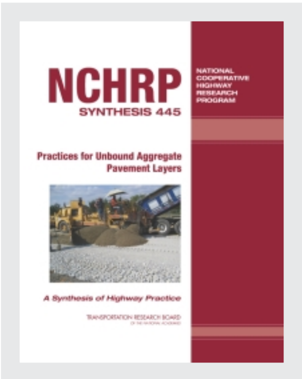


Practices for Unbound Aggregate Pavement Layers

DETAILS

180 pages | 8.5 x 11 | PAPERBACK
ISBN 978-0-309-27085-4 | DOI 10.17226/22469

AUTHORS

Tutumluer, Erol

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP SYNTHESIS 445

Practices for Unbound Aggregate Pavement Layers

A Synthesis of Highway Practice

CONSULTANT
Erol Tutumluer
University of Illinois–Urbana-Champaign
Urbana, Illinois

SUBSCRIBER CATEGORIES
Geotechnology • Highways • Materials

Research Sponsored by the American Association of State Highway and Transportation Officials
in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD

WASHINGTON, D.C.
2013
www.TRB.org

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NOTE: The Transportation Research Board of the National Academies, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

NCHRP SYNTHESIS 445

Project 20-05, Topic 43-03
ISSN 0547-5570
ISBN 978-0-309-27085-4
Library of Congress Control Number 2013936323

COPYRIGHT INFORMATION

NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program, conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council.

The members of the technical panel selected to monitor this project and to review this report were chosen for their special competencies and with regard for appropriate balance. The report was reviewed by the technical panel and accepted for publication according to procedures established and overseen by the Transportation Research Board and approved by the Governing Board of the National Research Council.

The opinions and conclusions expressed or implied in this report are those of the researchers who performed the research and are not necessarily those of the Transportation Research Board, the National Research Council, or the program sponsors.

The Transportation Research Board of the National Academies, the National Research Council, and the sponsors of the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of the report.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
Business Office
500 Fifth Street, NW
Washington, DC 20001

and can be ordered through the Internet at:
http://www.national-academies.org/trb/bookstore

Printed in the United States of America

Cover figure: Work is conducted on a pavement test study at the Advanced Transportation Research and Engineering Laboratory of the University of Illinois at Urbana–Champaign. Photo by the Illinois Center for Transportation, Advanced Transportation Research and Engineering Laboratory, University of Illinois at Urbana–Champaign.

FOREWORD

Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice.*

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

By Jon M. Williams
Program Director
Transportation Research Board

Properly designed and constructed unbound aggregate layers have the potential to improve pavement performance and longevity. This study gathers information on the current state of practice and research on unbound aggregate. The study finds that no common practice exists among state transportation agencies. Accordingly, the report summarizes important aspects and effective practices related to material selection, design, and construction of unbound aggregate layers. Prevalent agency practices are summarized and key lessons learned from research studies are highlighted.

Information for this study was acquired through a literature review and surveys of state and Canadian transportation agencies.

Erol Tutumluer, University of Illinois at Urbana–Champaign, collected and synthesized the information and wrote the report. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

PRACTICES FOR UNBOUND AGGREGATE PAVEMENT LAYERS

SUMMARY

Properly designed and constructed unbound aggregate layers have the potential to improve pavement performance and longevity while also addressing today's issues of the costs of other pavement materials, the need to save energy, and the desire to reduce greenhouse gas (GHG) emissions associated with the construction and reconstruction of pavements. Pavement projects using granular layers will have to be sustainable and cost-effective by (1) making more effective use of locally available materials through beneficiation and use of marginal aggregate materials, (2) increasing effective use of recycled aggregate products such as recycled crushed concrete (RCA) and reclaimed asphalt pavement (RAP), and (3) targeting long life and improvement in pavement performance.

North American transportation agencies have diverse specifications and construction practices for unbound aggregate base (UAB) and subbase layers. Sharing experiences and effective practices for unbound aggregate layers among transportation agencies would lead to the design and construction of better-performing, more economical, and sustainable pavement systems. The primary objective of this synthesis was to gather information on the current state of practice and the state-of-the-art research findings on the following topics:

1. Materials characterization and quality of natural aggregate and common recycled materials that relate to performance;
2. Properties of unbound aggregate layers that are used in the design of pavements and how they are determined;
3. Influence of gradation and other aggregate properties on permeability;
4. Current practices and innovations in construction, compaction, and quality assurance procedures [such as compaction in thicker layers, use of intelligent compaction (IC) systems, and the use of tests other than density in evaluating in-place modulus, stiffness, and quality];
5. Performances of different base types, such as the concept of a granular base over a stiff, often cement-treated subbase layer used in inverted pavements, in research pavement sections;
6. Potential to save energy and hauling costs by better utilizing local aggregates and recycled materials;
7. State specifications that lead to how contractors manage storage, transport, and placement of materials to minimize degradation of material properties and performance: lessons learned;
8. How states address climatic, subgrade, and drainage considerations in design of aggregate base layers.

Relevant information was gathered through a literature review, survey of U.S. state and Canadian provincial transportation agencies, industry input, and selected interviews. A total of 46 transportation agencies (including four Canadian provinces) responded to the survey questionnaire. Review of survey responses and subsequent interviews with agency personnel indicated that no common practice exists across agencies as far as the design and construction of unbound aggregate pavement layers is concerned. Most agencies do not have a defined protocol to introduce new and recycled materials into pavement construction. Although numerous

research and implementation projects over the years have recommended optimum design and construction practices for unbound aggregate pavement layers, there appears to be a significant delay before such recommendations are adopted into agency practice. Accordingly, this report summarizes important aspects and effective practices related to material selection, design, and construction of unbound aggregate pavement layers. Prevalent agency practices are summarized, and key lessons learned from research studies focusing on unbound aggregate pavement layers are highlighted. This information can be used to establish the need for and initiate the development of harmonized protocols for optimum design and construction of better performing, cost-effective unbound aggregate pavement layers.

CHAPTER ONE

INTRODUCTION

INTRODUCTION AND BACKGROUND

As defined by the ASTM International in ASTM D 8-11, an aggregate is "a granular material of mineral composition such as sand, gravel, shell, slag, or crushed stone, used with a cementing medium to form mortars or concrete, or alone as in base courses, railroad ballasts, etc." According to the National Stone, Sand and Gravel Association (NSSGA), nearly two billion metric tons of natural aggregate were produced from sand and gravel pits and stone quarries in 2010 at a value of approximately $17 billion, contributing $40 billion to the gross domestic product of the United States (http://nssga.org/ssgReview/index.cfm). Large quantities of produced sand, gravel, crushed stone, and, increasingly, industrial by-products and reclaimed construction materials go into the construction of transportation infrastructure for building road base, riprap, cement concrete, and asphalt concrete to provide bulk, strength, durability, and wear resistance in these applications. According to U.S. Geological Survey (USGS) reports, production and use of aggregates in the United States declined during the economic downturn in the years 2008 to 2010. The demand for all types and uses of aggregates in 2007 and 2008 was on the order of 2.5 to 3 billion tons (2.2 to 2.7 billion metric tons) per year, and Meininger and Stokowski (2011) have predicted the demand might return to such usage levels when construction volumes return.

According to *NCHRP Report 598* (Saeed 2008), unbound aggregate layers in flexible and rigid pavements generally serve to provide (1) a working platform, (2) structural layers for the pavement system, (3) drainage layers, (4) frost-free layers, and (5) "select fill" material (sometimes as part of the working platform). As a working platform, unbound aggregate layers often are constructed on soft, unstable subgrade soils or base to provide sufficient stability and adequate immediate support for equipment mobility and paving operations without excessive rutting. In flexible pavements, dense-graded unbound aggregate base (UAB) and subbase layers serve as major structural components of the pavement system to provide load distribution (that is, dissipation of high wheel load stresses with depth) and ensure adequate support and stability for the asphalt surfacing. In contrast, open-graded granular layers commonly are constructed in both rigid and flexible pavements primarily for drainage and frost-protection purposes. Note that UAB/subbase layers used in rigid pavement structures primarily provide uniform support conditions to the concrete slabs; the structural contribution of such layers often is not the primary design aspect.

The availability and cost of asphalt cement is directly related to the supply of petroleum and refining. Portland cement and steel require high fuel input for manufacturing, so use of asphalt contributes significantly to carbon dioxide emissions. Chehovits and Galehouse (2010) presented data from Chappat and Bilal (2003) to emphasize the significantly lower energy usage and greenhouse gas (GHG) emissions associated with aggregate production compared with other construction materials. From the data provided by Chappat and Bilal (2003), the energy consumption for aggregate production (per ton) ranges from 25,850 to 34,470 BTU/t (30 to 40 MJ/t), compared with 4.2 MBTU/t (4,900 MJ/t) for asphalt binder production. Similarly, the GHG emissions for aggregate production range from 5 to 20 lb. CO_2/t (2.5 to 10 kg CO_2/t) compared with 442 lb. CO_2/t (221 kg CO_2/t) for asphalt binder production. Given the higher cost of the cementitious portions of pavement layers and the subsequent adverse impact on natural resources, land use, and the environment, more effective and widespread use of unbound aggregate layers in pavement construction should result in significant conservation of energy and increased service life of transportation infrastructure.

An international scanning program sponsored by FHWA, AASHTO, and NCHRP in 2002 observed pavement design and construction practices in France, South Africa, and Australia and subsequently recommended the initiation of demonstration projects with deep subbase and deep base designs as cost-effective and sustainable pavement alternatives (Beatty et al. 2002). Properly designed and constructed unbound aggregate layers have the potential to improve pavement performance and longevity while also addressing today's issues of the costs of other pavement materials, the need to save energy and reduce GHG associated with the construction and reconstruction of pavements. Pavement projects using granular layers need to be sustainable and cost-effective by (1) making more effective use of locally available materials through beneficiation and use of marginal aggregate materials (aggregates that do not satisfy all material quality control (QC) requirements but may become allowable upon slight adjustment in material quality threshold parameters); (2) increasing effective use of recycled aggregate products, such as recycled concrete aggregate (RCA) and reclaimed asphalt pavement (RAP), in pavement construction; and (3) targeting long life and improvement in pavement performance.

North American transportation agencies have diverse specifications and construction practices for aggregate base and subbase layers. Sharing experiences and effective practices for unbound aggregate layers among transportation agencies would lead to better design and construction practices. For example, in flexible pavements, and especially for the most common applications of thinly surfaced low- to moderate-volume roads, it is critical that the unbound aggregates component of these transportation facilities is properly characterized by incorporating recent advances into solutions for a more accurate pavement analysis and improved field performance. Important new findings from major research studies [for example, from the International Center for Aggregates Research (ICAR)] provide proposed improvements in the design models and the compaction of unbound aggregate lifts in thicker layers (Allen et al. 1998; Adu-Osei et al. 2001; Tutumluer et al. 2001; Ashtiani and Little 2009). Furthermore, recent successful demonstration projects promoting more widespread use of intelligent compaction (IC) systems and the use of field tests other than just density in evaluating in-place stiffness and quality have received much attention through national and pooled fund studies (www.intelligentcompaction.com). Future use of modulus-based continuous compaction control approaches is being studied through an ongoing NCHRP study (NCHRP 10-84: Modulus-Based Construction Specification for Compaction of Earthwork and Unbound Aggregate) to potentially provide guidelines for standards and construction specifications for improved pavement construction and utilization practices with unbound aggregate layers.

Interest has also developed in domestic and foreign innovative construction practices, such as the "inverted pavement" concept of a granular base over a stiff, often cement-treated, subbase layer at depth. Such innovative practices fully emphasize the importance of unbound aggregates in terms of their functional usage and address potential and economic benefits from use in the construction of sustainable pavement infrastructure. In addition to such well-documented practices highlighted through international technology scanning programs, several test sections have been built, in Georgia, Louisiana, and Virginia, that apply the "inverted pavement" concept (Metcalf et al. 1998; Beatty et al. 2002; Titi et al. 2003; Lewis et al. 2012; Weingart 2012).

This important synthesis topic—"Practices for Unbound Aggregate Pavement Layers"—has consistently generated top priority rankings in recent ICAR/FHWA Technical Working Group meetings, clearly highlighting the need to organize and compress available information from current practices and recent advances in this field. Thus, this synthesis report concerns the full range of aggregate base and subbase issues for both flexible and rigid pavement systems in the following areas:

- Materials characterization and quality of natural aggregates and common recycled materials that relate to performance;
- Properties of unbound aggregate layers that are used in the design of pavements and how they are determined (different methods used by transportation agencies to design UAB/subbase layers is first determined);
- Aggregate properties that influence construction, compaction and performance;
- Current practices and innovations in construction, compaction, and QC and quality assurance (QA) procedures (such as compaction in thicker layers, use of IC systems, measuring and ensuring in situ drainage characteristics, and the use of tests other than just density in evaluating in-place modulus, stiffness, and quality, as well as measurements to ensure adequate in situ drainage characteristics);
- Performance trends of in-service pavements and experimental test sections, such as the Long-Term Pavement Performance (LTPP) and the Minnesota Department of Transportation's (MnDOT's) MnRoad, with different unbound base/subbase types, and climatic, subgrade, and drainage considerations in design of aggregate base;
- Role of unbound aggregates in sustainability and the potential to save energy and material hauling costs by better using local and marginal aggregates and recycled materials;
- How states manage storage, transport, and placement of aggregates to minimize segregation and degradation of material properties and maximize performance: lessons learned.

Significant benefits in consistency of UAB properties or performance could be derived from broader application and implementation of major findings from this synthesis. This type of work can also result in internal reviews within state transportation agencies of their processes and lead to implementation of new and improved construction practices, such as thicker lift aggregate bases, inverted pavement construction, IC, and innovative QC approaches, such as the "Percent Within Limits" (PWL) method, defined by the FHWA as: "the percentage of the lot falling above the lower specification limit (LSL), beneath the upper specification limit (USL), or between the USL and LSL" (http://www.fhwa.dot.gov/pavement/pwl/). Such advances can bring sustainability and offer economical and environment friendly green alternatives for road construction. This synthesis presents an extensive overview of the current states of practices concerning the design and construction of UAB/subbase layers, along with latest research findings in the corresponding areas. Suggestions for "effective practices" are provided for areas in which significant gaps between research findings and current practices are observed.

SYNTHESIS OBJECTIVES AND STUDY APPROACH

This study was initiated to gather and summarize information on existing practices for the design and construction of unbound aggregate pavement layers around the United States and Canada. The main objective of this synthesis study was to summarize the state of the art in design and state of the practice in construction of unbound aggregate pavement layers, as used by different transportation agencies. Agency surveys and reviews of research publications have been conducted to identify effective practices in characterization, design, placement, compaction, QC, and performance for unbound aggregate layers; the results have been compiled into this synthesis report. Therefore, this synthesis report primarily concerns the full range of UAB and subbase issues for asphalt, concrete, and rehabilitated pavements only and does not include unbound aggregate layer applications in unsurfaced pavements and gravel roads. In addition, other broader topics in the areas of chemical admixture (such as lime, cement, fly ash, or bitumen) and/or mechanical additive (geosynthetic, fiber, and so forth) stabilization of aggregates are excluded from the scope because such aggregate stabilization topics are subjects of separate synthesis studies. For example, the ongoing NCHRP 4-36 research study "Characterization of Cementitiously Stabilized Layers for Use in Pavement Design and Analysis" aims to recommend performance-related procedures for characterizing cementitiously stabilized pavement layers for use in pavement design and analysis and incorporation in the *Mechanistic–Empirical Pavement Design Guide* (MEPDG) (http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2494). Similarly, *NCHRP Synthesis 435* (Topic 40-01) "Recycled Materials and Byproducts in Highway Applications" aims to provide guidelines to states for revising their specifications to incorporate the use of recycled materials and other industrial by-products for pavement construction applications.

Information has been gathered through literature review on state, local, and international practices concerning design and construction of unbound aggregate pavement layers as well as through a comprehensive survey of the members of the AASHTO Highway Subcommittee on Materials (including Canadian provinces), and selected interviews. The survey questionnaire and a list of respondents are provided in the appendices of this synthesis report. The survey questionnaire had separate parts, which together were relevant to agencies with different experiences and needs regarding different designs and construction practices for unbound aggregate pavement layers. The information was requested to encompass all engineering aspects highlighted in the Summary of this synthesis report, primarily in the following categories:

1. Use of UAB and subbase layers;
2. Material selection and construction practices;
3. Characterization of UAB for design;
4. Compaction, QC, and field performance;
5. Recycling aggregates and recycled granular materials; and
6. Climatic effects and drainage.

Information was also gathered regarding possible special provisions governing the use of recycled materials in unbound aggregate layer applications. The questionnaire was purposely designed to be comprehensive and at the same time brief in an attempt to increase the response rate.

In addition, summaries of agency documents and research publications have been obtained as examples of current effective practices and recent advances and innovative techniques for improving pavement performance with UAB/subbase layers. Gaps in knowledge and current practices have been identified along with research needs to address these gaps. As a result, this synthesis report also provides information for potential harmonization of specifications (particularly on a regional basis) to ultimately benefit both North American transportation agencies and material producers without adverse impacts on pavement performance. Figure 1 shows a map of the United States with all the surveyed states highlighted. Note that four Canadian provincial agencies (Alberta, Newfoundland and Labrador, Ontario, and Saskatchewan) also responded to the survey questionnaire. Accordingly, information gathered from a total of 46 North American transportation agencies has been summarized in this synthesis.

Transportation Agency Use of Unbound Aggregate Base and Subbase Layers

The comprehensive synthesis survey questionnaire (see Appendix A) on Practices for Unbound Aggregate Pavement Layers was sent to all 50 U.S. states, the District of Columbia, Puerto Rico, and nine Canadian provincial transportation agencies. A total of 46 agencies responded to the survey in a timely manner and answered the first question in the general category, indicating that it was common practice for their agency to incorporate unbound aggregate layers into the design and construction of asphalt, concrete, and composite pavement structures, not including unbound aggregate layer applications in unsurfaced pavements and gravel roads in the survey focus. In accordance, Figure 2 shows in percentages the types of unbound aggregate layers commonly constructed by the responding transportation agencies. A great majority of the responses included construction of both the UAB (96%) and subbase (65%) courses in pavement layers. Nearly half of the responding agencies indicated they commonly built working platforms, and about one-fourth of all respondents often constructed open-graded drainage layers in their pavements. Note that the "others" category in the survey summary plots presented in this synthesis comprise "miscellaneous" responses reported by the surveyed agencies in lieu of the alternatives included in the questionnaire. A summary of all agency responses to the questionnaire is provided in Appendix C of this report.

