DOT Interstate pavements, from initial construction or reconstruction.

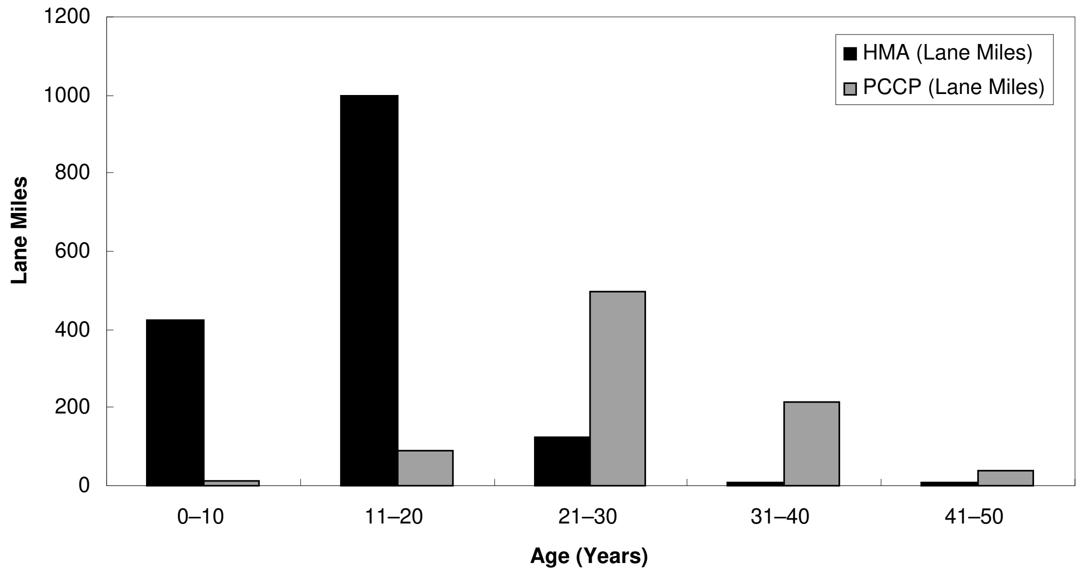

FIGURE 5 Washington DOT Interstate pavements, to first rehabilitation.

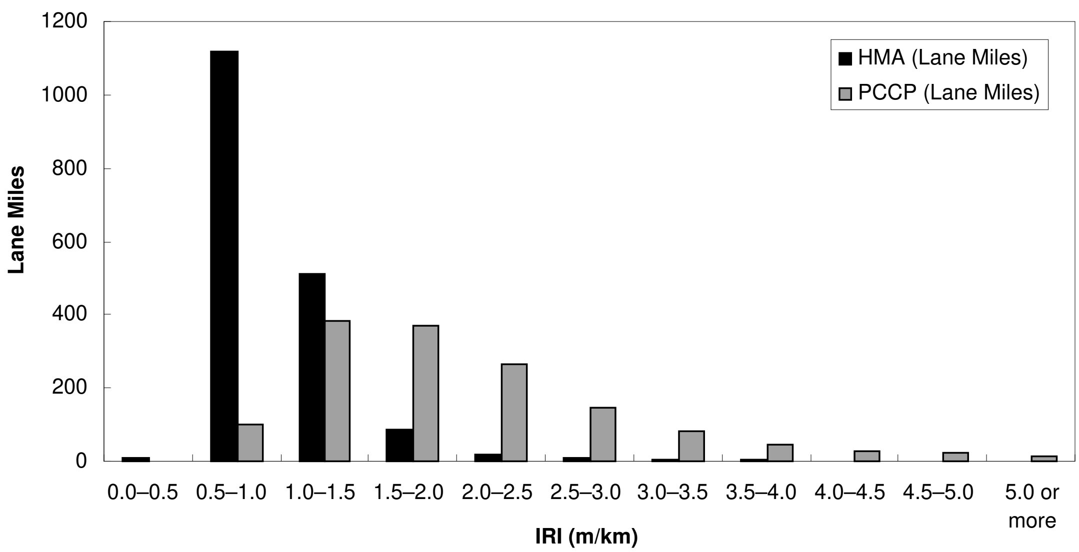

FIGURE 6 Washington DOT Interstate pavements, IRI in 2004.