Figure 3 shows in percentages the types of pavement structures incorporating unbound aggregate layers commonly designed and constructed by responding transportation agencies. All responding agencies routinely build flexible pavements with UABs. About 70% of the respondents construct rigid pavements with a granular base or subbase (note that it is unclear from the survey how many of the remaining 30% of the respondents construct rigid pavements on a regular basis; some agencies may construct rigid pavements on stabilized bases/subbases only). Accordingly, when it comes to

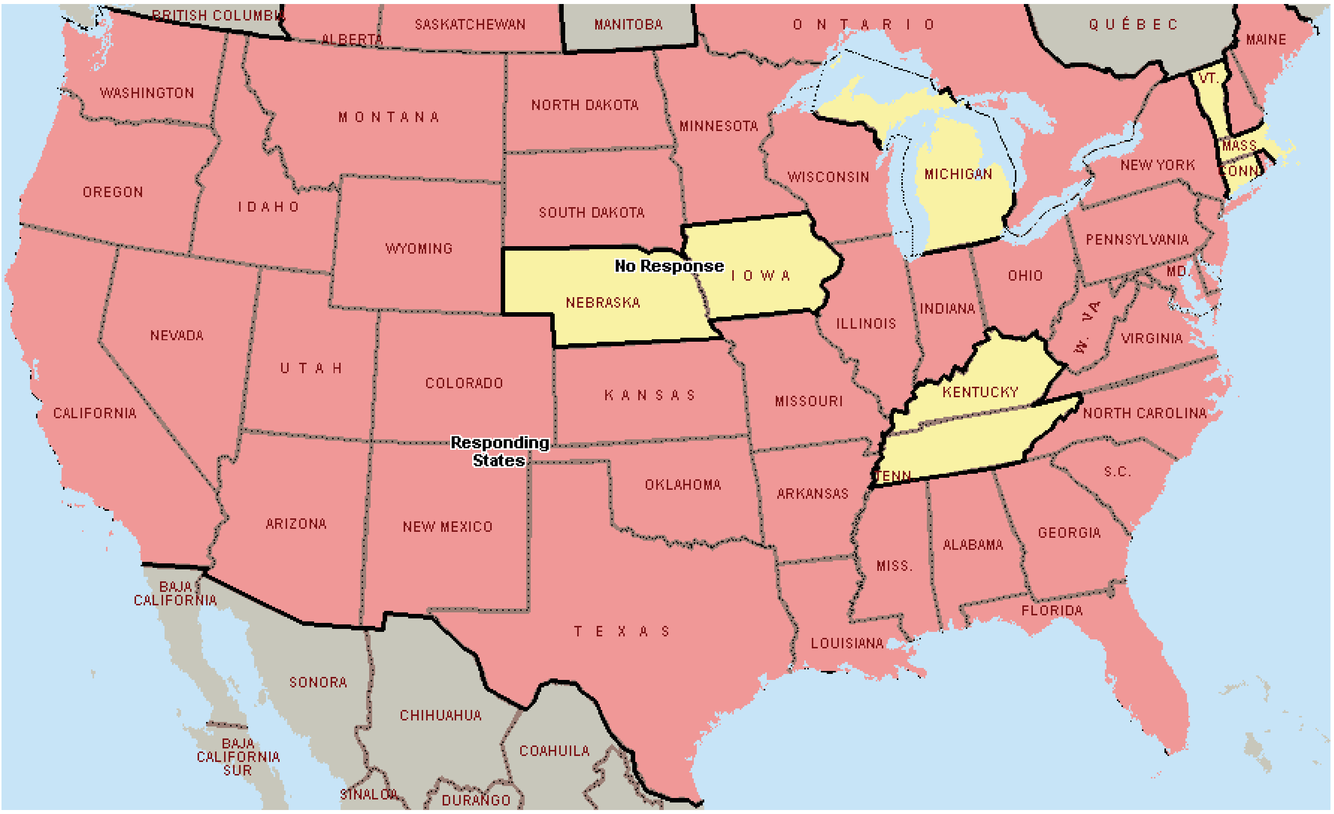


FIGURE 1 Map of the United States and Canada showing all surveyed agencies. Note the map does not show Alaska (response received), Puerto Rico (no response received), or Newfoundland and Labrador (response received).

rehabilitated pavements, the number of overlaid pavements using unbound aggregate layers in the rehabilitation process is only 30%. Interestingly, about one-fifth of all responding agencies also construct other types of pavements, such as composite and inverted, that have granular base and/or subbase layer(s).

Figure 4 shows in percentages the primary functionalities of unbound aggregate layers designed and constructed in pavement systems. Nearly all of the responding transportation agencies, as many as 94%, build dense-graded base courses as primary structural layers (only Virginia, Rhode Island, and Pennsylvania reported not using dense-graded base courses as primary structural layers). Consistently with Figure 2, 24% of those agencies construct open-graded aggregate layers under rigid pavements for uniform support and to provide drainage. This demonstrates that untreated open-graded drainage layers are not considered for flexible pavements by any agency. Furthermore, similar to the findings highlighted in Figure 2, about half of the responding agencies (52%)

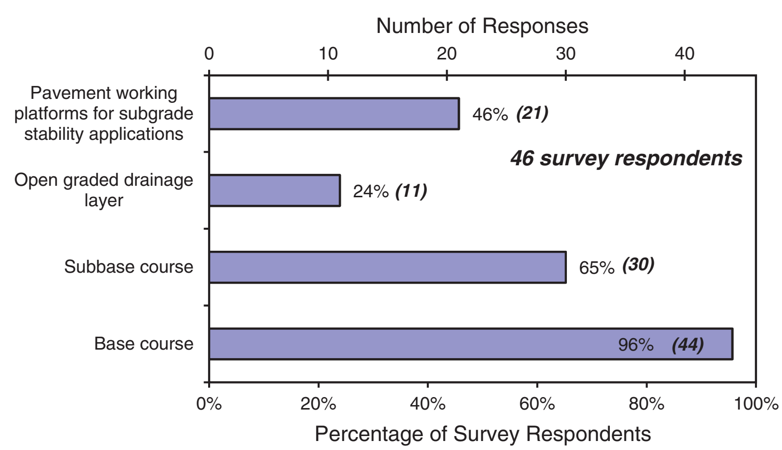


FIGURE 2 Types of unbound aggregate layers commonly constructed by transportation agencies.

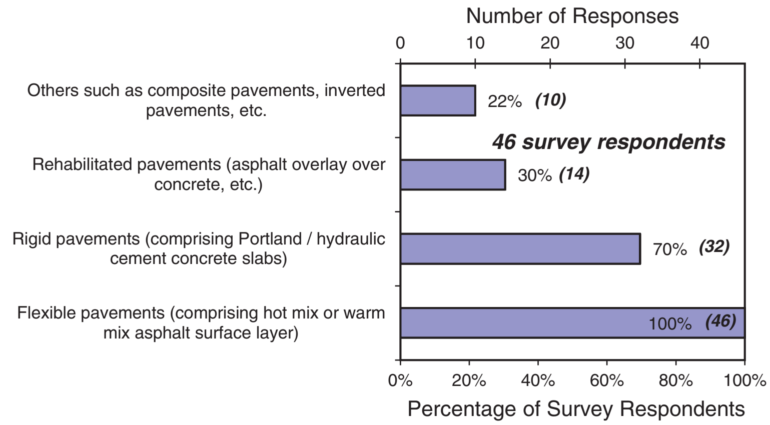


FIGURE 3 Types of pavement structures incorporating unbound aggregate layers commonly designed and constructed by transportation agencies.

build pavement construction platforms to protect weak subgrade layers from excessive rutting under heavy construction equipment loading. The pavement working platform, which is often referred to as the aggregate cover or subgrade replacement owing to its permanent foundation use in pavement construction, is clearly the second most common use of unbound aggregate layers after the structural base and subbase course application.

Key Lessons

- The use of UAB/subbase layers is a common practice across all transportation agencies in the United States and Canada.
- A synthesis summarizing the current state of the art and state of the practice concerning unbound aggregate pavement layers will help significantly in identifying desirable practices for the design and construction of better-performing sustainable pavement systems.

OUTLINE OF CHAPTERS

This synthesis contains six chapters. Chapter two provides a brief overview of different types of aggregate materials and their important properties and quality aspects that relate to agency acceptance criteria for good performance in pavement applications. Sustainable aggregate utilization practices in pavement construction are also highlighted, with special emphasis on how to make best use of local, marginal, and recycled aggregate materials in granular layers and how agencies could test and characterize recycled materials for unbound granular base/subbase acceptance and design. Chapter three summarizes aspects such as storage, transportation, material handling, placement methods, and lift thickness of aggregate materials adopted by transportation agencies to minimize segregation, degradation of material properties, and maximize performance through improved structural load-carrying ability and superior drainage characteristics. Applications of unconventional pavement types using unbound aggregate layers and related construction practices, such as the inverted pavement concept of a granular layer over a stiff layer at depth, are also described in chapter three. Gaps in knowledge

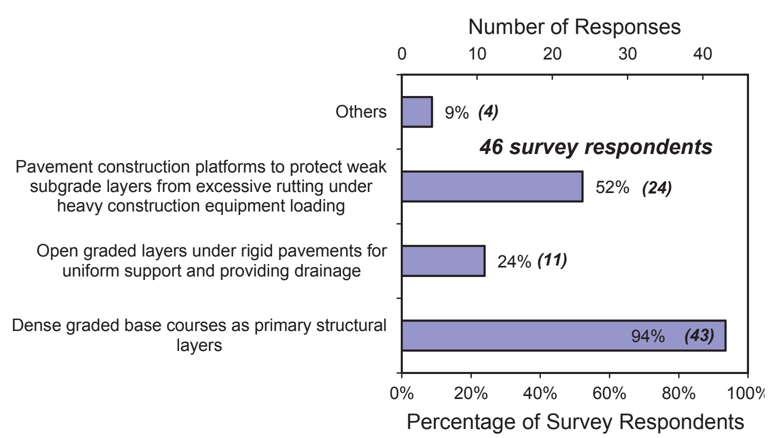


FIGURE 4 Primary functionalities of unbound aggregate layers intended to serve in pavement systems designed and constructed by transportation agencies.

concerning the "effective practices" for UAB and subbase layer construction, along with research needs to address these gaps, are described.

Chapter four reviews UAB/subbase structural pavement layer requirements by first defining typical load-transfer mechanisms and describing the related aggregate tests and characterization models for strength, modulus, and permanent deformation behavior; it is hoped this information will facilitate better designs of pavement systems and ultimately ensure adequate performance under traffic loading. Agency specifications and design approaches in use are reviewed, as are the new characterization tools and improved models (such as stress-dependent and anisotropic modulus, ICAR model, and so forth) developed for aggregate base/subbase layers through comparisons of the predicted and field-measured values in constructed unbound aggregate layer applications. Chapter four also reviews significant climatic effects, moisture or pavement drainage and temperature, and their significance on the design and performance of pavement systems with unbound aggregate layers.

Chapter five presents detailed findings from the literature review and extensive survey results on different approaches used by transportation agencies for compaction testing on laboratory samples, field compaction, QC/QA, and field performance evaluations of constructed UAB/subbase layers. Finally, chapter six provides a summary of the key findings of the synthesis report, including the state of the practice for unbound aggregate material selection and sustainable utilization, characterization, design, construction, compaction and QC, as well as performance evaluations. Chapter six also provides a summary of opportunities for additional research needs.

There are six appendices of this synthesis report. Appendix A presents the complete survey questionnaire that was sent to highway agencies in the United States and Canada. Appendix B lists the complete survey respondent information. Appendix C provides a detailed compilation of the survey responses. Appendices D and E present reviews of current unbound aggregate material resilient modulus and permanent deformation models, respectively. Finally, Appendix F provides additional information gathered from 14 state highway agencies through a follow-up survey on resilient modulus testing.

Note that the terms "unbound" and "bound" have been used interchangeably in this synthesis report to highlight the particulate nature of aggregate base and subbase layers when constructed without the application of any binding or stabilizing agent.

REFERENCES

Adu-Osei, A., D.N. Little, and R.L. Lytton, *Structural Characteristics of Unbound Aggregate Bases to Meet AASHTO 2002 Design Requirements,* International Center for Aggregates Research (ICAR) Report 502-1, Texas Transportation Institute, The Texas A&M University System, College Station, 2001.

Allen, J.J., J.L. Bueno, M.E. Kalinski, M.L. Myers, and K.H. Stokoe II, *Increased Single-Lift Thicknesses for Unbound Aggregate Base Courses,* ICAR Report No. 501-5, International Center for Aggregate Research, The University of Texas at Austin, 1998.

Ashtiani, R.S. and D.N. Little, *Methodology for Designing Aggregate Mixtures for Base Courses,* ICAR Report No. 501-5, International Center for Aggregate Research, Texas Transportation Institute, The Texas A&M University System, College Station, 2009, 336 pp.

Beatty, T.L., et al., Pavement Preservation Technology in France, South Africa, and Australia, Office of International Programs, Federal Highway Administration, U.S. Department of Transportation, and the American Association of State Highway and Transportation Officials, Alexandria, Va., 2002.

Chappat, M. and J. Bilal, *The Environmental Road of the Future: Life Cycle Analysis, Energy Consumption and Greenhouse Gas Emissions,* Colas Group, 2003 [Online]. Available: http://www.colas.co.uk/about-colas-detail.asp?pageId=14.

Chehovits, J. and L. Galehouse, "Energy Usage and Greenhouse Gas Emissions of Pavement Preservation Processes for Asphalt Concrete Pavements," In *Proceedings of the First International Conference on Pavement Preservation,* Newport Beach, Calif., Apr. 13–15, 2010, pp. 27–42.

Lewis, D.E., K. Ledford, T. Georges, and D.M. Jared, "Construction and Performance of Inverted Pavements in Georgia," Paper No. 12-1872, Poster Presentation in Session 639, 91st Annual Meeting of the Transportation Research Board, Jan. 22–26, 2012, Washington, D.C.

Meininger, R.C. and S.J. Stokowski, "Wherefore Art Thou Aggregate Resources for Highways," FHWA-HRT-11-006, *Public Roads,* Vol. 75, No. 2, Sep./Oct. 2011 [Online]. Available: http://www.fhwa.dot.gov/publications/publicroads/11septoct/06.cfm.

Metcalf, J.M., S. Romanoschi, L. Yongqi, and M. Rasoulian, *Construction and Comparison of Louisiana's Conventional and Alternative Base Courses under Accelerated Loading,* Interim Report 1, Phase 1, Louisiana Transportation Research Center, Baton Rouge, 1998.

Saeed, A., *NCHRP Report 598: Performance-Related Tests of Recycled Aggregates for Use in Unbound Pavement Layers,* Project 4-31, Transportation Research Board of the National Academies, Washington, D.C., 2008, 53 pp.

Titi, H., M. Rasoulian, M. Martinez, B. Becnel and G. Keel, "Long-Term Performance of Stone Interlayer Pavement," *Journal of Transportation Engineering,* Vol. 129, No. 2, 2003, pp. 118–126.

Tutumluer, E., A. Adu-Osei, D.N. Little, and R.L. Lytton, *Field Validation of the Cross-Anisotropic Behavior of Unbound Aggregate Bases,* International Center for Aggregates Research (ICAR) Report 502-2, Texas Transportation Institute, The Texas A&M University System, College Station, 2001.

Weingart, R., "Inverted Base: The Virginia Experience," 2012 Transportation Research Board Mineral Aggregates Committee (AFP 70) Meeting Presentation, 91st Annual Meeting of the Transportation Research Board, Jan. 22–26, 2012, Washington, D.C.

CHAPTER TWO

AGGREGATE TYPES AND MATERIAL SELECTION

INTRODUCTION

Continuous removal of available natural resources and increased material hauling and transportation costs have put an emphasis on finding "acceptable" materials to be used in pavement construction. The performance of any constructed pavement system largely depends on the quality of materials used in different layers. To ensure adequate performance of pavements under loading, transportation agencies have developed specifications that address certain minimum properties or qualities of construction materials. Performance of aggregates used in unbound pavement layers under loading is also influenced by such material properties of individual particles and the particle arrangement within the aggregate matrix as a bulk material.

This chapter provides a brief overview of the different types of aggregate materials available as natural resources and mined from sand and gravel pits and crushed stone quarry operations throughout the United States. Important aggregate properties and quality aspects, which enable aggregate material to meet agency specifications for pavement granular base/subbase use, are summarized to establish guidelines for aggregate source selection. Next, the concept of best value granular material use is introduced for pavement projects with the potential to save energy and material hauling costs through examples of recent sustainable construction practices, highlighting how local natural aggregates and recycled materials could be better used in granular base/subbase applications. The increasing trend to use recycled granular materials in base and subbase layers is discussed in detail. Important issues are reviewed to shed light onto what tests are used by agencies to characterize recycled materials for unbound granular base/subbase acceptance and design.

AGGREGATE TYPES AND SOURCES

According to the ASTM, aggregates are defined as "granular materials of mineral composition such as sand, gravel, shell, slag, or crushed stone, used with a cementing medium to form mortars or concrete, or alone as in base courses, railroad ballasts, etc." Based on the nature of their extraction from natural resources, aggregates used in pavement applications can be divided into two broad categories: (1) stone deposits and (2) sand and gravel deposits (Barksdale 1991). Industrial by-product materials, such as slags, have also been specified and used in granular base and subbase applications in some states.

Stone Deposits

Stone deposits can be broadly classified into the following three categories: (1) sedimentary rocks, (2) igneous rocks, and (3) metamorphic rocks. A brief discussion of the mechanism of formation for these three rock types is presented here, along with examples of each rock type. These three rock types usually are obtained from quarries through a blasting process and are processed through a series of crushers, pulverizers, and screening units to obtain aggregate materials for pavement and other construction applications. Note that depending on the processing methods, crushed aggregates produced by a crusher operation may be dry or wet immediately after production.

1. **Sedimentary Rocks:** These rock types are formed by chemical precipitates and the settlement of sediments or organic matter at or near the earth's surface and usually within bodies of water. Examples of sedimentary rocks include limestone, dolomite or dolostone, shale, and sandstone.

   The generic name "limestone" is used for commonly found carbonate rocks, including limestone, dolomite, and marble (Langer 2011). Limestone and dolomite usually form as a result of the consolidation and sedimentation of the shells of marine animals and/or plants. They may also form as a result of the precipitation of fine carbonate mud from marine waters. Limestone and dolomite constitute approximately 70% of crushed stone production in the United States (Willett 2011).
2. **Igneous Rocks:** These rock types are formed by the cooling and solidification of magma or lava. The process of magma solidification can occur below or on the earth's surface. Accordingly, igneous rocks formed below the earth's surface are called intrusive or plutonic rocks, whereas those formed on the earth's surface are called extrusive or volcanic rocks. As a result of the longer duration associated with cooling of magma in the formation of intrusive igneous rocks, the individual minerals have a chance to grow large enough

to see with a naked eye, and the rock has a coarse-grained texture, but extrusive igneous rocks, which cool rapidly from magma at or near the earth's surface, are too fine-grained to distinguish individual minerals. Igneous rocks often have high amounts of silica. Examples of igneous rocks used in pavement applications include granite (intrusive), basalt (extrusive), and rhyolite (extrusive).

The generic classification "granite" sometimes includes coarse-grained igneous or metamorphic rocks such as true granite, syenite, gneiss, and dark-colored gabbro (Langer 2011). Granites account for approximately 16% of crushed stone production in the United States (9% of total aggregate production). Although the hardness of individual particles leads to granite usually being classified as excellent crushed stone, some granitic type aggregates are weak and brittle because of their poorly bonded mineral grains, usually caused by weathering.

Fine-grained igneous rocks are also called "trap rocks." Trap rocks include dark-colored, fine-grained, volcanic rocks and make up about 9% of the crushed stone production (5% of the total aggregate production) (Willett 2008). Examples of trap rock are basalt and diabase. Excellent resistance to chemical reactions and ability to withstand high mechanical stresses led to the classification of trap rock as an excellent crushed stone material.

3. **Metamorphic Rocks:** These rocks are formed by the transformation of existing rocks (may be sedimentary or igneous) under heat and pressure. Examples of metamorphic rocks include quartzite, marble, slate, and gneiss. Metamorphic rocks as an aggregate can have widely variable characteristics. Many quartzites and gneiss can have properties similar to those of granite, whereas shale can be slabby and schist can be soft and flaky because of its high mica content.

Sand and Gravel Deposits

Aggregates are also extracted from sand and gravel pits, where the parent material has been transported from another location by fluvial, glacial, or alluvial processes to form loose deposits of natural sand and gravel. They are usually found in existing or historic river valleys or older, consolidated bedrock, glacial deposition, and mountain alluvial fans. Sand and gravel make up approximately 42% of the total aggregate production in the United States (Langer 2011). Depending on agency specifications and the nature of the deposits, aggregates obtained from gravel and sand pits may or may not be processed through a series of crushers before being used for pavement applications. Coarser sand and gravel materials are better for this purpose because the coarse particles can be crushed to smaller sizes. Note that in some cases, cobbles (particles larger than 75 mm or 3 in.) and small boulders (particles larger than 305 mm or 12 in.) in gravel deposits are also crushed.