Figure 4 shows the same type of data but for Washington State. The trends are somewhat similar to Oregon. The flexible pavements were generally constructed about 20 to 50 years ago, with an average of about 35 to 40 years. The PCCP were mostly constructed 20 to 50 years ago, with the oldest about 60. The average PCCP age is about 35 years. Like Oregon, few lane miles have been constructed over the past 10 years.

Time to First Major Rehabilitation

Figures 2 and 5 are used to overview the time to first major rehabilitation for Interstate pavements in Oregon and Washington.

Oregon data (Figure 2) show that for Interstate flexible pavements the time to first major rehabilitation ranges from near zero to 30 years. Typically, rehabilitation (mostly HMA overlays) occurs about 15 years following construction. PCCP time to first rehabilitation occurs later than for flexible pavements (as expected). On average, rehabilitation occurs about 30 years following construction within a range of about 20 to 40 years.

Washington data (Figure 5) reveal the same trends as Oregon. Various studies in Washington State have shown that the short times to first rehabilitation for flexible pavements are largely because of construction-related issues or stage construction (stage construction was done more frequently in Washington State in the 1950s and 1960s—though not today).

IRI

Figures 3 and 6 are used to overview the current IRI (as of 2004) for both flexible and rigid Interstate pavements in Oregon and Washington.

Oregon data (Figure 3) show that HMA-surfaced pavements are smoother than PCCP. The average IRI for HMA is about 1.0 m/km. The PCCP IRI average is closer to 1.5 m/km. The distribution of IRI values is spread over a wider range than HMA—likely reflective of the longer times to first rehabilitation.

Washington data (Figure 6) show that HMA-surfaced pavements are substantially smoother than PCCP. The average IRI for HMA is less than 1.0 m/km and falls within a tight range with a maximum of about 1.5 m/km. PCCP has an average IRI of about 2.0 m/km within a distribution of IRI values ranging up to about 5.0 m/km. Again, the higher IRI values for PCCP are likely reflective of longer times to rehabilitation. Overall, the IRI variability for PCCP is higher for Washington when compared with Oregon.

Oregon and Washington allow the use of studded tires during the winter seasons. This type of wear, undoubtedly, impacts the reported IRIs.

The Caltrans Highway Design Manual notes that a terminal IRI of 2.5 m/km is the maximum allowed at the end of life for rigid pavement (Caltrans, 1996).

SUMMARY AND CONCLUSIONS

The following summary and conclusions can be drawn for the IHS pavements for these three states.

Summary

Flexible pavements in all three states use more HMA today than 50 years ago. However, this suggests that over the years these state DOTs observed that thick HMA pavements make for a high performing structural section. Both pavement types typically have long lives. Overall, on the basis of the data available, flexible pavements are somewhat smoother than PCCP.

PCC pavements for all three states started out in the 1950s at either 8- or 9-in.-thick slabs. Today, both California and Washington appear to be converging toward similar slab thicknesses, use of dowels, and base type. Oregon has used and will continue to use CRCP for their rigid pavements.

Conclusions: Flexible Pavements

- Early Interstate pavements: HMA thicknesses were thinner than today but were often on stabilized bases.
- Today: Thick HMA is typical. Pavement age distributions are similar for Oregon and Washington.
- The time since original construction for flexible pavements is about 40 years, on the basis of data from Oregon and Washington. This suggests that thick HMA structural sections can be long-lived. In fact, there is little difference in the ages of the structural sections for both flexible and rigid pavements.
- The time to first major rehabilitation (mostly HMA overlays) is about 15 years.
- The smoothest of the Interstate flexible pavements are, in general, quite good—typically about 1.0 m/km or less.

Conclusions: Rigid Pavements

- Early Interstate pavements: PCC slab thicknesses were about 3 in. less than today. Base types were more varied.
- Today: California and Washington are converging toward similar PCC pavement designs. Slab thicknesses are about the same for all three states; however, Oregon uses CRCP and California and Washington JPCP. HMA is emerging as a preferred base type—certainly for Oregon and Washington.
- The use of CTB for PCCP was widespread in California but ranged from limited use to none for Oregon and Washington. Performance trends and state policies suggest that CTB will not be used for future designs.
- The time since original construction for rigid pavements is about 30 to 40 years, on the basis of data from Oregon and Washington.
- The time to first major rehabilitation is about 30 years.
- The smoothest of the Interstate rigid pavements are typically about 1.5 to 2.0 m/km. The range of IRI for PCCP is higher than for flexible Interstate pavements on the basis of Oregon and Washington data.
- The IRI values for Oregon Interstate pavements are lower than those observed in Washington State. This might be significant (JPCP versus CRCP pavement types) or due to some other factor not readily apparent.

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Interstate Highway System Challenges

North Atlantic States

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The IHS, originally conceived for its military value, has seen service in a variety of ways, only some of which were foreseen by its creators. The original aspect is now a small portion of the system use. The Interstate system is a key component of freight delivery for just-in-time manufacturing, an important link in commuting, emergency evacuation, and tourism. The challenges to maintain the aging system are huge, with the traveling public having higher expectations for rapid project delivery. This creates new challenges to provide reconstruction with new and innovative methods, as well as maintain the portion of the system that is in good condition with the use of pavement preservation techniques.

The New York State DOT demonstrated innovation in reconstruction by using precast concrete segments to reconstruct the Tappan Zee Toll facility. The work required tight tolerances for the prefabrication of the panels, as well as for elevations. Liquidated damages charged to the contractor at a rate of $1,300 per minute per lane closure after 6 a.m. indicates the importance of minimizing inconvenience to the driving public.

New Hampshire deals with lower traffic volumes but high potential for freeze–thaw damage and has responded to the challenge by attempting full-depth reclamation on several Interstate sections. Following a few trials, some sections have experienced tenting where water infiltrates the reclaimed material and initiates formation of an ice lens, which expands. New Hampshire is evaluating alternate treatments and improved compaction methods to deal with this distress.

Pavement preservation efforts are reported in a number of states, including North Carolina and Vermont. North Carolina has applied 25 mi of polymer-modified thin-wearing course to a four-lane divided JPCP in order to improve ride quality and extend the useful life of the pavement by 6 to 8 years. Vermont has placed open-graded friction course on Interstate highways to reduce truck spray. While successful in reducing wet weather spray, when the open-graded friction course began raveling, failure was rapid. As a result, Vermont has removed this treatment from its wearing course alternates.

The North Atlantic states are responding to the challenges with our Interstate system with creativity in construction, by trying and refining new methods and by conserving good pavements with pavement preservation treatments.

INTRODUCTION

As part of a TRB Pavement Management Section (AFD00) effort, two sessions were developed to commemorate the anniversary of the Interstate highway system. One of those sessions dealt with the AASHO Road Test and the important lessons learned and applied to the Interstate system. This session reflects the regional Interstate lessons learned from pavement performance. In this presentation, the focus is on system use, key challenges for the Interstate system, and recent efforts to reconstruct, maintain, and improve the Interstates. Engineers from six states in the North Atlantic region were interviewed; their collective opinions and experiences are included in this regional perspective.

The IHS was born out of military need: the now-famous trip across the country by Dwight Eisenhower that took more than a month was one motivator. The United States also saw the benefit of high-quality roadways for the German army in moving manpower and equipment during World War II.

Use of the IHS has expanded rapidly since completion of key segments. Tourism and travel have been facilitated by the system, enabling many Americans to visit national parks, festivals, tourist attractions, and cities. "See the USA in a Chevrolet" was a popular marketing jingle during the 1960s. This "see the USA" concept would not be feasible without high-speed roadways reducing travel time to reasonable levels.

The Interstate system has also become an important evacuation route for coastal areas. North and South Carolina both have evacuation plans that temporarily turn four-lane divided highways into one direction of traffic away from expected storm routes. The North Carolina poster for hurricane evacuation is shown in Figure 1 and is posted in rest areas and welcome centers in the coastal region. Houston, Texas, also used Interstate highways to evacuate during the summer of 2005. Work still needs to be done in the area of coordinating these evacuations to prevent traffic congestion to the point where little safety benefit is achieved. South Carolina used a directional switch to add capacity during hurricane evacuation of its coastal areas, as shown in Figure 2.