Apart from the above two types of natural sources, other sources of aggregates include recycled materials and industrial by-products. A detailed discussion on different recycled materials and industrial by-products used in the construction of UAB and subbase layers is presented later in this chapter.

SUPPLY AND DEMAND FOR AGGREGATES IN THE UNITED STATES

According to the USGS, the demand for all types and uses of aggregates in 2007 and 2008 was on the order of 2.5 to 3.0 billion tons (2.2 to 2.7 billion metric tons) (Meininger and Stokowski 2011). These aggregates are obtained from natural resources or from recycled materials and/or industrial by-products. To improve and maintain the present conditions of the nation's infrastructure at an acceptable level, the demand for aggregates is presumed to increase with time. However, the supply of available natural aggregates is limited and undergoes gradual depletion with continual extraction and usage. Moreover, the availability of natural aggregate resources is constrained by geologic formations, encroaching land development, and the resource's proximity to the intended place of usage. Therefore, although some regions in the country may have abundance of available natural aggregate supply, natural aggregate supplies are scarce in most regions of the country.

Figure 5 shows the relative locations of aggregate resources in the conterminous United States (Langer 2011). As indicated, there is a limited supply of natural aggregates in the Coastal Plain and Mississippi embayment, Colorado Plateau and Wyoming Basin, glaciated Midwest, High Plains, and the nonglaciated Northern Plains. Thus, construction projects in these regions often require transportation of natural aggregates from other sources. Moreover, the limestone found in several regions of the country does not meet the hardness and durability requirements for use in pavement base and subbase layer applications. These conditions often demand the transportation of "good quality" natural aggregates from nearby sources to be used in pavement applications.

According to a 1998 USGS report, 27% of the crushed stone produced annually in the United States was used in pavement base construction (Wilburn and Goonan 1998). Similarly, 43% of the cement concrete debris produced was used for road base construction. On the other hand, 23% of the total sand and gravel production was used for road base construction, with portland cement concrete (PCC) production accounting for the highest proportion (45%) of use. According to the 2010 *Minerals Yearbook* published by the USGS, approximately 58.7 million metric tons of crushed stone was used in the United States for graded road base or subbase applications (Willett 2011). Similarly,

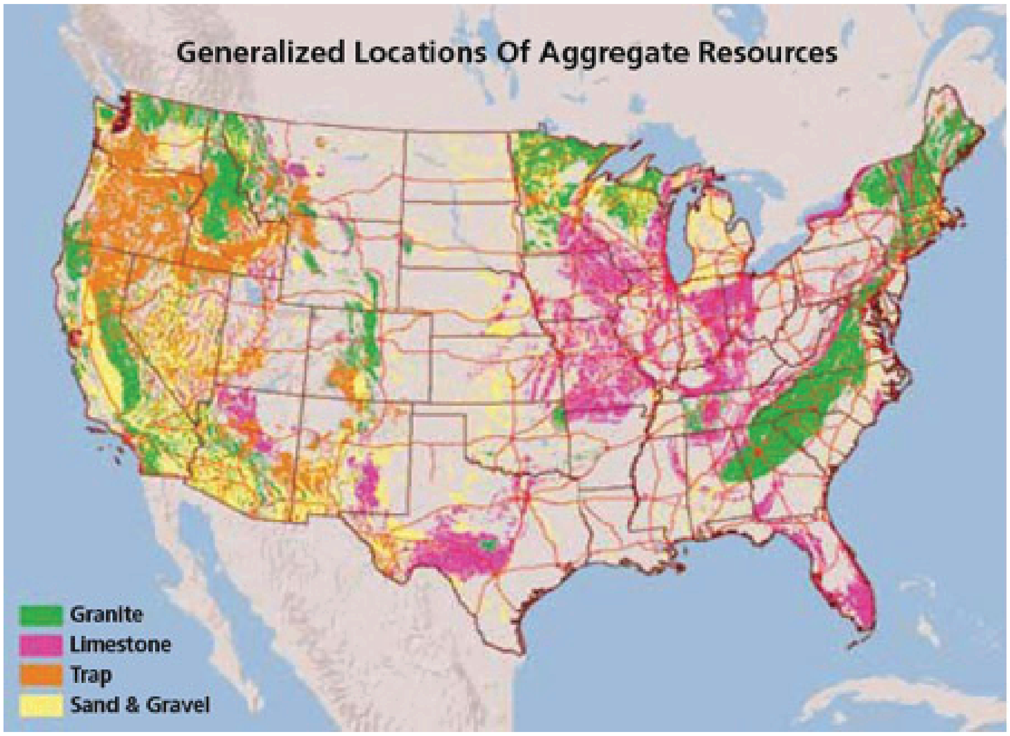


FIGURE 5 Generalized locations of aggregate resources in the conterminous United States (Langer 2011).

approximately 83 million metric tons of construction sand and gravel were used for road base and subgrade coverings (Bolen 2012).

AGGREGATE PROPERTIES AFFECTING UNBOUND AGGREGATE LAYER BEHAVIOR

Physical characteristics of the rocks that govern load-dissipating and particle-interlocking aspects differentiate "good" and "poor" quality aggregates with respect to the suitability for application in pavement unbound base/subbase courses. Moreover, chemical properties of the aggregates governing their durability and soundness are critical to ensuring long-lasting pavement structures. NCHRP Project 4-23, *NCHRP Report 453: Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers,* summarizes the most important tests that relate to the performance of aggregates in unbound pavement layers (Saeed et al. 2001). Among the tests highlighted, the screening tests (sieve analysis, Atterberg limits, moisture–density relationship, flat and elongated particles, uncompacted voids), durability test (sodium and magnesium sulfate soundness), shear strength tests [triaxial tests conducted on wet and dry samples and California bearing ratio (CBR) test], stiffness test (resilient modulus conducted on wet and dry samples), toughness and abrasion resistance tests (Los Angeles Abrasion and Micro-Deval), and frost susceptibility test (tube suction) are the most relevant for unbound aggregate pavement layers.

Extensive review of technical literature was conducted to identify the most important physical properties affecting aggregate strength, modulus, and deformation behavior in unbound and bound pavement layers. A summary of the findings on important physical properties from the literature review is presented here.

Mineralogy

Mineral composition of aggregates has a significant effect on the physical and chemical characteristics that ultimately govern the performance of UAB/subbase layers under loading. This is particularly true as far as degradation and polishing of aggregates resulting from interparticle friction is concerned. For example, calcareous aggregates, such as limestone and dolomite, show significantly lower resistance to particle degradation and polishing. Therefore, UAB/subbase layers constructed using these aggregates are likely to undergo significant changes in gradation during compaction and subsequently under traffic loading. Note that not many research studies have evaluated directly the effects of aggregate mineralogy on UAB/subbase performance. On the other hand, research studies have generally focused on evaluating the effects of aggregate physical characteristics influenced by mineralogy on performance. Woolf (1952) presented extensive data on the results from physical tests on road building aggregates. From his data, the effects of aggregate mineralogy on physical characteristics are clearly apparent. For example, the average reported loss by abrasion for granite was 4.3%, whereas the corresponding values for limestone and dolomite were 5.7% and 5.5%, respectively.

Particle Size Distribution and Fines Content

One of the primary variables in any laboratory testing of aggregate materials is the grain size distribution. Differences in aggregate gradations can often lead to significantly different behavior for the same aggregate type. This is the result of the different packing order and void distributions that play a crucial role in load carrying through particle-to-particle contact in an aggregate matrix. To control the gradation of an individual aggregate sample, sieving and size separation of the aggregate materials need to be undertaken based on washed sieve analysis. Gradation itself is a key factor influencing not only the mechanical response behavior characterized by resilient modulus, shear strength, and permanent deformation, but also permeability, frost susceptibility, erosion susceptibility, and so forth (Bilodeau et al. 2007, 2008).

Note that sieve shakers used to separate aggregate sizes based on dry sieving of the aggregate stockpiles can give erroneous size distributions. In a recent study, wet sieving results showed that the actual fines content (note that unless otherwise specified, “fines” in this synthesis refers to material finer than 0.075 mm or passing No. 200 sieve) of an aggregate sample was always higher than the target fines content during a blending operation (Tutumluer et al. 2009). This difference in achieved versus target fines content was attributed to the significant amount of fines that remained stuck to the surfaces of larger particles during dry sieving and contributed to changing the performance of the aggregate layer as a whole. For example, aggregate samples blended with targeted 0% fines (material passing sieve No. 200 or 0.075 mm) contained 4.4% fines for a limestone and 2.9% fines for an uncrushed gravel material (Tutumluer et al. 2009). Therefore, these fines had to be accounted for appropriately during study of the effects of fines on aggregate strength and deformation behavior.

Gradation and fines content are interconnected in their effects on strength and resilient and permanent deformation characteristics. For a dense-graded crushed aggregate base material having a 25-mm (1-in.) top size, Gray’s (1962) pioneering work indicated that maximum strength was achieved at a fines content of about 8%. As the maximum aggregate size increased, the optimum amount of fines that gave the maximum strength typically decreased. Using a directional modulus approach by changing the pulsing direction in repeated load triaxial tests, Tutumluer and Seyhan (2000) also determined an optimum fines content of 7% for a dense-graded crushed limestone aggregate base material.

Well-graded aggregates have been found to have higher resilient modulus values to the point at which the fines content of the mixture displaces the coarse particles and the properties of the fines dominates (Jorenby and Hicks 1986; Kamal et al. 1993; Lekarp et al. 2000a). Barksdale and Itani (1989) found a dramatic 60% reduction in the resilient modulus when the fines content was increased from 0% to 10%. Thom and Brown (1988) found that the effect of grading varied with the compaction level; when uncompacted, specimens with uniform grading accumulated the least permanent deformation, whereas the resistance to permanent deformation was similar for all gradations when the specimens were heavily compacted. Kamal et al. (1993) and Dawson et al. (1996) found the effect of grading to be more significant than the degree of compaction (DOC), with the densest mix having the highest permanent deformation resistance. Brown and Chan (1996) successfully reduced rutting in granular base layers by selecting an optimum aggregate material grading that maximized compacted density. These performance characteristics were demonstrated through experiments with two types of wheel tracking and the use of repeated load triaxial tests at the University of Nottingham in the United Kingdom.

Increasing the amount of fines in a mix reduces the permanent deformation resistance (Barksdale 1972, 1991; Thom and Brown 1988). Moreover, the type of fines (nonplastic or plastic fines) in an aggregate layer has been found to affect the performance significantly. The results of a recent Illinois Department of Transportation (DOT) field study, Experimental Feature IL 03-01, indicate that increased aggregate fines had a significant effect on their performance in working platform applications (IDOT 2005).

Bilodeau et al. (2009) identified, from a laboratory study conducted on the performance of unbound granular materials with six gradations and three aggregate sources commonly used in Canada, one fines-related volumetric parameter (termed fine fraction porosity, represented as a ratio between the total amount of voids in aggregate matrix to the total amount of voids if the entire matrix comprised coarse particles only) that described satisfactorily not only the mechanical performance but also the environmental stresses sensitivity of materials tested. Also identified from their study were the adapted (or optimized) gradation zones that ensured adequate overall performance of those three aggregate sources.

Particle Shape, Surface Texture, and Angularity

The gradation, shape, and hardness have a great influence on the mechanical behavior and the strength properties of aggregate particles in contact. In general, it is preferable to have somewhat equidimensional (cubical) and angular particles rather than flat, thin, or elongated particles (Barksdale et al. 1992). Aggregate gradation is also critical for achieving good packing and minimal porosity in an aggregate mix. The maximum size of aggregates, the size distribution, and the shape of the particles determine the packing density that can be derived with an aggregate sample, assuming sufficient compaction is provided. Angularity, a measure of crushed faces and sharpness of edges in an aggregate, is important because it determines the level of internal shear resistance that can be developed in the particulate medium. Round, uncrushed aggregates such as gravel, particularly with a smooth surface texture, tend to "roll" out from under traffic loads with low rutting resistance.

Increasing particle angularity and roughness increase the resilient modulus while decreasing the Poisson's ratio (Hicks and Monismith 1971; Allen and Thompson 1974; Thom 1988; Thom and Brown 1988; Barksdale and Itani 1989). The reported research indicates that aggregates made with uncrushed or partially crushed gravel particles have a lower resilient modulus than do those with angular crushed particles. This effect has been attributed to the higher number of contact points in crushed aggregates, which distribute loads better and create more friction between particles (Lekarp et al. 2000a).

Allen (1973) and Barksdale and Itani (1989) investigated the effects of the particle surface characteristics of unbound aggregates and found that angular materials resisted permanent deformation better than did rounded particles because of the improved particle interlock and higher angle of shear resistance between particles. Similarly, Thom and Brown (1988) observed that permanent deformation was primarily affected by visible roughness of particles. Barksdale and Itani (1989) also concluded that blade-shaped crushed particles are slightly more susceptible to rutting than are other types of crushed aggregate and that cube-shaped, rounded river gravel with smooth surfaces is more susceptible than is crushed aggregate.

In the base courses, although compaction is important from a shear resistance and strength viewpoint, the size, shape, angularity, and texture of coarse aggregates are as important in providing stability (Barksdale 1991). Field tests of conventional asphalt pavement sections with two different base thicknesses and three different base gradations showed that crushed-stone bases gave excellent stability because of a uniform, high degree of density and little or no segregation (Barksdale 1984). Rounded river gravel with smooth surfaces was found to be twice as susceptible to rutting as was crushed stone (Barksdale et al. 1989).

Based on a review of several studies, Janoo (1998) concluded that shape, angularity, and roughness have significant effect on base performance and there could be as much as 50% change in resilient modulus of base materials owing to geometric irregularities of coarse and fine aggregate particles. Saeed et al. (2001) showed a linkage between aggregate properties and unbound layer performance. That study showed that aggregate particle angularity and surface texture mostly affected shear strength and stiffness.

Rao et al. (2002) studied the impact of imaging-based aggregate angularity index variations on the friction angle of different aggregate types and reported an increase in aggregate performance when the percentage of crushed particles was increased. An increase in crushed materials beyond 50% significantly increased the friction angle obtained from rapid shear triaxial tests, indicating a higher resistance to permanent deformation accumulation. Coarse aggregate angularity provides rutting resistance in flexible pavements as a result of improved shear strength of the UAB. The interlocking of angular particles results in a strong aggregate skeleton under applied loads; whereas, round particles tend to slide by or roll past each other, resulting in an unsuitable and weaker structure. Later, Pan et al. (2006) prepared unbound specimens by blending six aggregate materials with uncrushed gravel and tested for resilient moduli. The modulus values of the aggregate specimens blended in different percentages were linked to the imaging-based shape indices. As the aggregate angularity and surface roughness increased, the resilient moduli were considerably improved, which was primarily because of the increased shear strength, with better aggregate interlock and frictional properties and the increased confinement levels expressed by higher bulk stresses.

The NCHRP 4-30A project, *Test Methods for Characterizing Aggregate Shape, Texture, and Angularity* (*NCHRP Report 555*), recommended the Aggregate Image Measurement System (AIMS) and the University of Illinois Aggregate Image Analyzer (UIAIA) as viable imaging systems for analyzing aggregate morphology and quantifying aggregate morphologic effects to influence strength and permanent deformation behavior of unbound aggregate materials (Masad et al. 2007). Using the UIAIA system, Uthus et al. (2007) studied the aggregate morphologic property changes resulting from the rounding of aggregate particles in a ball mill drum. For cubical aggregates, the changes in angularity and surface texture appeared to have a significant effect on the elastic and plastic aggregate shakedown threshold limits, which will be discussed in more detail in chapter four. Tutumluer and Pan (2008) reported that aggregate blends comprising angular, rough particles consistently showed lower permanent deformation accumulations when studied using the UIAIA system. The angularity property was found to contribute mainly to the strength and stability of aggregate structure through confinement, whereas the surface texture property tended to mitigate the dilation effects through increasing friction between individual aggregate particles.

A recent study (Gates et al. 2011) sponsored by the FHWA conducted an interlaboratory study using the recently improved Aggregate Image Measurement System 2 (AIMS2) device. Analyzing results obtained across 32 laboratories, the study concluded that aggregate size and shape properties determined using the AIMS2 device showed reasonable coefficients of variation for all aggregate particle sizes greater than 0.075 mm. Findings from the study have led to increased use of the AIMS2 device as an automated device capable of providing objective and reproducible shape characterization of aggregates.

Degree of Compaction

Before the aggregate samples are tested for strength, modulus, and deformation behavior, the first task is to compact them at the corresponding gradation to determine their moisture–density relationships. Because pavement layers in the field often are compacted to predetermined percentages of the maximum dry density (MDD) values, it is important to establish the values of MDD and optimum moisture content (OMC) for each aggregate gradation. Thus, the objective of compaction is to improve the engineering properties of the soil mass. Through compaction, strength can be increased, deformation tendency can be reduced in the field, bearing capacity of the granular layer can be improved, and undesirable volume changes (such as those caused by frost action, swelling, and shrinkage) may be controlled (Holtz 1990).

Compaction methods applied on aggregate samples have a considerable effect on the moisture–density relationship for determining MDD and OMC. Commonly, an impact type compaction effort (similar to Proctor compaction) is applied on aggregate samples using the methods specified in the AASHTO T 99 Standard and AASHTO T 180 Modified test procedures (also ASTM D 698 and D 1557). The MDD values obtained from impact-hammer based methods, such as the AASHTO T 99 and AASHTO T 180, are subsequently corrected, as per AASHTO T 224, to compensate for particles larger than 19.0 mm (¾ in.). Note that other laboratory compaction procedures, such as the vibratory and gyratory compaction techniques, have been shown to be more realistic for providing adequate modulus and strength in laboratory-compacted samples and simulating properly field loading and applied stress conditions under vibratory rollers (Adu-Osei et al. 2000). Although the use of vibratory compaction for establishing the compaction characteristics of granular soils is covered under ASTM D 7382, no such specification is provided by AASHTO. Kaya et al. (2012) compared the effects of two different compaction methods (impact compaction and vibratory compaction) on the mechanical behavior of UAB materials. Comparing the gradation of aggregate specimens before and after compaction, Kaya et al. observed that impact compaction caused a change in aggregate gradation through crushing and particle breakage. This ultimately resulted in an increase in the OMC value. No such particle crushing and resulting change in gradation were observed for specimens prepared using the vibratory compaction method. Although the vibratory compaction method resulted in higher CBR values, the resilient modulus (M_R) values for specimens prepared using impact compaction were consistently higher, except for one aggregate type.