One unanticipated use of the Interstate system has been freight delivery. Just-in-time delivery means that components arrive at the fabrication site just when they are needed. Little to no inventory of parts reduces plant overhead and improves productivity as long as shipments are received. Nancy Dunn, a member of the North Carolina Board of Transportation, reported that Dell Computers in Winston–Salem has a 4-h window from the time of component delivery at the plant to when a finished computer is ready for shipment. This philosophy has become the method of choice among manufacturers, and the result is a substantial increase in truck traffic. For this business model, congestion is not just an inconvenience, but a direct impact to the bottom line of a manufacturing plant. Freight companies require safe, high-speed, and dependable roadways to ensure that client schedules are met.

Commuting has also changed significantly, and the use of Interstate highways as part of daily commuting is the norm, not an exception. Initially the Interstate was envisioned for long trips instead of daily travel. More and more commuters use the Interstates for one or more legs of their daily commute. And increasing real estate costs around traditional suburban areas have pushed the length of daily commutes. Nancy Dunn reported that 100,000 people left their home county to travel to Greensboro, North Carolina, to work in the urban area. Most of that number, and a large number of people within the city's county, use Interstate highways.

Traffic growth on Interstate highways was reported by all states in the North Atlantic region to outstrip the traffic projections. When the Interstate system was developed, the idea of building for a 20-year design life came into being. During the middle to late 1950s, the designs were to last until 1975. The ability to design adequately for 20 years of traffic requires that correct estimates of both current traffic and projected traffic. Even recent estimates have been known to be inaccurate. When the segment of I-40 near Burlington, North Carolina, was opened in the mid-1990s, traffic exceeded the 20-year design traffic in a period of only 4 years (Figure 3). This is a grand challenge for pavement designers, and state DOT maintenance forces required to maintain the underdesigned roadway into the future as well.

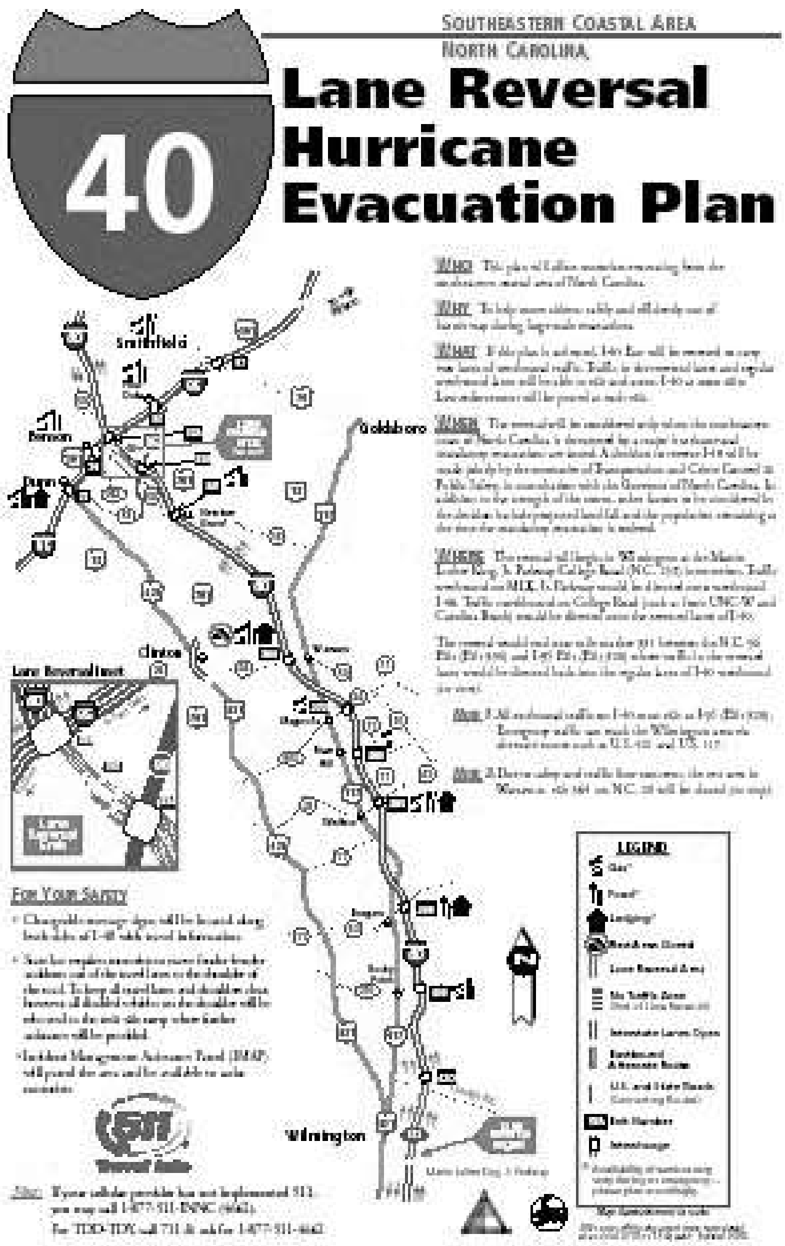

FIGURE 1 Lane-reversal hurricane coastal evacuation plan for North Carolina DOT.

FIGURE 2 Lane reversal in South Carolina during Hurricane Charley.

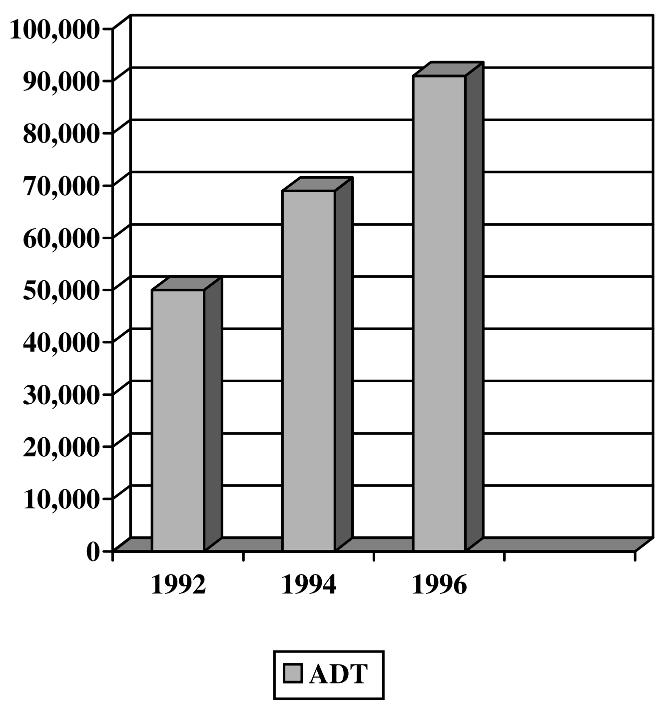

FIGURE 3 Traffic growth after opening of I-40 in North Carolina.

CONSTRUCTION–RECONSTRUCTION DILEMMA

Jack Lettiere, highway administrator for New Jersey DOT, commented at the 2005 North Carolina DOT/Consulting Engineers Conference on the importance of transportation to our citizens. "Transportation is the game board on which everything is played—getting children to school, folks to work, freight delivery, etc. Imagine the impact of reduced congestion on family life and community service. Fifty million Americans are homebound by virtue of lack of transportation. Imagine the cost to our productivity."

Lettiere mentioned the efforts by highway agencies to engage the American public in the infrastructure needs of the highway system: to be willing to fund necessary maintenance, rehabilitation, and reconstruction. This has included discussions on the number of miles of poor condition pavement, the number of deficient bridges, and even the number of fatalities caused by pavement conditions. The public has not responded to any of these appeals. To what have they responded? According to Lettiere, the public has responded to project delivery. The public wants transportation agencies to deliver projects in a timely manner with minimal traffic impacts. There is risk associated with these projects because they require agencies to try new methods and materials, but it is a risk that we must take.

An example of such a high-risk project was the reconstruction of the Tappan Zee Toll Plaza by New York DOT. Because of traffic requirements, the contractor was required to be off the roadway every morning by 6 a.m., with a $1,300 per minute liquidated damage if not. The agency selected to use prefabricated concrete panels for the roadway reconstruction. Each panel was carefully measured; detailed survey controls as well as manufacturing controls were enforced (Figure 4). Once the panels were constructed and cured, sawing of the joints was done at the plaza on the night before reconstruction. As soon as traffic control was in place, the

FIGURE 4 Precast concrete panel for Tappan Zee Toll Plaza reconstruction.

sawed slabs were removed to flatbed trucks and taken off-site. Grading of the subgrade was carefully controlled and select material was placed (Figure 5). Each precast slab was seated, connecting dowels were epoxied into place, and the dowel slots were patched. The progression of the work is shown in Figures 6 and 7.