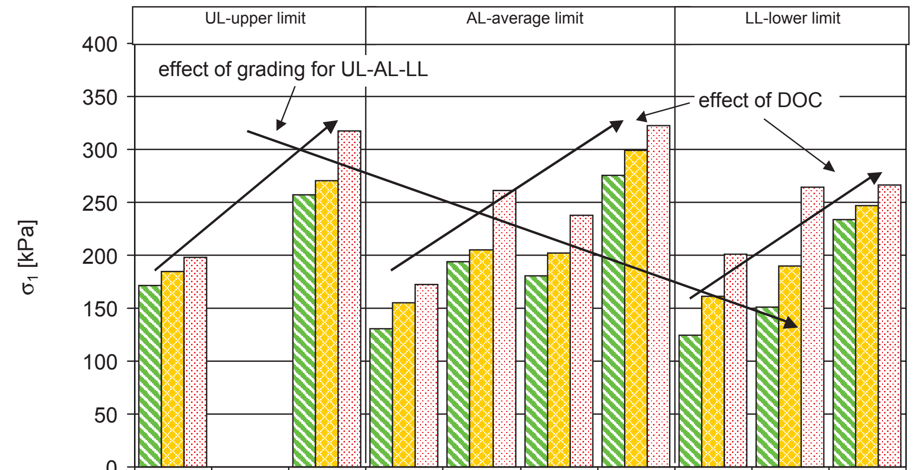
Density is used in pavement construction as a QC measure to help determine the compaction level of the constructed layers. Holubec (1969) found that increased density improves properties of unbound aggregates with angular particles more than for aggregates with rounded particles, provided there is no increase in the pore pressure during repetitive loading. Generally, increasing the density of a granular material makes the aggregate layer stiffer and reduces the magnitude of the resilient and permanent deformation response to both static and dynamic loads (Seyhan and Tutumluer 2002). Although some have found the research on density to be ambiguous with regard to the resilient behavior of soils causing little change in the resilient modulus (Knutson and Thompson 1977; Elliott and Thornton 1988; Lekarp et al. 2000), others have found that there is a general increase in the resilient modulus with increasing density (Rowshanzamir 1995; Tutumluer and Seyhan 1998).

The impact of density appears to be larger on the permanent deformation behavior of aggregates. Decreased density, as measured by DOC, substantially increases permanent deformation. Barksdale (1972) found that decreasing the DOC from 100% to 95% of maximum dry density increased permanent axial strain by 185% (on average). Increasing density from the standard Proctor to the modified Proctor maximum density decreased permanent deformation 80% for crushed limestone and 22% for gravel (Allen 1973). The DOC was reported as the most important factor controlling permanent deformation development by Van Niekerk (2002), who observed that 50% to 70% higher axial stresses were needed to cause similar magnitude of permanent deformation when the DOC increased from 97% to 103% for the investigated gradations (see Figure 6). Note that in Figure 6, “UL,” “LL,” and “AL” refer to the finest allowable grading, the coarsest allowable grading, and the average of upper and lower limits, respectively.

Moisture Content

Moisture has been widely recognized to adversely affect the performance of unbound aggregate layers in pavement structures and can affect aggregates in three different ways: (1) make them stronger with capillary suction, (2) make them weaker by causing lubrication between the particles, and (3) reduce the effective stress between particle contact points resulting from increasing pore water pressure, thus decreasing the strength.

Holubec (1969) conducted repeated load triaxial tests on crushed aggregates and gravel sands over a range of moisture contents. He reported an increase in permanent deformation by 300% for crushed aggregates and 200% for gravel sands when the moisture content was increased by 2.8% and 3.6%, respectively. Thompson and Robnett (1979) and Dempsey (1982) found that open-graded aggregates did not develop pore pressures upon loading, but uniformly graded dense



	UL-97%	UL-100%	UL-103%	AL-97%	AL-100%	AL-103%	AL-105%	LL-97%	LL-100%	LL-103%
1%-N=1000000	171		257	130	194	180	276	125	151	234
5%-N=1000000	184		270	155	205	202	299	162	190	247
10%-N=50000	198		317	172	261	238	323	201	264	266

Degree of compaction [%] and grading (UL-AL-LL)

FIGURE 6 Stress (σ_1) levels at which ε_p = 1%, 5%, and 10% at $N = 10^6$, 10^6, and 50,000, respectively, at DOC 97%, 100%, 103%, and 105% (Van Niekerk 2002).

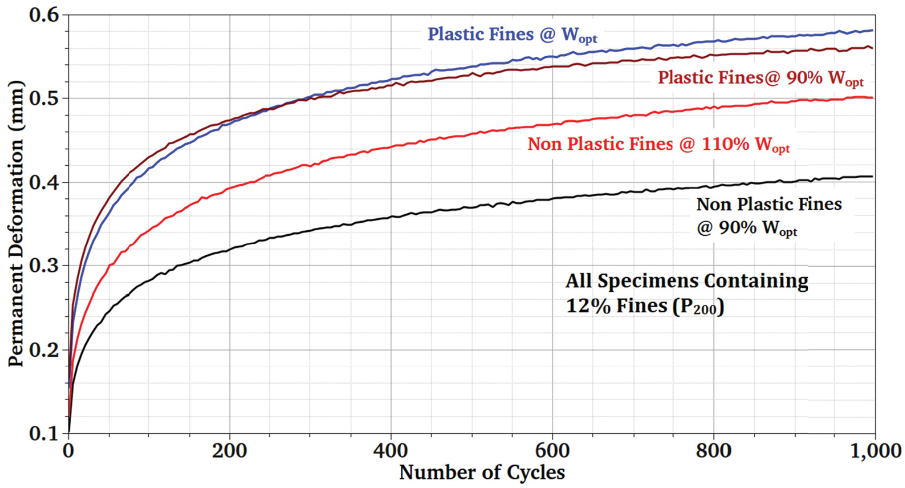
aggregates with higher fines contents did develop pore pressures that resulted in a reduction in resilient modulus values. Thom and Brown (1987) found that no noticeable pore water pressures developed below 85% saturation and that most of the reduction in resilient moduli was the result of the lubricating effect of the water. It can also be assumed that increasing the water content in a soil reduces the capillary suction between particles, thus decreasing the effective stress and the resilient moduli. Therefore, moisture can have a positive effect on unbound granular materials as long as the moisture increases the capillary suction between particles. Once the saturation reaches a point at which it reduces the capillary suction, the moisture becomes a detriment to preventing residual deformation and can cause a lubricating effect. At even higher saturation levels, where excess pore water pressure can develop and reduce the effective stress, the rutting resistance can decrease dramatically, resulting in deeper ruts (Thom and Brown 1987). Maree et al. (1982) conducted Heavy Vehicle Simulator (HVS) tests on pavements with untreated granular bases and reported higher permanent deformation for layers with higher moisture contents. Moreover, he observed that "unstable"' conditions in unbound aggregates were triggered at lower values of stress ratio (defined as the ratio of applied stress to aggregate shear strength) when the degree of saturation was increased.

Degree of saturation is a factor that reflects the combined effect of density and moisture content. The resilient modulus is strongly correlated with degree of saturation (Thompson and Robnett 1979). Based on the comprehensive subgrade soil resilient modulus testing study, Thompson and LaGrow (1988) proposed using the following "moisture adjustment" factors to adjust resilient modulus values for moisture contents in excess of optimum. For example, resilient modulus of a silt loam soil may decrease approximately 1,500 psi for a 1% increase in moisture content (Thompson and Robnett 1979).

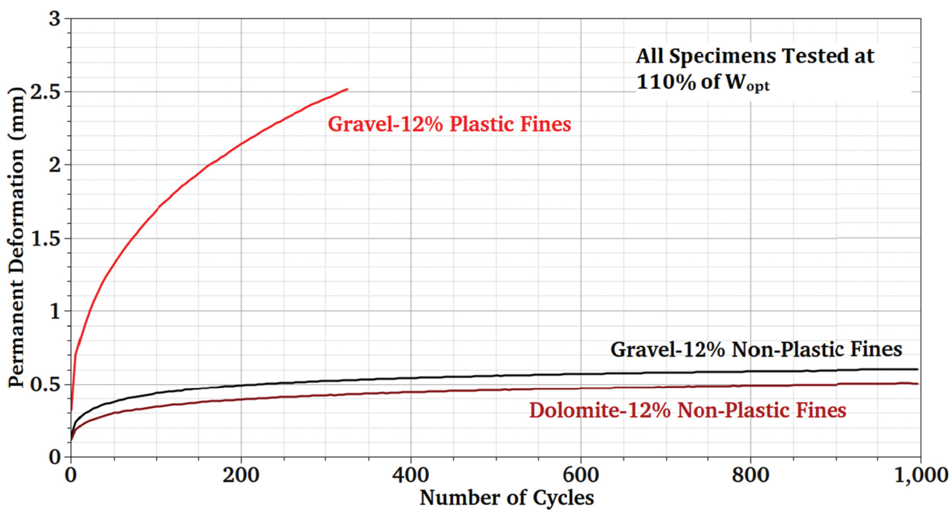
Wetting up from a shallow groundwater table (GWT) by capillarity or by increase in the GWT level reduces suction and may cause a constructed unbound pavement layer to deform permanently. Moisture sensitivity varies depending on specified gradations and the amount and plasticity index (PI) of the fines: that is, percent passing No. 200 sieve (P_{200}). Tutumluer et al. (2009) compared relative impacts of molding (as-compacted) moisture content and plasticity of fines on the permanent deformation behavior of crushed (dolomite) and uncrushed (gravel) aggregate materials with P_{200} = 12% (see Figure 7). A drastic reduction in aggregate performance can be seen when plastic fines are combined with increased molding moisture: that is, compare permanent deformation of gravel at 110% of the optimum moisture content (w_{opt}) with plastic and nonplastic fines in Figure 7b. Accordingly, the specification limits for compaction moisture content are best based on accumulated permanent deformation.

Key Lesson

The following factors have been identified as primarily affecting UAB/subbase layer performance under loading: (1) aggregate mineralogy, (2) gradation, (3) fines content (material passing No. 200 sieve), (4) type of fines (plastic or nonplastic), (5) particle shape, texture and angularity, (6) DOC, and (7) moisture content.



(a)



(b)

FIGURE 7 Relative effects of varying moisture content and plasticity of fines on permanent deformation behavior of crushed dolomite and uncrushed gravel aggregates (Tutumluer et al. 2009).

TESTS TO CHECK AGGREGATE QUALITY FOR PAVEMENT APPLICATIONS

Background

An extensive review of published literature indicates the previously discussed properties are critical in governing the performance of UAB and subbase layers in pavement systems. Accordingly, agency specifications for aggregate usage in pavement base/subbase applications often include requirements related to gradation (particle size distribution), degree of crushing (100% crushed, 100% uncrushed, number of fractured faces), plasticity (liquid limit and plasticity index), durability, and soundness (Barksdale 1991). Commonly used specifications include those developed by ASTM, AASHTO, the U.S. Army Corps of Engineers (USACE), and individual state and provincial transportation agencies.

Based on their underlying philosophy, material specifications can be divided into the following four general categories: (1) methods or "recipe" specifications, (2) proprietary product specifications, (3) performance specifications, and (4) end result specification-statistically based. Among these different specification categories, end result specifications commonly are employed for aggregate usage in pavement base/subbase layer applications. Discussions on the other specification types are presented elsewhere (Barksdale 1991).

AASHTO specification M 147-65, Materials for Aggregate and Soil-Aggregate Subbase, Base, and Surface Courses, suggests several tests for sampling and testing of aggregates before their use in pavement applications. Different tests recommended by AASHTO for material quality testing, selection, and control testing of aggregates are listed here:

- AASHTO T 2: Standard Method of Test for Sampling of Aggregates
- AASHTO T 11: Standard Method of Test for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing
- AASHTO T 19: Unit Weight and Voids in Aggregate
- AASHTO T 27: Standard Method of Test for Sieve Analysis of Fine and Coarse Aggregates
- AASHTO T 84: Specific Gravity and Absorption of Fine Aggregate
- AASHTO T 85: Specific Gravity and Absorption of Coarse Aggregate
- AASHTO T 88: Standard Method of Test for Particle Size Analysis of Soils
- AASHTO T 89: Standard Method of Test for Determining the Liquid Limit of Soils
- AASHTO T 90: Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils
- AASHTO T 96: Standard Method of Test for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
- AASHTO T 104: Soundness of Aggregate by Use of Sodium or Magnesium Sulfate
- AASHTO T 112: Clay Lumps and Friable Particles in Aggregate
- AASHTO T 113: Lightweight Pieces in Aggregate
- AASHTO T 146: Standard Method of Test for Wet Preparation of Disturbed Soil Samples for Test
- AASHTO T 176: Standard Method of Test for Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test
- AASHTO R 58: Standard Practice for Dry Preparation of Disturbed Soil and Soil-Aggregate Samples for Test
- AASHTO T 210: Aggregate Durability Index
- AASHTO T 248: Reducing Field Samples of Aggregate to Testing Size
- AASHTO T 255: Total Moisture Content of Aggregate by Drying

Similarly ASTM specification D 2940 Standard Specification for Graded Aggregate Material for Bases or Subbases for Highways or Airports (ASTM D 2940 2009) specifies the following test methods to evaluate the quality of aggregates for use in pavement base and subbase layers:

- ASTM D 75: Standard Practice for Sampling Aggregates
- ASTM C 136: Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates
- ASTM D 422: Grain-Size Analysis (Wet Sieving and Determination of Subsieve Size Fractions, by Hydrometer Analysis)
- ASTM D 4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D 2419: Standard Test Method for Sand Equivalent Value of Soils and Fine Aggregate
- ASTM D 4792: Standard Test Method for Potential Expansion of Aggregates from Hydration Reactions

The following test methods have been used by agencies for characterizing the toughness/abrasion resistance of aggregates:

- Los Angeles abrasion (AASHTO T 96)
- Aggregate impact value (British)
- Aggregate crushing value (British)
- Micro-Deval abrasion (AASHTO T 327)—coarse and fine aggregates
- Degradation in the SHRP Superpave® Gyratory Compactor

Similarly, the following test methods are used to characterize the soundness and durability of aggregates:

- Sodium and magnesium sulfate soundness tests (AASHTO T 104)
- Freezing and thawing soundness (AASHTO T 103)
- Aggregate durability index (AASHTO T 210)
- Canadian freeze-thaw test

Wu et al. (1998) evaluated different toughness/abrasion resistance as well as durability/soundness tests for characterizing aggregates used in asphalt concrete. Testing aggregates from sources with poor to good performance histories and correlating the laboratory test results with field performance, they concluded that the Micro-Deval Abrasion and Magnesium Sulfate soundness tests provided the best correlation with field performance. The survey of state and Canadian provincial transportation agencies conducted under the scope of this synthesis study aimed to assess the state of practice in aggregate quality checking before their use in UAB and subbase layer construction.

Current Practices on Tests to Check the Quality of Aggregate Sources

Figure 8 shows the relative distributions of different test methods used by state transportation agencies to check the quality of virgin aggregates for use in UAB/subbase layers. Forty-three of 46 respondents use sieve analysis as the primary method of aggregate quality check for virgin aggregate sources. Moreover, sodium sulfate/magnesium sulfate (Na_2SO_4/$MgSO_4$) soundness test, some form of abrasion tests (Los Angeles abrasion or Micro-Deval), and percent deleterious materials were also found to be common practices among agencies. Some transportation agencies also use tests, such as absorption and specific gravity, Atterberg limits, and state-specific degradation tests for checking the quality of aggregate sources.

Frequency of Checking Aggregate Sources for Quality

The survey of state and Canadian provincial transportation agencies also gathered information on the frequencies of

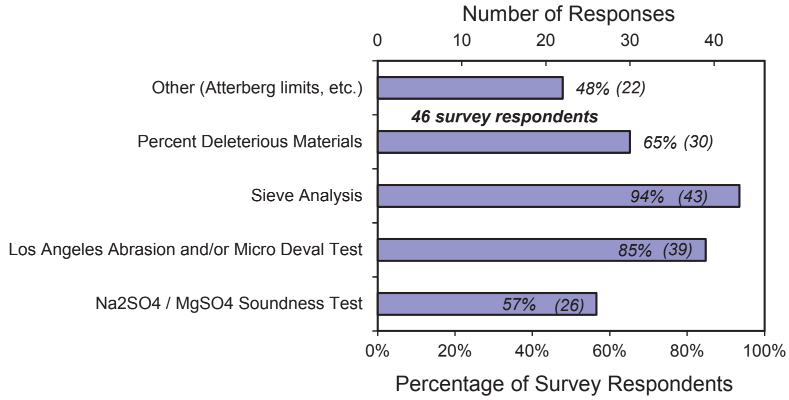


FIGURE 8 Tests used by agencies for evaluating quality aspects of virgin aggregate materials for pavement base and subbase applications (46 respondents).

quality assurance tests on virgin aggregate materials. Results from the survey are presented in Figure 9.

Apart from the testing frequencies shown in Figure 9, several other agencies also reported policies for aggregate material quality testing based on the quantity of aggregate used in a particular project. For example, two states reported requirements for conducting at least one quality assurance check for every project per every 2,000 and 2,500 tons of aggregate used, respectively.

Crushed versus Uncrushed Aggregates

Particle shape and angularity, often expressed as "crushed" or "uncrushed" particles, play an important role in governing the behavior of unbound aggregate layers under loading. Aggregate layers with uncrushed particles undergo significant particle reorientation under loading, thus accumulating large amounts of permanent deformation, which ultimately may lead to internal shear failure. A recent study at the University of Illinois (Mishra 2012) evaluated the effects of particle shape and angularity on unsurfaced pavement performance. Through laboratory testing and accelerated loading of full-scale unsurfaced pavement test sections, the study highlighted the increased potential for internal shear failure within uncrushed aggregate layers. It is therefore important for transportation agencies to distinguish between crushed and uncrushed aggregates while developing material specifications for aggregates to be used in base and subbase layers. Continued research on the quantification of aggregate particle shape, surface texture, and angularity indices through imaging-based methods may lead to the establishment of an aggregate packing index representing the degree of particle interlock in aggregate matrices. Such a packing index potentially could highlight the differences between uncrushed and crushed particles as far as packing within in aggregate matrix and load transfer mechanisms are concerned.

An equal number of agencies (20 each of 46 respondents) replied "Yes" or "No" when asked whether uncrushed aggregates were allowed in UAB and subbase layers. The remain-

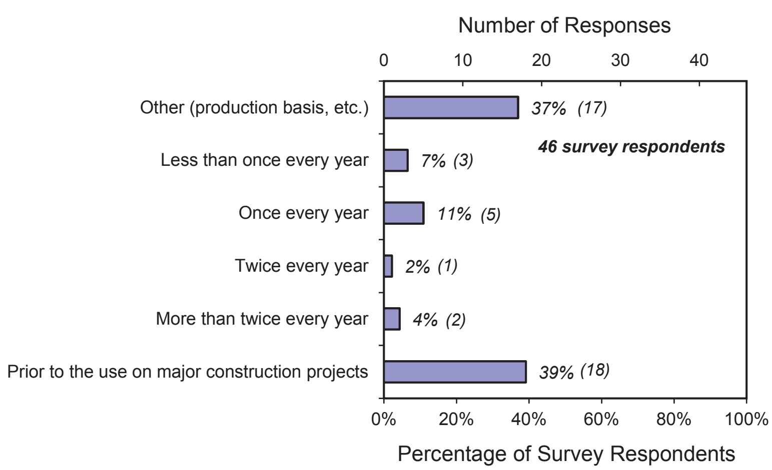


FIGURE 9 Frequencies of aggregate acceptance checks by state transportation agencies (46 respondents).

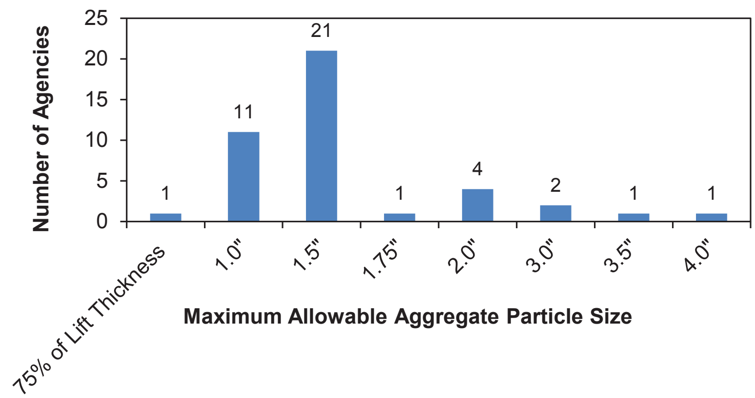


FIGURE 10 Base layer dense-graded unbound aggregate: maximum aggregate particle size allowed by agencies.

ing six agencies require partially crushed particles for base course applications (often by requiring at least one fractured face or by specifying a minimum proportion of fractured particles in the aggregate blend). Moreover, several agencies allow the use of uncrushed aggregates in subbase layers but prohibit their use in base courses.

Maximum Allowable Aggregate Particle Size

Particle size distribution or gradation has been found to be the most important parameter affecting aggregate performance in unbound and bound pavement layers. State and Canadian provincial transportation agencies were surveyed for the maximum aggregate particle sizes allowed in different types of aggregate base and subbase layers, and their responses are reported in Figures 10 to 13.

As shown in Figures 10 to 13, no consistent practice exists among transportation agencies regarding the maximum aggregate particle size allowed in UAB and subbase layers. Nevertheless, most of the respondents reported similar maximum aggregate particle size limits for dense-graded base and subbase layers. For example, 32 agencies limit the maximum aggregate particle size for dense-graded base courses to 1.0 or 1.5 in. Similarly, 20 agencies limit the maximum aggregate particle size in dense-graded subbase layers to between 1.5 and 2.0 in. In general, larger top-size aggregates are allowed in dense-graded subbase layers compared with those in dense-graded base layers. No such clear trend was observed from comparing the specifications for open-graded drainage base and subbase layers.

Type and Amount of Fines

The type of fines, often indicated by the PI value (PI test usually conducted on material finer than 0.425 mm or passing No. 40 sieve), plays an important role in governing the shear strength, resilient modulus, and permanent deformation behavior of unbound aggregate layers in pavement structures. As mentioned, unless otherwise specified, the term “fines” in the current synthesis refers to material finer than 0.075 mm or passing the No. 200 sieve. Aggregate materials with high amounts of plastic fines exhibit higher moisture susceptibility and undergo significant reduction in the shear strength in the presence of moisture when compared with aggregates with nonplastic fines. Recent research at the University of Illinois

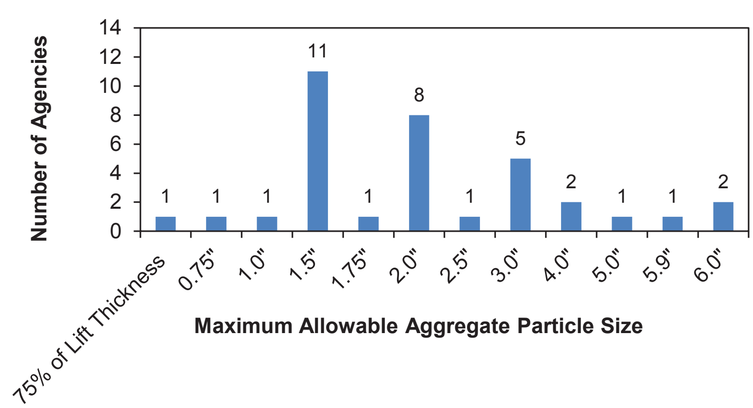


FIGURE 11 Subbase layer dense-graded unbound aggregate: maximum aggregate particle size allowed by agencies.

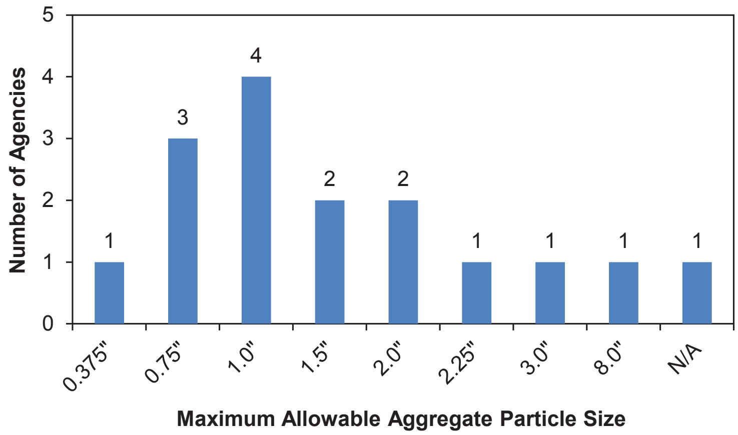


FIGURE 12 Base layer open-graded drainage: maximum aggregate particle size allowed by agencies.

(Mishra et al. 2010a, 2010b; Mishra and Tutumluer 2011; Mishra 2012) has established the increased moisture susceptibility of aggregates with high amounts of plastic fines, and thus has emphasized the importance of specifying different values for the maximum allowable fines contents for aggregates with nonplastic and plastic fines. Accordingly, the current synthesis study gathered information on the state of the practice regarding the maximum amounts of nonplastic and plastic fines allowed in an aggregate matrix. Only one agency (Maryland) currently specifies different threshold limits for the maximum amount of plastic and nonplastic fines allowed in aggregates to be used in pavement construction. It is important to note that some agencies may not consider differentiating between plastic and nonplastic fines. This is because with adequate production, storage, and construction practices the amount of plastic fines in an aggregate material usually can be controlled. For crushed stones produced from quarry operations, the fines (material passing No. 200 sieve) usually are nonplastic in nature although the nature of fines also depends on the mineralogy of the parent rock. With proper storage, sampling, and transportation practices, contamination of the stockpiles with natural soil and corresponding plastic fines can be controlled adequately. Therefore, aggregate materials used in construction of UAB/subbase courses may primarily comprise nonplastic fines. Accordingly, the lack of differentiation between plastic and nonplastic fines in UAB/subbase courses may not always indicate poor practice. Rather, it should be emphasized that in cases in which the aggregate material may comprise plastic fines, it is critical to control the maximum amount of fines allowed in the aggregate matrix because plastic fines in the presence of moisture often lead to significant deterioration in the aggregate shear strength.

As discussed, researchers and practitioners in the past have established that unbound aggregate materials containing the optimum amount of fines (material passing No. 200 sieve or finer than 0.075 mm) perform the best as far as shear strength, resilient modulus, and permanent deformation characteristics are concerned. Insufficient fines in an aggregate matrix results in unstable matrix behavior because of

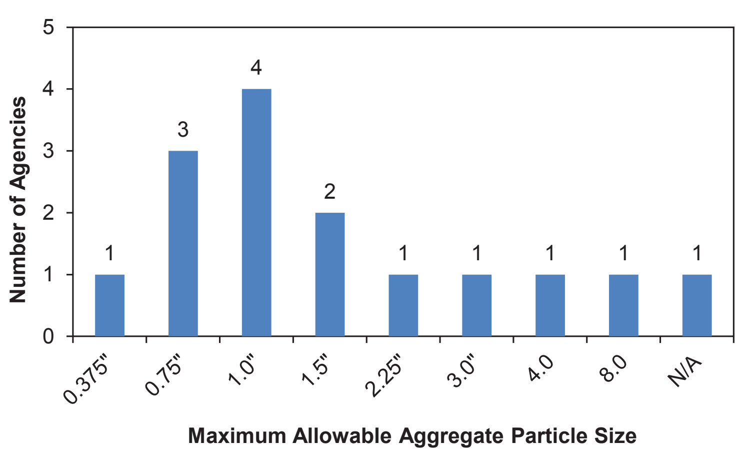


FIGURE 13 Subbase layer open-graded drainage: maximum aggregate particle size allowed by agencies.

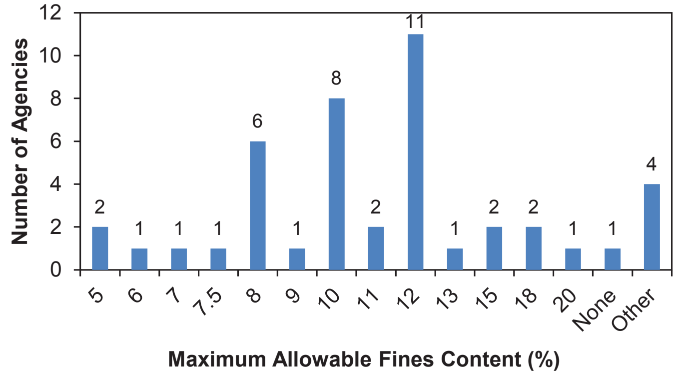


FIGURE 14 Base layer dense-graded aggregate base: maximum allowable fines content in aggregates used (44 respondents).

the excessive movement of the coarse particles with respect to each other. On the other hand, the presence of excessive fines in an aggregate matrix compromises particle interlock through lubricating action at the contact points. This leads to the aggregate material exhibiting lower shear strength and resilient modulus values and accumulating large permanent deformations. Thus, it is critical for transportation agencies to control the amount of fines in aggregates used in pavement applications.

Figures 14 to 16 summarize the survey responses concerning the maximum amount of fines allowed in different UAB and subbase layer types. There is a wide variation in the maximum allowable fines contents from one agency to the other. Although 33 of 46 respondents restrict the amount of fines (P_{200}) in dense-graded base courses to less than 12%, five agencies reported allowing more than 15% fines. One agency (Georgia) currently specifies different limits for the maximum allowable fines (P_{200}) contents for different aggregate types (11% and 15% for silicate and carbonate rocks, respectively). Another agency is in the process of modifying its state specifications to impose a restriction on the maximum allowable fines content.

Figures 17 and 18 show the maximum value of PI allowed by agencies for the fines fraction (P_{200}) in dense-graded UAB and subbase courses, respectively. As shown in the two figures, PI = 6 is commonly used as the maximum PI value for the fines fractions in UAB and subbase layers. Moreover, it is important to note that three agencies do not impose any restrictions on the plasticity of fines in the aggregates used for constructing aggregate base and subbase layers. In addition, one transportation agency allows the use of aggregates with fines fraction PI as high as 15. Such high plastic fines, when present in large amounts in an aggregate matrix, may render the aggregate highly moisture susceptible, thus significantly reducing the aggregate shear strength in the presence of moisture.

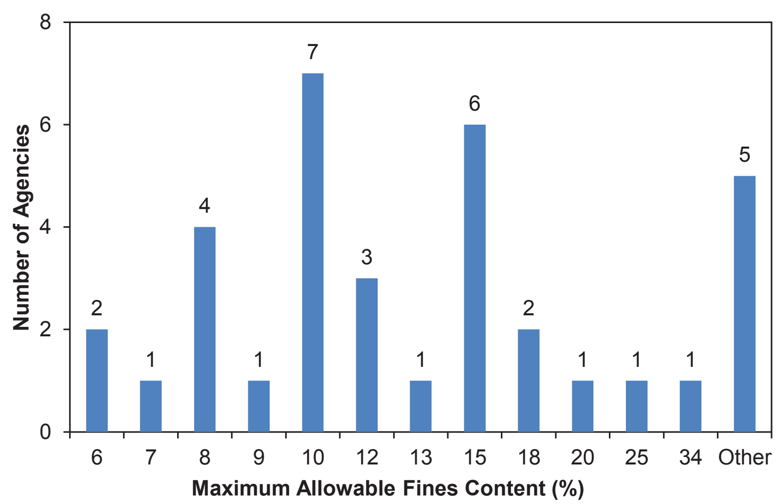


FIGURE 15 Subbase layer dense-graded aggregate subbase: maximum allowable fines content in aggregates used (35 respondents).

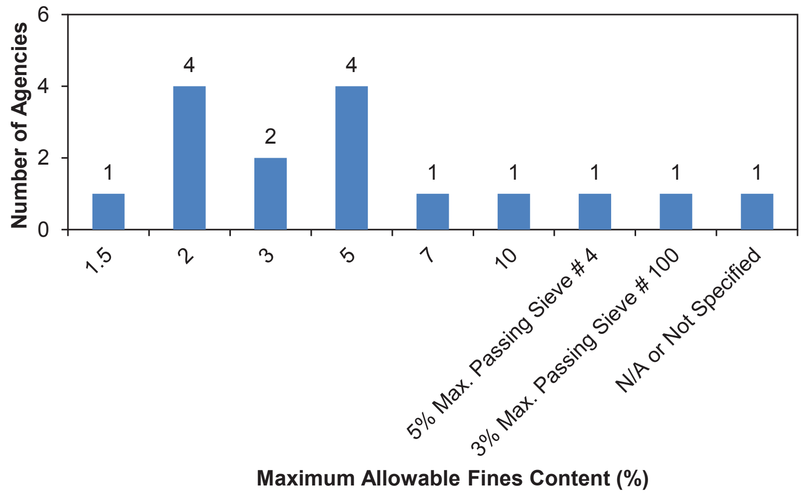
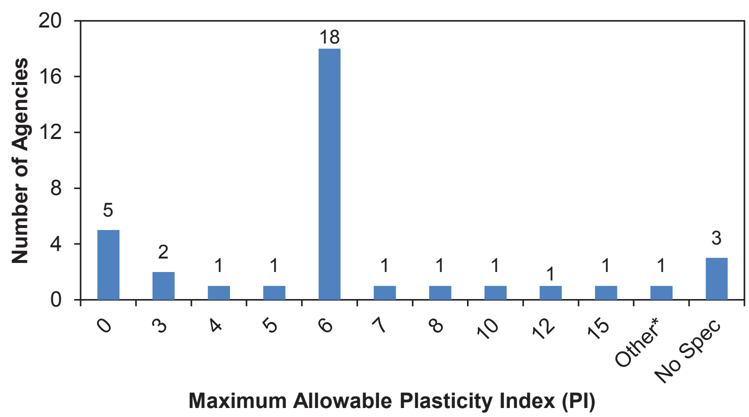


FIGURE 16 Base/subbase open-graded drainage layers: maximum allowable fines content in aggregates used (17 respondents).



*Other: PI = 6 or graded aggregate base and Coquina base; PI = 9 for sand-clay base

FIGURE 17 Maximum value of PI allowed for the P_{200} fines fraction in dense-graded UAB.

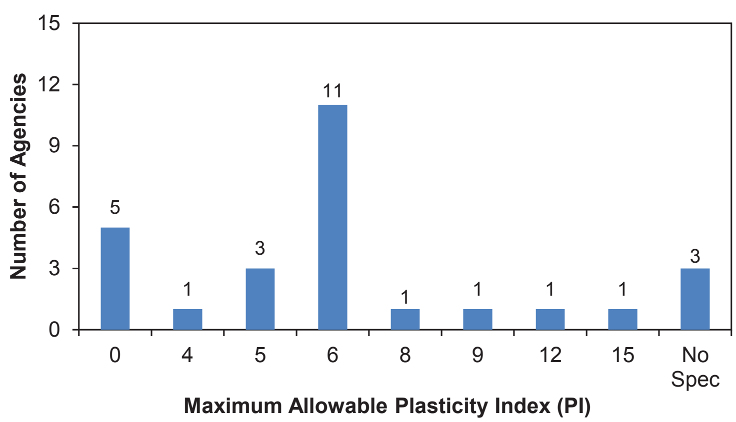


FIGURE 18 Maximum value of PI allowed for the P_{200} fines fraction in dense-graded UAB.

Key Lessons

- No common practice exists among transportation agencies as far as testing of aggregates for material quality before their use in unbound base/subbase layers is concerned.
- The use of 100% uncrushed aggregates in UABs/subbase layers is best done with caution, taking into account their substandard strength properties and high rutting damage potentials.
- No common practice exists concerning the maximum aggregate particle size allowed in different unbound aggregate pavement layers. To ensure ease of construction and adequate compaction, the maximum particle size allowed in UAB, subbase, and drainage layers are best restricted to 1.5 in., 2 in., and 4 in., respectively.
- Excessive fines (P_{200}) deteriorate aggregate layer performance, especially in the presence of moisture. The maximum amount of fines (P_{200}) allowed in UAB/subbase layers are best restricted to 12% or less.
- The presence of plastic fines in an aggregate layer needs to be limited. For instances in which the presence of plastic fines is unavoidable, different threshold limits are best set for the maximum allowable fines content for nonplastic and plastic fines.

SUSTAINABLE PRODUCTION AND UTILIZATION OF AGGREGATES

Aggregate crushed stone quarry processes such as blasting, crushing, and screening of coarse aggregates produce by-product materials, at approximately 8% of the mined aggregate, commonly known as quarry waste or quarry dust. Quarry wastes typically are less than 0.25 in. (6 mm) in size and consist of coarse, medium, and fine sand-sized particles and a clay/silt-sized fraction, which is less than No. 200 sieve (0.075 mm) in size. Current economic conditions and an increased emphasis in the construction industry on sustainability and recycling require production of aggregate gradations with lower dust and smaller maximum sizes. These new production limitations have "unbalanced" the aggregates production stream, in part because of the demand for cleaner aggregates with smaller top sizes in increased utilization of finer asphalt concrete mixes, resulting in excessive energy use and increased waste fines. Owing to these increased energy and disposal costs for the aggregate production, reusing and recycling of waste products (e.g., RAP, and RCA) may sometimes exceed the potential economic benefits. If research can lead toward more beneficial use of the by-product quarry wastes, making more effective use of locally available materials through beneficiation and use of marginal aggregate materials and increasing effective use of recycled materials, aggregate production can become more sustainable through energy conservation and efficient use of aggregate resources. Mineralogical properties of the parent rock and the type of crusher greatly influence the amount of fines produced during quarry operations. From a report published in the United Kingdom, depending on the type of crusher, limestone quarrying may produce as much as 25% fines. Filler materials that are less than 0.075 mm in size typically account for 10% to 20% of the total crushed rock aggregate fines produced (Manning and Vetterlein 2004). A more recent review article from *Stone, Sand and Gravel Review* on utilization of quarry fines for sustainable construction cites a number of successful applications for alternative uses of quarry fines in construction applications (Halmen and Kevern 2010). Among them, the ones linked to pavement applications with the most potential to utilize large amounts of quarry fines are listed in the order of ranking as follows:

1. Pavement working platform, also known as aggregate subgrade/granular subbase construction;
2. Unbound aggregate for road base and as embankment material;
3. Fine aggregate/filler for controlled low-strength materials (aka flowable fill);
4. Filler material in hot mix asphalt (HMA) and slurry seals for asphalt pavements;
5. Fine aggregate/filler for PCC, such as use of manufactured sand in PCC with higher fines content according to the ongoing work at the ICAR by Fowler and co-workers (http://aftre.nssga.org/Reports/Project-102-1.pdf).

FHWA also published and updated user guidelines for use of by-products in pavement construction, in which they identified flowable fill as a possible application for the use of quarry fines among many other applications (FHWA 2008).

Several studies in the literature report successful utilization of quarry fines in road base/embankment and flowable fill applications. Preliminary studies conducted at the University of Texas indicated that quarry fines could be used economically as flowable fill and in cement-treated pavement subbases (Kumar and Hudson 1992; Hudson et al. 1997). Based on these findings, a recent ICAR research study tested the acceptability of high fines content in aggregate pavement layers and reported that aggregate systems with higher fines benefited considerably from low percentages (1% to 2%) of cement stabilizer (Ashtiani and Little 2007). The study found that with the proper design of fines content, cement content, and moisture, the performance of the stabilized systems with high fines content could perform equivalent to or better than systems with standard fines content. Cement-treated quarry fines also were used as a pavement base material on SH-360 in Arlington, Texas, as part of a research project (Puppala et al. 2008). The study reported that the unconfined compressive strength of cement-treated quarry fines was adequate and that field monitoring indicated low permanent deformation during service. A recent Iowa DOT study also focused on road construction using admixture stabilized limestone fines and found that stabilized fines could perform satisfactorily as a structural layer in road construction through visual observations (Rupnow et al. 2010). Laboratory compaction, unconfined compression, freezing and thawing, and wet-dry durability test results showed that cement kiln dust (CKD) was not an acceptable stabilizer because of poor durability performance; however, class C fly ash and CKD mixtures were determined to be acceptable.