An example that illustrates the risk aspects of alternative methods is the use of reclaiming of Interstate flexible pavements by New Hampshire DOT. Because of the deep frost penetration issues, a dominant form of distress is cold temperature cracking and frost heave. When the flexible pavements reach maximum tolerable distress, they are reclaimed through the full depth of the asphalt, the ground material is compacted, and a new surface course is applied. Figure 8 shows the distress level typical for an Interstate pavement recommended for reclaiming. Unfortunately, New Hampshire DOT has experienced some return of cold weather distress in the form of tenting in some of the reclaimed Interstate pavement (Figure 9). Tenting is differential deflection at a crack caused by moisture accumulating and freezing below the crack. New Hampshire DOT measured tenting of 0.75 in. over a 5-ft straight edge. The department is investigating the causes of the premature failures and is evaluating other alternate methods of rehabilitating pavement in cold environments.

ENVIRONMENTAL ISSUES

Most of the Interstate system was designed and constructed before environmental initiatives to minimize impacts, protect endangered species and their habitat, and control pollution during and after the construction process. Many sections of Interstate would require extensive mitigation or bridging if it were constructed today. This impacts the development, expansion, and reconstruction of Interstate pavements.

FIGURE 5 Placement of precast panel at Tappan Zee Toll Plaza.

FIGURE 6 Progression of work on Tappan Zee Toll Plaza.

FIGURE 7 Construction nearing completion on Tappan Zee Toll Plaza.

FIGURE 8 Candidate road condition for reclaiming by New Hampshire DOT.

FIGURE 9 Tenting failure of reclaimed Interstate in New Hampshire.

Maintaining project timelines is easiest if modernization projects can be limited to the existing right of way of the system. According to North Carolina State Highway Administrator, Len Sanderson, “We must demonstrate a commitment to environmental stewardship if we hope to maintain a schedule of interstate reconstruction.”

PAVEMENT PRESERVATION

The philosophy of pavement preservation states that it is most cost effective to extend the time when pavements are in good condition with relatively low-cost treatments rather than letting the pavement condition deteriorate until more extensive work is required. This philosophy runs counter to the worst-first treatment approach that was the norm during most of the past 30 years. However, North Atlantic states are using the approach on their Interstate pavements to preserve those in good condition.

Vermont has had a program of applying open-graded friction course to Interstate pavements. The open-graded friction course provides excellent pavement surface frictional characteristics and reduces truck spray during rain events. There have been issues with freezing and icing during winter weather and with sudden raveling as the mode of failure. These sudden failures have lead Vermont to remove these treatments over time. North Carolina, with a more favorable weather regime, has successfully used polymer modifiers in open-graded friction course to reduce the tendency for raveling. Use of salt brine in advance of freezing temperatures has been successful in preventing ice buildup.

North Carolina has also used polymer-modified ultrathin wearing course as a preservation treatment to improve ride quality on 30-year old JCP. The pavement had shallow spalling at almost 100% of the joints. After these were repaired, the wearing course was applied, and significant improvement in ride quality and appearance was noted. While reflection cracking appeared within a year, no degradation of the cracks or decline in ride quality has occurred in the 5 years since application of the wearing course (Figure 10).

Other pavement preservation strategies that have been used in the region include sealing joints and cracks, slab replacement, diamond grinding, thin overlays, and high-quality seals. New Hampshire has experimented with a proprietary glass–based grid sheeting product to impede reflection cracking.

CONCLUSIONS

The challenges to the states in maintaining, rehabilitating, and reconstructing the IHS are the result of the success of the system. Most of the system is 30 years old and has had four or more times its design loadings. Capacity is an issue in many locations, but environmental issues and costs make it beneficial to remain within the current right of way footprint.

Manufacturing depends on just in time delivery of components and rapid delivery of completed products. This has increased the user costs associated with congestion in general and construction related congestion in particular. Citizens relate to the rapid delivery of projects, which impacts their commutes, their family life, and their communities.

The North Atlantic states demonstrated a willingness to share both successes and challenges. When challenges occurred, new methods or modifications of existing methods were

FIGURE 10 High-quality polymer-modified ultrathin wearing course, Burke County, North Carolina.

found to improve processes. The region is moving forward in construction, in maintenance and in pavement preservation while keeping an eye on the needs and desires of customers.

ACKNOWLEDGMENTS

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APPENDIX

List of Participants

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