Mishra and Tutumluer (2011) characterized the strength, stiffness, and deformation behavior of different types and qualities of aggregates commonly used in Illinois for subgrade replacement and subbase. The project focus was on establishing aggregate cover thickness correlations with aggregate material properties to modify and improve the current IDOT *Subgrade Stability Manual* thickness requirements through laboratory and field testing. Thick layers of the uncrushed gravel placed over a weak subgrade were observed to undergo internal shear failure owing to high amount of fines and excessive movement of the aggregate particles. On the other hand, crushed aggregate layers showed significantly higher resistance to internal shear deformation, and test sections constructed using crushed aggregates failed primarily as a result of subgrade deformation. The influence of compactive effort on aggregate layer performance was clearly apparent; higher relative compaction exhibited better resistance to permanent deformation accumulation.

Prolonged exposure to moisture and freeze-thaw effects was found to have a beneficial effect on the crushed dolomite with high amounts (12% to 13%) of nonplastic minus No. 200 fines (Mishra and Tutumluer 2011). Interestingly, carbonate cementation within the fine fraction was identified as the most probable mechanism contributing to “stiffening” of the aggregate sections, which resulted in the aggregate layer sustaining a significantly higher number of load applications without undergoing shear failure. Note that upon further loading, the aggregate sections demonstrated “punching” failure into the underlying subgrade (CBR = 3%) similar to the failure of concrete slabs over weak support conditions.

Unbound Pavement Applications

Often different test methods are adopted by transportation agencies to check the adequacy of aggregate materials for use in unbound pavement applications. However, these “recipe-based” test methods are focused primarily on checking the physical and chemical (if applicable) properties of aggregates, which often are related to basic geologic origin, mineralogy, and other properties, such as hardness and durability, and they may not necessarily offer the best means to judge the mechanistic properties and performance of unbound aggregate layers. One major disadvantage associated with such physical classification systems is that they could possibly accept unsuitable materials in some cases and reject desirable materials in other cases, as summarized by Cook and Gourley (2002). Under such a physical classification framework, naturally occurring materials could be excluded from use, based on any combination of grading, plasticity, particle hardness, strength, and so forth lying outside the specification-demanded requirements, as outlined in Figure 19.

In many areas with a shortage of “standard” or traditional aggregate materials that satisfy normal requirements for road paving, nonstandard local aggregate sources have been successfully applied in low volume road constructions; typical examples are documented in Table 1. In addition, an early field trial constructed by the Transportation Research Laboratory in 1978, in which three marls (local calcareous materials) outside the recommended gradation envelope were substituted for the crushed stone base, indicated that the use of a much wider range of marls, if properly stabilized, is viable technically and economically, as justified by the low values of rut depth and deflection and the high strength of the base (Woodbridge 1999). Bullen (2003) also showed that the use of local aggregate materials in Australia, with appropriate design, can not only provide the desired pavement performance, but also can promote sustainability in terms of significant cost saving, natural resource conservation, and even environment protection.

In the United States, for instance, the taconite aggregate resources in Minnesota, the industrial by-products from iron ore mining, recently have been demonstrated in MnROAD low-volume test section studies to be a promising supply of high-quality, low-cost aggregates for roadway use (Clyne et al. 2010). In Texas, locally available materials (mostly Grade 4), sometimes even with high amount of fines, have been used (with or without stabilization) not only for low-volume roads but also for major roads in some districts.

Despite all of the potential benefits and documented successful applications of local aggregate sources, one major obstacle to their widespread use is the significant engineering uncertainty (or risk) inherent with their long-term performance. Such uncertainties cannot be addressed by current physical classification systems to be later considered properly in pavement design. Furthermore, several state transportation agencies are reluctant to relax the traditionally conservative standard specifications.

Separate from the physical classification presented previously, the mechanistic classification discerns different qualities of unbound aggregates from mechanical properties that are required as input to the constitutive relationships incorporated into mechanistic-empirical pavement design procedures, as illustrated in Figure 20. It is expected that such mechanistic classification systems, in combination with certain levels of local experiences, have direct relevance or even robust linkage to the actual performance of materials used

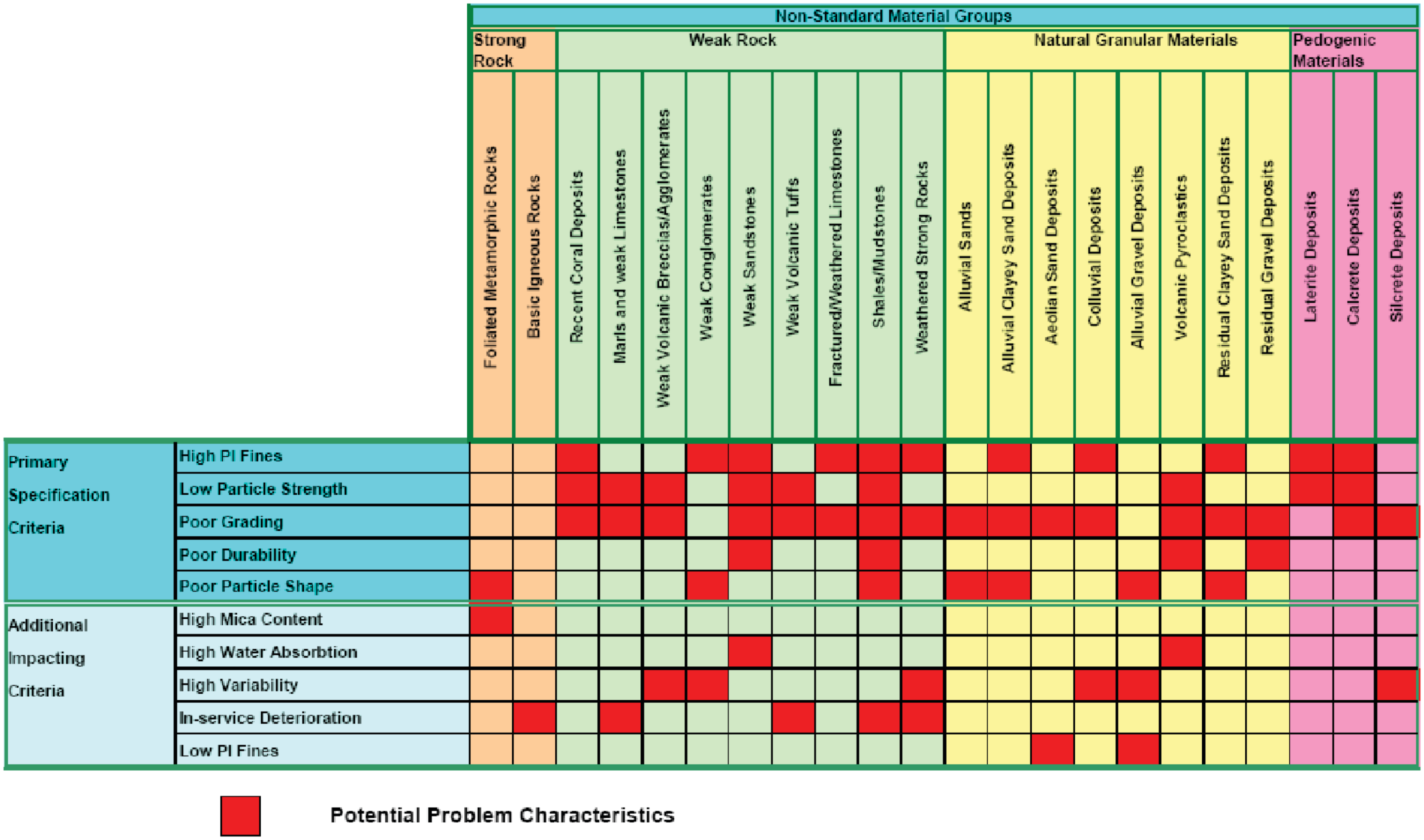


FIGURE 19 Nonstandard material groups and their likely problems (Cook and Gourley 2002).

TABLE 1
EXAMPLES OF USING NONSTANDARD MATERIALS IN LOW-VOLUME SEALED ROADS

Material & Reference	Location	Climatic Environment	Material Characteristics	Utilisation
Calcrete Lionjanga et al (1987) Greening and Rolt (1997)	Botswana	Semi-arid	Low particle strength Low compacted strength Poor grading High plasticity	Roadbase: Revised specifications developed for both sealed and unsealed shoulder designs. Successfully used as roadbase with acceptable performance (0.3 x 10^6 esa) for materials with soaked CBR >35% and PI <30 if shoulders are sealed.
Laterite Grace and Toll (1987) Gourley and Greening (1997) CIRIA (1988)	Malawi	Seasonally wet tropical	Low particle strength Low compacted strength Poor grading High plasticity	Roadbase: Construction procedure modified to allow traffic to run on roadbase for one rainy season before proof rolling, shaping and sealing in the following dry season. All sites well drained and with crown-height at least over 1m.
Marl Woodbridge et al (1987)	Belize	Wet humid tropical	Low particle strength Poor grading	Roadbase and sub-base: Embankment construction (600-750mm of fill) used throughout due to seasonally high water-table. Only non-plastic or slightly plastic materials selected. Controlled heavy compaction used to lock material and achieve >98% MDD. Good maintenance regime adopted including regular clearing of drains and unsealed shoulder maintenance.
Basalt Pinard & Jakalas, (1987).	Botswana	Sub-tropical	Crushed material (with added fines) passed specification criteria; but had demonstrably poor in-service durability.	Roadbase. Addition of plastic (active) fines to improve the grading along with modification using too low a percentage (below ICL) of lime (lime also suspect i.e. inactive) led to early failure due to moisture interaction/volumetric change in the road base material. Unsealed shoulder design.
Weathered Basalt Gourley and Greening (1999)	Botswana	Sub-tropical	Ripped weathered (Grade III+) basalt selected. Grading out of recommended specification; PI <12 and soaked CBR >55.	Roadbase: Normal construction methodology adopted. 1m embankment and sealed shoulders.
Coral Cardno & Davies (1994) Beavan (1971)	Papua New Guinea	Wet humid tropical	Low particle strength Poor grading (including oversize) High plasticity	Roadbase: Modified specification based on the requirement of high compaction giving dense layers (max. 150mm). Selection of appropriate compaction plant vital (a function of grading and PI)
Cinder Gravels Newill et al (1987)	Ethiopia	Semi Arid	Low particle strength and high porosity Poor grading	Roadbase: Procedures developed to control selection; mechanical stabilisation with ash fines and selection of appropriate compaction plant vital.
Schist/Phyllite Fookes & Marsh 1981)	Nepal	Monsoonal sub-tropical	Poor aggregate shape	Modified processing procedure to ensure better shape

Source: Cook and Gourley (2002).

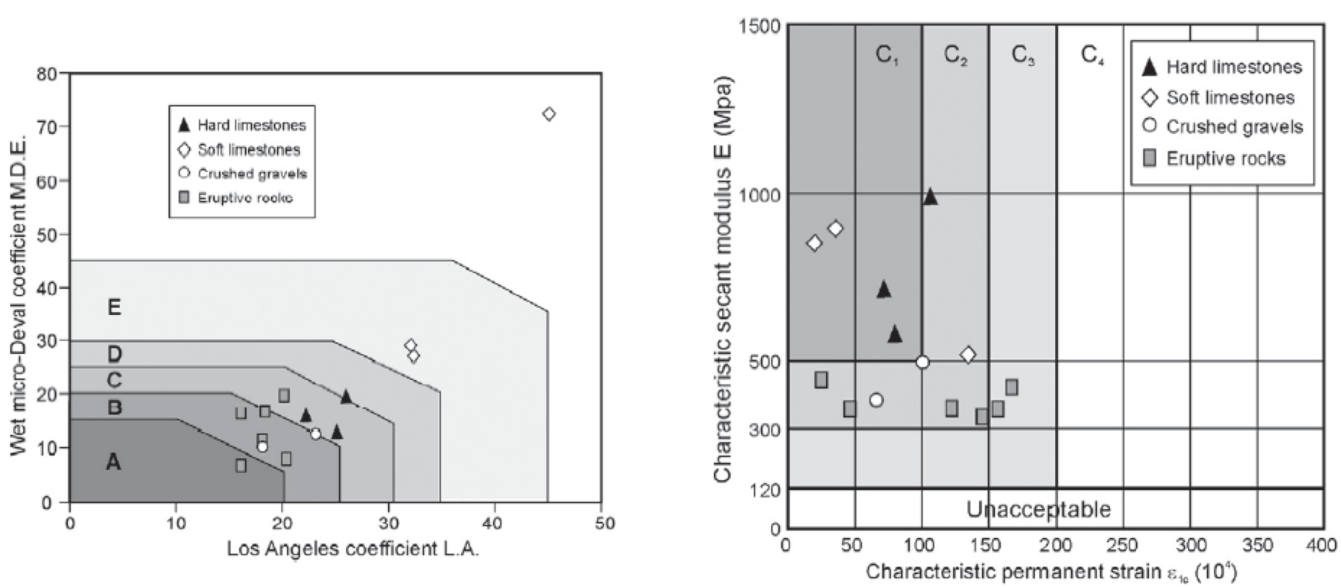


FIGURE 20 Physical (*left*) versus mechanical (*right*) classification for various unbound granular materials (Paute et al. 1994).

in pavement layers. The mechanistic nature of the responses of unbound aggregate materials can be characterized by resilient modulus (stiffness), whereas permanent deformation linked to shear strength often relates to rutting damage accumulation.

Because both the resilient (recoverable) and permanent deformation/strain components are to be considered simultaneously for mechanistic-empirical (M-E) evaluation of unbound aggregate behavior, the resistance to permanent deformation under repeated traffic loading relates to rutting damage accumulation in unbound aggregate materials. For example, the Australian Road Association determines both resilient modulus and permanent deformation from repeated load triaxial tests to characterize unbound aggregates and marginal materials (Austroads 2003). Khogali and Mohamed (2007) developed a mechanistic aggregate classification system based on a test procedure for combined determination of the resilient modulus and permanent deformation potential involving both elastic and plastic responses. Recently, Tao et al. (2010) introduced a mechanistic-based design approach to characterize and compare the behavior of traditional and recycled pavement base materials that employed dissipated energy concept to explain different shakedown responses of materials obtained from laboratory repeated-load triaxial tests and full-scaled accelerated loading tests. It was implied that permanent deformation characteristics of pavement materials provided a better measure for evaluating recycled and marginal materials against traditional unbound aggregates.

Shear strength is an important mechanistic property of unbound aggregate materials. The shear resistance of the material mainly contributes to developing a load resistance quality that greatly reduces the stresses transmitted to the underlying layers (Garg and Thompson 1997). Saeed et al. (2001) found under the NCHRP Project 4-23 study that shear strength of unbound aggregates under repeated loading had the most significant influence on pavement performance. Seyhan and Tutumluer (2002) suggested that a limiting value of the shear stress ratio (the level of applied shear stress as a fraction of the shear strength of the material) controlled the permanent deformation behavior of aggregates and that "good" quality aggregates typically had low shear stress ratios in the range of 0.2 to 0.5.

To better assess performance and rank different sources of aggregate materials, coupling mechanistic characteristics, including moduli, strength, and permanent strains, under representative ranges of operating environmental conditions is of essential importance from the MEPDG perspective. Development of performance-based material specifications is critical for optimized material use with reduced waste and eventually better utilization of construction dollars.

From a MEPDG perspective, determining the best use of different qualities of locally available aggregate materials in road bases/subbases may be a challenge. For example, Lukanen (1980) found early on that certain MnDOT) Class 3 aggregates were even stronger than Class 6 aggregates when placed in pavement granular layers. This was a surprising field evaluation considering that as MnDOT aggregate classes increase, usually better materials, such as a high-quality Class 6, are designated. During MnROAD study, similar contradictory trends were observed in backcalculated base layer moduli from falling weight deflectometer (FWD) testing of flexible pavements (Ovik et al. 2000). For both thin (<15 cm) and thick (>15 cm) asphalt concrete surfacing, the backcalculated base moduli of Class 3sp materials often were found to be greater than those of higher material classes (i.e., 4sp, 5sp, and 6sp) (Ovik et al. 2000). In light of these findings, several issues may need to be addressed, such as how to specify material properties based on their end-use performances; where in pavements to place locally available materials (either natural or recycled) of marginal quality; what type of pavements and critical traffic design levels should be determined beyond which no satisfactory pavement performance can be cost-effectively maintained by using marginal materials; and finally, what would be the optimum combination of high- and marginal-quality aggregate uses considering certain design features and site factors so that aggregate base and granular subbase materials can be optimized for satisfactory pavement performance.

Key Lessons

- The use of quarry by-products and other marginal aggregates in UAB/subbase layers can lead to sustainable pavement construction practices.
- In addition to commonly used tests for evaluating the physical characteristics, the mechanical performance of such marginal aggregates needs to be carefully studied.
- Currently used performance-based tests often fail to adequately evaluate unbound aggregate materials for application in pavement base/subbase layers. Additional research is required toward the development of new and modification of existing performance-based tests.
- Marginal aggregates and quarry by-products may be mixed with high-quality aggregates to develop material blends with adequate physical, chemical, and mechanical properties.
- It is not uncommon for a "weak" rock, such as limestone, to show very high resilient modulus values. However, the permanent deformation behavior also is best evaluated before classifying the material as a "good quality" aggregate for use in unbound base/subbase layers.

Best Value Granular Material Concept

Continual depletion of available natural aggregate resources has led an increasing number of transportation agencies to haul aggregate material for use in pavement construction from long distances. Such long distance hauling significantly increases the material cost for aggregates. According to NSSGA, transporting aggregates by truck over a distance of 30 to 50 miles can double the material cost for the end user. A 1998 USGS study indicated that for an assumed 56-km (35-mile) transportation distance, the cost of transporting aggregate materials for use in pavement base/subbase layers may exceed the estimated purchase price of the product at the source (Wilburn and Goonan 1998). Thus, more emphasis is placed by transportation agencies on the utilization of locally available "best value" granular materials, which do not require hauling aggregates from sources farther away from the project locality and incurring significant material hauling and transportation expenses.

Best value granular materials are locally available aggregate materials (natural or recycled) that can be used in pavement construction through slight modification to the design and/construction procedures. Various locally available aggregate materials currently are classified as "out of specification" according to traditional "recipe-based" testing techniques and specifications, but still there likely is significant opportunity for better value to be achieved. These materials may not satisfy all the requirements specified by transportation agencies for quality assurance of aggregates used in pavement applications. However, through slight modifications to the design and/or construction procedures, these materials can be used in pavement applications and thus will significantly reduce the overall construction cost and energy expenditure. This is particularly true for low-volume road applications for which the traffic volume is sufficiently low to allow the use of these "marginal quality" materials without significantly affecting the pavement performance under loading.

This "sustainable" alternative has garnered significant attention from different transportation agencies, and more attention is being paid to better utilization of best value granular materials, the use of which reduces the cost and energy associated with material hauling.

A recently completed research study sponsored by MnDOT conducted mechanistic–empirical pavement analyses to evaluate the performances of pavement structures with base/subbase layers constructed with locally available aggregate materials (Xiao et al. 2011; Xiao and Tutumluer 2011). The primary objective was to demonstrate that locally available aggregate materials could be economically efficient in the implementation of available mechanistic-based design procedures. Findings from the study indicated that for low-volume roads, base and subbase quality was not significant for achieving 20-year fatigue and rutting performance lives. Thus, for low-volume roads, using locally available and somewhat "marginal" materials was a significantly cost-effective alternative. However, for traffic volumes greater than 1.5 million equivalent single-axle loads (ESALs), aggregate material quality was critical in governing fatigue and rutting performances. Note that these findings may need to be verified in the field before being implemented into pavement design and construction practices.

In addition, the study found that a change in the subbase material quality had a more significant impact on pavement rutting performance than did a similar change in the base material quality. When the base quality was decreased from high to low, its effect on rutting performance was almost negligible for pavements with design traffic levels between 0.6 and 6.0 million ESALs. However, a similar drop in subbase material quality resulted in significant reduction in the rutting life. Accordingly, based on the research findings, for a pavement structure comprising "good quality" aggregates in the subbase, locally available "marginal" aggregates may be used in the base layer, while ensuring adequate pavement performance. A high-quality, stiff subbase exhibits almost a bridging effect to better protect the subgrade and offset any detrimental effects of low base stiffness, and as a result, the quality of base materials becomes trivial. Note that this is the same concept as that used in the South-African "inverted pavement" designs, which often use a cement-stabilized subbase over soft soils to effectively protect the subgrade while providing a stiff underlying layer for the base to enable compaction of granular base materials, often in excess of 100% of achieved Proctor densities.

Figure 21 presents the concept of best value granular materials illustrated as an implementation challenge of recent research study findings (Xiao et al. 2011; Xiao and Tutumluer 2011). Three components were proposed for incorporation into the MnDOT's mechanistic–empirical pavement analysis and design program MnPAVE to implement the best-value granular material aggregate selection, utilization, and mechanistic-based design concepts: (1) geographic-information system-based aggregate source management component, (2) aggregate property selection component for design, and (3) aggregate source selection/utilization component. To accomplish pavement designs, aggregate material source locations are identified with certain aggregate properties to be linked to mechanistic MnPAVE pavement analysis property inputs. The quality aspects of the used aggregates are then assessed for cost-effectiveness and unbound aggregate layer design thickness requirements for a sustainable pavement performance.

Key Lesson

The concept of best-value granular materials involves the use of locally available aggregates (natural or recycled) in pavement construction through slight modifications to design/construction procedures.

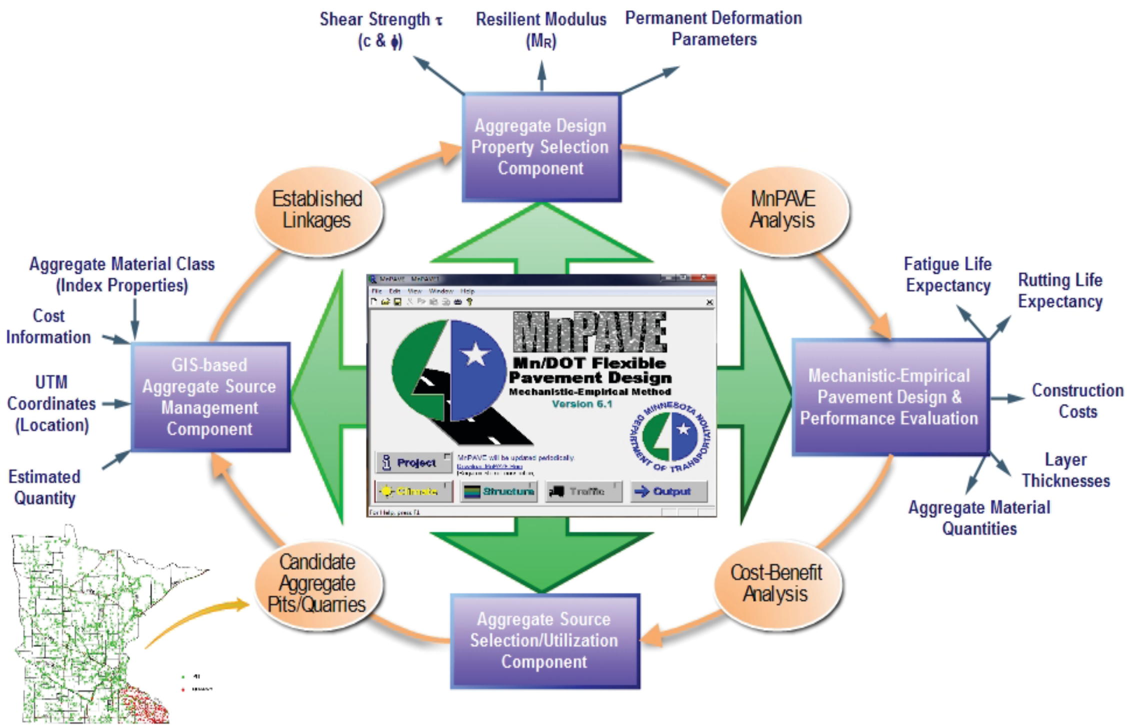


FIGURE 21 Best value granular material MnPAVE design implementation (Xiao and Tutumluer 2011).

RECYCLING AGGREGATES AND RECYCLED GRANULAR MATERIALS

Fluctuating oil prices in recent years have magnified the importance of building sustainable pavement systems with stronger and less moisture-susceptible unbound granular layers as the primary load-bearing structural components. High construction demands and accompanying geologic restrictions imposed by urbanization and environmental concerns have resulted in a scarcity of good-quality aggregate sources in many locations. As a result of this and sustainability issues, increased amounts of recycled or reclaimed aggregates are used to supplement virgin aggregate supplies. The FHWA lists the following recycled aggregate types as being used by different agencies in pavement granular base layer applications (http://www.fhwa.dot.gov/publications/research/infrastructure/pavements/97148/004.cfm): (1) blast furnace slag, (2) coal bottom ash, (3) coal boiler slag, (4) mineral processing wastes, (5) municipal solid waste combustor ash, (6) nonferrous slags, (7) reclaimed asphalt pavement, (8) reclaimed concrete, (9) steel slag, and (10) waste glass. According to the USGS, the highway industry (American Concrete Pavement Association, Construction Materials Recycling Association, FHWA, and National Asphalt Pavement Association) has estimated the quantities of reclaimed and recycled asphalt and concrete materials used in construction at closer to 100 million tons each in 2009; approximately 14 and 18 million tons were used as RCA and RAP aggregate materials, respectively.

Of the previously mentioned material types, only the following three recycled materials were considered in this synthesis study: (1) unbound aggregate materials recycled from old pavement base/subbase layers, and recycled materials: that is, (2) RAP and (3) RCA. Consideration was given in this synthesis to the different test methods used by state transportation agencies to check the adequacy of recycled materials before allowing their use in the construction of UAB and subbase layers. Particular emphasis was given to whether state transportation agencies impose additional requirements for quality assurance of recycled aggregate materials compared with those for virgin aggregates.

The following properties of recycled aggregates have been identified by *NCHRP Report 598* as relevant to their use in unbound pavement layers (Saeed 2008): (1) shear strength, (2) CBR, (3) cohesion and angle of internal friction, (4) resilient or compressive modulus, (5) density, (6) permeability, (7) frost resistance, (8) durability index, and (9) resistance to moisture damage.

AASHTO specification PP 56-06, Evaluating the Engineering and Environmental Suitability of Recycled Materials, outlines a framework for assessing the feasibility to use recycled materials in the highway environment by considering issues such as (1) engineering and material properties; (2) environmental, health, and safety properties; (3) implementation aspects; and (4) recycling aspects of the recycled materials. Although the specification recommends a general framework to be adopted before the use of recycled materials, it also clearly recommends the evaluator consider local conditions before selecting different criteria and the corresponding threshold values.

In addition to the general evaluation framework listed in AASHTO PP 56-06, agencies may sometimes adopt the toxicity characteristic leaching procedure (TCLP) to chemically evaluate the potential harmful effects of leaching through an UAB/subbase layer constructed using recycled materials. Designated as Method 1311 by the EPA (http://www.epa.gov/osw/hazard/testmethods/sw846/pdfs/1311.pdf), TCLP determines the mobility of both organic and inorganic hazardous materials in recycled and waste materials. For example, the Georgia Department of Transportation (GDOT) lists the TCLP as a required “acceptance” test for recycled concrete base aggregates for sources of RCA that are not from GDOT projects or GDOT pavements.

The ongoing Transportation Pooled Fund Study TPF-5(129), Recycled Unbound Pavement Materials (http://www.pooledfund.org/Details/Study/361), has the objective of monitoring the performance of several test cells at MnROAD constructed using recycled materials in the granular base layers, including blended with virgin materials and 100% recycled asphalt and concrete pavement materials. Issues that are being considered include variability in material properties, purity of material, and how to identify and control material quality. The project findings will include laboratory studies, examination of existing field sites, and evaluation of data from MnROAD test sections. Anticipated results from this project include a suite of tests and/or protocols that may be used to identify the critical characteristics of these recycled materials, as well as optimum design criteria and best construction practices needed for a durable base that meets the properties proposed for layer design.

Reclaimed Asphalt Pavement

Since a principal constituent of RAP is its mineral aggregates, the overall chemical composition of RAP is similar to that of the mineral aggregates. Asphalt cements constitute only a minor percentage of RAP. The principal elements in asphalt cement molecules are carbon and hydrogen. Other materials, such as sulfur, nitrogen, and oxygen, usually are present in very small amounts. Asphalt cements are made up of asphaltenes, resins, and oils. Upon oxidation, the oils convert to resins and asphaltenes, in which the resins convert to asphaltene-type molecules, resulting in age hardening and a higher viscosity binder (Roberts et al. 1996). This change in the chemical composition would influence the unbound layer stiffness and shear strength and, consequently, its performance parameters, such as rutting and fatigue cracking.

RAP can be used as granular base or subbase material in pavement structures (e.g., Garg and Thompson 1996; Maher and Popp 1997; Bennert et al. 2000; Chini et al. 2001). Garg and Thompson (1996) conducted a field testing research program to investigate the potential of using RAP as a pavement base. This study demonstrated that the performance of the RAP base was comparable to that of a crushed stone base.

According to Stroup-Gardiner and Wattenberg-Komas (2013), when RAP is used as an aggregate in an unbound application, the volume of asphalt in the RAP reduces the specific gravity, and the presence of asphalt seals most of the surface area of the particles. These characteristics result in a lower unit weight and a reduced amount of water needed to achieve the desired compaction level. A study by Taha et al. (1999) recommends blending granular RAP with virgin aggregates to attain the proper bearing strengths because the RAP-bearing capacity usually is lower than that of conventional granular aggregate bases. As conventional granular aggregate content increased, dry density and CBR values increased (Taha et al. 1999). Therefore, it is important to characterize and quantify the expected range of RAP properties before application. Findings from the ongoing Transportation Pooled Study TPF-5(129) have indicated that although RAP materials may show high resilient modulus values, aggregate layers constructed using 100% RAP materials often accumulate high permanent deformation values.

The degree of expansion for the RAP materials is not well known. Expansion of the RAP material is particularly critical when the RAP contains expansive components, such as steel slag, which may not be commonly allowed in pavement base/subbase layers. Note that steel slag aggregates often are used in HMA surface courses where their high frictional characteristics are particularly useful. Therefore, any RAP material obtained from these surface courses with steel slag aggregates potentially may lead to expansion and resulting pavement heave when used in UAB/subbase courses. Recent experiences with volume changes of 10% or more have been attributable to hydration of the calcium and magnesium oxides in the recycled steel slag aggregate when water was encountered in the pavement base layer (Collins and Ciesielski 1994). The free lime hydrates rapidly and can cause large volume changes over a relatively short period of time (weeks), whereas magnesia hydrates much more slowly and contributes to long-term expansion that may take years to develop. The potential expansion depends on the origin of the slag, grain size and gradation, and the age of the stockpile (Rohde et al. 2003). Deniz et al. (2010) studied the expansive properties of 17 RAP materials, including recycled steel slag aggregates, with respect to those of the virgin aggregates in the laboratory following the ASTM D4792 Potential Expansion of Aggregates from Hydration Reactions test method. The RAP materials had much lower tendencies to expand than did the virgin steel slag aggregates, most likely owing to an effective asphalt coating around the aggregate that prevents any significant ingress of water into the aggregate. Depending upon the level of expansion and the material gradation, dense-graded aggregate base applications with steel slag under pavements and structures may have to be avoided.

Recycled Concrete Aggregate

Kuo et al. (2001) investigated the feasibility of using RCA as a base course material in asphalt pavements. Through literature

TABLE 2
PROPOSED SPECIFICATIONS FOR USE OF RCA IN UNBOUND AGGREGATE BASE LAYERS

Type of Test	**Proposed Specifications**
Gradation test Sieve No. 50 mm 37.5 mm 19 mm 9.5 mm #4 #10 #50 #200	Gradation limits (90% confidence interval) 100 98–100 65–100 40–83 27–63 20–49 8–24 2–6
Limerock bearing ratio Test	Minimum 120
LA abrasion loss	<48%
Sodium sulfate test	<5%
Sand equivalent	>70%
Heavy metals	5 ppm
Asbestos	Free of asbestos
Optimum moisture content	10%–12%
Maximum dry unit weight	108–120 pcf
Permeability	0.10–1.40 (ft/day)
Impurities	<2.0% by weight
Structural coefficient	0.30
Thickness requirement	Minimum 8.0 in. (20.4 cm)
Thickness equivalency	1.0 in. (2.54 cm)

Source: Kuo et al. (2001).

review, laboratory testing, accelerated performance testing and pavement distresses monitoring, FWD testing, and theoretical analysis of pavements, Kuo et al. developed the following specifications for use of RCA in Florida (see Table 2).

According to the American Concrete Pavement Association (2008), RCA typically is highly angular and has a higher water absorption capacity, lower specific gravity, lower strength, and lower abrasion resistance than do conventional construction aggregates. Recommendations provided for the design, construction, and QC of RCA bases and subbases in pavement applications were reviewed by Stroup-Gardiner and Wattenberg-Komas (2013). Pavement design needed to consider the stiffer RCA layer properties compared with unstabilized base materials, which was a function of the additional hydration associated with the RCA materials. The stiffening effect was enhanced when using dense gradations with high RCA fines (minus 4.75 mm size) contents. Properties that influenced the performance of RCA base materials for pavements included aggregate toughness, frost susceptibility, shear strength, and stiffness. Recommendations for QC/QA testing include Micro-Deval (AASHTO T327), tube suction, static triaxial (AASHTO T234), repeated load testing, and resilient modulus. Unbound RCA bases might limit the fines (minus 4.75 mm size) content to prevent clogging drainage features and might be used below the drainage systems. Stabilizing the RCA could bind excess fines.

Recently, Ooi et al. (2011) recommended the following practices when using RCA as a base course: (1) allow only uncrushed concrete that can be visually inspected to be used as RCA, (2) accept RCA only from suppliers who can guarantee the quality, (3) RCA from unknown sources should not be accepted unless certified by a qualified engineer or scientist as being free of deleterious materials (such as aluminum), (4) avoid using building demolition RCA, (5) require a paper trail to document the RCA source, (6) use a nonferrous metal detector to determine whether aluminum is present and inspect the RCA visually before use.

Key Lessons

- Extensive testing of locally available natural and recycled aggregates to characterize their shear strength, resilient modulus, and permanent deformation behaviors will enable optimum use of these materials and limit material hauling and transportation costs.
- RAP materials are best tested in the laboratory for resilient modulus and permanent deformation behavior before being used in UAB/subbase layers. Several studies have reported high resilient modulus

values for RAP accompanied by significantly high permanent deformation accumulations.
- The expansive properties of RAP materials containing expansive components such as steel slag are best carefully evaluated before their application in UAB/subbase layers.
- Recycled crushed concrete often can be adequately used in UAB/subbase layers.
- Care is to be taken while blending two different recycled aggregate types to ensure that the resulting blend possesses adequate physical, chemical, and mechanical properties.

POTENTIAL ENVIRONMENTAL IMPACTS FROM USING RECYCLED MATERIALS

The potential environmental impact from using recycled materials in UAB and subbase layers remains a concern for transportation agencies. Although the environmental concerns regarding the use of RAP and RCA in unbound aggregate layers are not as pronounced as are those associated with the use of some other recycled materials, such as fly ash and silica, several transportation agencies require these materials to meet environmental quality requirements (Saeed 2008).

There is a concern that RAP and RCA used in unbound aggregate pavement layers, when subjected to intermittent wetting and drying, may leach contaminants into the groundwater. This is particularly true for RCA because it presents a potential for leaching of residual hydroxyl (OH−) ions from the cement paste, thus raising the pH of groundwater. Moreover, concrete that already has been subjected to alkali-silica reaction and sulfate attacks may involve deleterious expansion that adversely affects the performance of UAB and subbase layers (Rathje et al. 2002).

RAP also has certain environmental implications with respect to the potential for contamination of ground and surface water systems. The aromatic compound is an organic compound of asphalt cement that has become a great concern because the levels of this aromatic hydrocarbon present in asphalt cement could exceed published soil clean-up standards available in several states. Concerns about the use of RAP are also addressed in *NCHRP Report 443: Primer Environmental Impact of Construction and Repair Materials on Surface and Ground Waters,* which was prepared from NCHRP project 25-09. Most binder treatment, required for mechanical reasons, significantly modifies the leaching behavior of a recycled material. Sadecki et al. (1996) analyzed the leachate water from three experimental stockpiles made from coarse concrete, fine concrete, and salvaged bituminous material obtained from pavement millings, respectively. Following EPA-approved methods for analyzing the leachate quality, they observe that the pH value exceeded the Minnesota standards, whereas the chromium content may have exceeded the standard sometimes. The polynuclear aromatic hydrocarbon concentrations from the bituminous piles often were near or below detectable limits. Measured parameters, such as alkalinity, chloride, sodium, potassium, total solids contents, and the conductivity were, in general, higher for the concrete piles than for the bituminous pile. They suggested that the potential impact of stockpile runoff on groundwater can be controlled with proper management of stockpile locations. A study by Hill et al. (2001) has shown that the binder treatment may dilute or amend leachable levels, alter the pH, and reduce the permeability. However, the addition of alkali binder could introduce some contaminants, such as calcium. The study by Hill et al. recommends determining optimum binder treatment with regard to type of recycled materials, especially when there is a lack of local moisture- and performance-related material properties. The environmental impacts to soils or groundwater need to be evaluated when RAP is stockpiled or used as an unbound granular material.

Although RAP is usually free from damaging chemical compounds, RAP obtained from milling pavements subjected to deicing salts may contain some hazardous chemicals. Cosentino et al. (2003) analyzed RAP for heavy metal (silver, cadmium, chromium, lead, and selenium) concentrations and concluded that the levels were well below the limits specified by the EPA. Cosentino et al. reported that the strength-deformation characteristics of field sites constructed using RAP improved over the 8-week study period, as reflected from field CBR, FWD, Clegg impact hammer, and soil stiffness gauge test results. Based on the results, the authors also recommended permitting the use of RAP as a subbase below rigid pavements.

Snyder and Bruinsma (1996) conducted an extensive literature review evaluating the effects of RCA usage in UAB layers on pavement drainage. Later, using thermodynamic techniques, Bruinsma et al. (1997) observed that the Portlandite [$Ca(OH)_2$] present in RCA can be dissolved in water and subsequently lead to the precipitation of calcite ($CaCO_3$) upon coming in contact with atmospheric carbon dioxide (CO_2). Such precipitation often can lead to a reduction in the permittivity of pavement subdrainage systems. They concurred with Muethel (1989) and Tamirisa (1993) in stating that the calcite precipitation can be controlled by reducing the amount of Portlandite [$Ca(OH)_2$] readily available for dissolution, which may be accomplished through reduction in the amount of concrete cement fines.

According to Stroup-Gardiner and Wattenberg-Komas (2013), crushing concrete reveals previously unexposed surfaces that contain some calcium hydroxide and partially unreacted cement grain and that react with air to form calcium carbonate precipitate. High levels of sodium chloride have been found in RCA produced from pavements subjected to deicing salts over years of service and may cause corrosion concerns if used in new PCC with steel. The alkalinity decreased rapidly when diluted with low pH water and exposure of the dissolved calcium hydroxide with CO_2. The runoff could also be highly alkaline because of leaching of calcium hydroxide from freshly crushed concrete. Precipitate could clog drain pipes and filter fabrics, but washing the crushed concrete helped minimize some of these problems.

AASHTO specification M 319-02, Reclaimed Concrete Aggregate for Unbound Soil-Aggregate Base Course, clearly identifies the high likelihood of increased pH values for water percolating through UAB layers constructed using RCA. The AASHTO specification further recommends setting appropriate limits on the proximity of such layers to groundwater and surface waters. Moreover, such layers should not be used in the vicinity of metal culverts susceptible to corrosion under such high-alkaline environments. Finally, precipitation of soluble minerals from the water percolating through base course layers containing RCA may lead to the clogging of drainage layers or other pavement drainage features. Thus, it is important to closely monitor and regulate the use of RCA in close proximity to such components.

In some instances, recycled aggregates may not satisfy agency QC requirements for use as unbound aggregate pavement layers. In such cases, slight adjustments may be made to the QC specifications accompanied by design modifications to allow the use of such "marginal" recycled materials. However, in cases in which the recycled material properties deviate significantly from agency specifications, the use of those materials in unbound aggregate pavement layers is best prohibited. The survey of state and Canadian provincial transportation agencies indicated the presence of environmental concerns in several agencies regarding the use of RAP and RCA in UAB and subbase layers. One respondent (Indiana DOT) mentioned an experience with loss of vegetation caused by leaching from an unbound aggregate layer constructed using RCA. Indiana DOT has since prohibited the use of recycled materials in UAB and subbase layers, particularly for pavements with underdrain systems, because precipitation of leachates potentially may lead to clogging of the underdrain system.

A recent report by FHWA summarized the experience of several state transportation agencies concerning the use of RCA in transportation applications. Based on the experience of MnDOT, RCA could be used up to 100% as a filter/separation layer under a permeable aggregate base drainage layer in accordance with the applicable drainage specifications. In the presence of drainage layers and/or perforated drainage pipes, a blend of RCA with new aggregate could be used as subgrade when at least 95% of the RCA was retained on the 4.75-mm sieve. Alkaline effluent from RCA layer was not a significant issue when RCA was kept a sufficient distance from the drainage outlets. A blend of open-graded RCA with new aggregate could be used for improved stability and density (FHWA 2004).

Recycling of Unbound Aggregate Material from Existing Pavements

The survey of state and Canadian provincial transportation agencies collected information on agency policies regarding recycling of unbound aggregate materials from base and subbase layers of existing pavements. Twenty-four of 46 (52.2%) respondents indicated that recycling of unbound aggregate materials from existing pavement base and subbase layers was a common practice in their respective states. Moreover, seven responded that such recycling was done occasionally in their states. Twenty-one of 46 respondents indicated that the use of recycled aggregates from existing base and subbase courses was incorporated into their state specifications, whereas 22 states did not allow the inclusion of such materials into specifications. Only two states allow contractors to use locally available "marginal" or "out of specification" aggregates for UAB and subbase applications. Six states allow the use of such "marginal" materials occasionally because of economic issues. Most agencies have a stricter material quality requirement for UAB layers than for subbase layers. Therefore, these agencies sometimes allow the use of marginal aggregates in subbase layers while prohibiting their use in base layers. Some states also indicated that marginal materials occasionally were blended with virgin aggregates to lower the cost associated with material procurement and transportation.

Commonly Used Recycled Materials in Unbound Aggregate Base and Subbase Layers

The survey of state and Canadian provincial transportation agencies indicated that RCA and RAP are the two most commonly used recycled materials in UAB and subbase layers. Some agencies also reported the use of less commonly available materials, such as air-cooled blast furnace slag (three agencies), glass cullets (seven agencies), aggregates blended with oil field waste (one agency), and so forth. Figure 22 shows the relative distribution of state transportation agencies using different recycled aggregate materials in UAB and subbase layer construction.

Current State of the Practice Regarding Testing of Recycled Materials

In the survey of state and Canadian provincial transportation agencies conducted under the scope of this synthesis study, information was gathered on the current state of practice regarding the testing of recycled materials for quality acceptance. Agency responses indicated that sieve analysis, abrasion tests such as Los Angeles abrasion or Micro-Deval, and percent deleterious materials are the most commonly used tests for evaluating recycled granular material quality (see Figure 23). Four agencies also indicated the use of Atterberg limit tests on recycled granular materials. Note that soundness tests using sodium or magnesium sulfate may result in RCA being susceptible to sulfate attack, therefore resulting in high loss values. Thus, AASHTO specification M 319-02 recommends the use of soundness tests using sulfate solutions only when

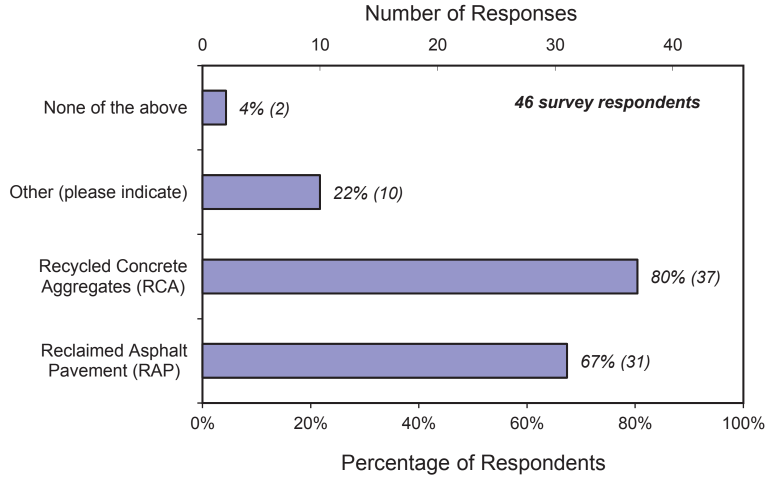


FIGURE 22 Use of different recycled aggregate materials by state transportation agencies in unbound aggregate pavement base and subbase applications (46 respondents).

local experience has found these methods to be satisfactory. In lieu of the sulfate soundness tests, agencies may opt to waive the soundness requirements or adopt one of the following alternative test methods:

- AASHTO T 103: Soundness of Aggregates by Freezing and Thawing;
- New York State DOT Test Method NY 703-08: Resistance of Coarse Aggregate to Freezing and Thawing; or
- Ontario Ministry of Transportation (MOT), Test Method LS-614: Freezing and Thawing of Coarse Aggregate.

When asked about environmental concerns regarding the use of recycled granular materials in pavement unbound aggregate layers, 68% (17 of 25 respondents) indicated no such concerns. This indicates a potential gap in knowledge concerning phenomena such as leaching from RCA and the resulting contamination of groundwater. For the agencies that did report environmental concerns with recycled granular materials, leaching and resulting change in the pH level of groundwater were reported to be the primary concerns. Sixty-four percent of the responding agencies do not require any strength, deformation, or modulus characterization of recycled materials such as RCA and RAP before their use in unbound aggregate pavement layers. For the agencies that require such tests to be conducted on recycled materials, the quality requirements are the same as those for virgin aggregates.

Key Lessons

- Test recycled materials for potential environmental impacts before use in UAB/subbase layers.

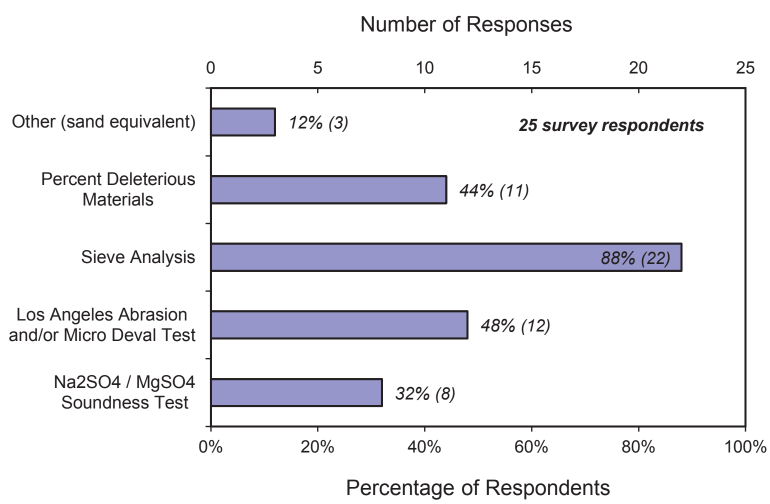


FIGURE 23 Different tests used by agencies for evaluating the material quality of recycled granular materials.

- Evaluate the use of recycled materials in pavements on a project basis, rather than by following generic guidelines. For example, the use of RCA in UAB/subbase layers may or may not be allowed depending on whether the pavement has an underdrain system or not. This is particularly important for RCA obtained from sources that have not been tested previously for material quality. Some of the quality testing requirements may be relaxed for "properly tested and documented" RCA sources.

SUMMARY

This chapter presents an overview of different types of aggregate materials available as natural resources in the United States. Geologic phenomena responsible for the formation of different rock types are discussed, and the distribution of different rock types in the conterminous United States is presented. Important aggregate properties that affect the performance of UAB and subbase layers are discussed. Accordingly, different test procedures commonly used by transportation agencies to check the quality of aggregates before including them in UAB and subbase layers are listed. At this stage, the state of the practice in aggregate material selection and quality check is discussed by presenting the information gathered through survey of state and Canadian provincial transportation agencies. The concept of best value granular material utilization was introduced for pavement projects with the potential to save energy and material hauling costs. The sustainable production and use of aggregate and recycled granular materials in base and subbase layers are discussed, as are different tests recommended by researchers to check the quality of recycled aggregate materials in base and subbase layer applications.

REFERENCES

Adu-Osei, A., D.N. Little, E. Tutumluer, and R.L. Lytton, "Effect of Compaction on Orthotropic Properties of Aggregates," In *Proceedings of the European Workshop on Compaction of Soils and Granular Materials,* ISSMGE, Paris, France, May 19, 2000.

Allen, J., *The Effect of Non-constant Lateral Pressures of the Resilient Response of Granular Materials,* Ph.D. thesis, University of Illinois at Urbana–Champaign, Urbana, 1973.

Allen, J.J. and M.R. Thompson, "Resilient Response of Granular Materials Subjected to Time Dependent Lateral Stresses," *Transportation Research Record 510,* Transportation Research Board, National Research Council, Washington, D.C., 1974, pp. 1–13.

American Concrete Pavement Association (ACPA), *Recycled Concrete in Subbases: A Sustainable Choice,* Technical Series, TS204.9P, ACPA, Washington, D.C., 2008 [Online]. Available: http://www.pavement.com/Concrete_Pavement/Technical/Downloads/TSs.asp.

Ashtiani, R.S. and D. Little, *Acceptability Criteria for High Fines Content Aggregate Pavement Layers,* Report No ICAR 401131, International Center for Aggregates Research (ICAR), College Station, Tex., 2007, 100 pp.

Austroads, *Development of Performance-based Specifications for Unbound Granular Materials. Part A: Issues and Recommendations,* AP-T29/03, Sydney, Australia, 2003.

Barksdale, R.D., "Laboratory Evaluation of Rutting in Base Course Materials," *Proceedings of the 3rd International Conference on Structural Design of Asphalt Pavements,* 1972, pp. 161–174.

Barksdale, R.D., "Crushed Stone Base Performance," *Transportation Research Record 954,* Transportation Research Board, National Research Council, Washington, D.C., 1984, pp. 78–87.

Barksdale, R.D. and S.Y. Itani, "Influence of Aggregate Shape on Base Behavior," *Transportation Research Record 1227,* Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 173–182.

Barksdale, R.D., S.F. Brown, and F. Chan, *NCHRP Report 315: Potential Benefits of Geosynthetics in Flexible Pavements,* Transportation Research Board, National Research Council, Washington, D.C., 1989.

Barksdale, R.D., *The Aggregate Handbook,* National Stone Association, Washington, D.C., 1991.

Barksdale, R.D., C.O. Pollard, T. Siegel, and S. Moeller, *Evaluation of the Effect of Aggregate on Rutting and Fatigue of Asphalt,* Technical Report FHWA-AG-92-8812, Georgia Department of Transportation, Atlanta, 1992.

Bennert T., W.J. Papp, A. Maher, and N. Gucunski, "Utilization of Construction and Demolition Debris Under Traffic-Type Loading in Base and Subbase Applications," *Transportation Research Record: Journal of the Transportation Research Board, No. 1714,* Transportation Research Board of the National Academies, Washington, D.C., 2000, pp. 33–39.

Bilodeau, J.-P., G. Dore, and P. Pierre, "Erosion Susceptibility of Granular Pavement Materials," *International Journal of Pavement Engineering,* Vol. 8, No. 1, Mar. 2007, pp. 55–66.

Bilodeau, J.-P., G. Dore, and P. Pierre, "Gradation Influence on Frost Susceptibility of Base Granular Materials," *International Journal of Pavement Engineering,* Vol. 9, No. 6, Dec. 2008, pp. 397–411.

Bilodeau, J.-P., G. Dore, and P. Pierre, "Pavement Base Unbound Granular Materials Gradation Optimization," In *Proceedings of the 8th International Conference on the Bearing Capacity of Roads, Railways and Airfields,* Vol. 1, Taylor & Francis Group, London, U.K., 2009, pp. 145–154.

Bolen, W.P., *2010 Minerals Yearbook: Sand and Gravel, Construction* [advance release], U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C., June 2012.

Brown, S.F. and F.W.K. Chan, "Reduced Rutting in Unbound Granular Pavement Layers through Improved

Grading Design," *Proceedings of the Institution of Civil Engineers, Transport,* Vol. 117, No. 1, 1996, pp. 40–49.

Bruinsma, J.E., K.R. Peterson, and M.B. Snyder, "Chemical Approach to Formation of Calcite Precipitate from Recycled Concrete Aggregate Base Layers," *Transportation Research Record: Journal of the Transportation Research Board, No. 1577,* Transportation Research Board of the National Academies, Washington, D.C., 1997, pp. 10–17.

Bullen F., "Design and Construction of Low-Cost, Low-Volume Roads in Australia," Paper No. LVR8-1116, TRR 1819, School of Engineering, University of Tasmania, Hobart, Australia, 2003.

Chini, A., S. Kuo, J. Armaghani, and J. Duxbury, "Test for Recycled Concrete Aggregate in Accelerated Test Track," *Journal of Transportation Engineering,* ASCE, Vol. 127, No. 6, 2001, pp. 486–492.

Clyne, T.R., E.N. Johnson, and B.J. Worel, *Use of Taconite Aggregates in Pavement Applications,* Final Report MN/RC-2010-24, Minnesota Department of Transportation, St. Paul, June 2010.

Collins, R.J. and S.K. Ciesielski, *NCHRP Synthesis of Highway Practice 199: Recycling and Use of Waste Materials and By-Products in Highway Construction,* Transportation Research Board, National Research Council, Washington, D.C., 1994.

Cook, J.R. and C.S. Gourley, "A Framework for the Appropriate Use of Marginal Materials," World Road Association (PIARC), Technical Committee C12 Seminar, Mongolia, 2002.

Cosentino, P.J., et al., *Developing Specifications for Using Recycled Asphalt Pavement as Base, Subbase or General Fill Materials, Phase II,* Report No. FL/DOT/RMC/06650-7754, Florida Department of Transportation, Tallahassee, 2003.

Dawson, A.R., N.H. Thom, and J.L. Paute, "Mechanical Characteristics of Unbound Granular Materials as a Function of Condition," Flexible Pavements, Proceedings of European Symposium Euroflex 1993, A.G. Correia, ed., Balkema, Rotterdam, the Netherlands, 1996, pp. 35–44.

Deniz, D., E. Tutumluer, and J.S. Popovics, "Evaluation of the Expansive Characteristics of Reclaimed Asphalt Pavement (RAP) and Virgin Aggregate Used As Base Materials," *Transportation Research Record: Journal of the Transportation Research Board, No. 2167,* Transportation Research Board of the National Academies, Washington, D.C., 2010, pp. 10–17.

Dempsey, B.J., "Laboratory and Field Studies of Channeling and Pumping," *Transportation Research Record 849,* Transportation Research Board, National Research Council, Washington, D.C., 1982.

Elliott, R.P. and S.I. Thornton, "Resilient Modulus and AASHTO Pavement Design," *Transportation Research Record 1196,* Transportation Research Board, National Research Council, Washington, D.C., 1988, pp. 116–124.

Federal Highway Administration (FHWA), *Transportation Applications of Recycled Concrete Aggregate,* FHWA State of the Practice National Review, FHWA, Washington, D.C., Sep. 2004.

Federal Highway Administration (FHWA), *User Guidelines for By-product and Secondary Use Materials in Pavement Construction,* FHWA, Washington, D.C., 2008 [Online]. Available: http://www.fhwa.dot.gov/pavement/pub_details.cfm?id=384.

Garg, N. and M.R. Thompson, "Lincoln Avenue Reclaimed Asphalt Pavement Base Project," *Transportation Research Record 1547,* Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 89–95.

Garg, N. and M.R. Thompson, "Triaxial Characterization of Minnesota Road Research Project Granular Materials," *Transportation Research Record 1577,* Transportation Research Board, National Research Council, Washington, D.C., 1997.

Gates, L., E. Masad, R. Pyle, and D. Bushee, *Aggregate Imaging System 2* (*AIMS2*): *Final Report,* Report FHWA-HIF-11-030, Highway for Life Program Office, Office of Infrastructure, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 2011.

Gray, J.E., (1962). "Characteristics of Graded Base Course Aggregates Determined by Triaxial Tests," *Engineering Research Bulletin No. 12,* National Crushed Stone Association, Alexandria, Va., 1962.

Halmen, C.K. and J.T. Kevern, "Utilization of Quarry Fines for Sustainable Construction," *Stone, Sand & Gravel Review,* July/Aug. 2010, pp. 26–27 [Online]. Available: http://www.nssga.org/ssgReview/.

Hicks, R.G. and C.L. Monismith, "Factors Influencing the Resilient Properties of Granular Materials," *Transportation Research Record 345,* Transportation Research Board, National Research Council, Washington, D.C., 1971, pp. 15–31.

Hill, A.R., A.R. Dawson, and M. Mundy, "Utilization of Aggregate Materials in Road Construction and Bulk Fill," *Journal of Resources, Conservation and Recycling,* Vol. 32, No. 3-4, July 2001, pp. 305–320.

Holtz, R.D., "Compaction Concepts," Chapter 3, In *Guide to Earthwork Compaction, State of the Art Report* 8, Transportation Research Board, National Research Council, Washington, D.C., 1990.

Holubec, I., *Cyclic* Creep of Granular Materials, Report No. RR147, Department of Highways, Ontario, Canada, 1969.

Hudson, W.R., D. Little, A.M. Razmi, V. Anderson, and A. Weismann, *An Investigation of the Status of By-Product Fines in the United States,* Research Report 101-1, International Center for Aggregate Research, Austin, Tex., Nov. 1997.

Illinois Department of Transportation (IDOT), "Subgrade Stability Manual," Bureau of Bridges and Structures, IDOT, Springfield, May 1, 2005, 27 pp. [Online]. Available: http://www.dot.il.gov/bridges/pdf/subgrade stability manual.exe.

Janoo, V.C., *Quantification of Shape, Angularity, and Surface Texture of Base Course Materials,* U.S. Army Corps of Engineers Special Report 98-1, Cold Regions Research & Engineering Laboratory, Hanover, N.H., 1998.

Jorenby, B.N. and R.G. Hicks, "Base Course Contamination Limits," *Transportation Research Record 1095,* Transportation Research Board, National Research Council, Washington, D.C., 1986, pp. 86–101.

Kaya, Z., A. Cetin, B. Cetin, and A.H. Aydilek, "Effect of Compaction Method on Mechanical Behavior of Graded Aggregate Base Materials," In *Proceedings of the 2012 GeoCongress, Oakland, California,* Mar. 25–29, 2012, pp. 1486–1494.

Kamal, M.A., A.R. Dawson, O.T. Farouki, D.A.B. Hughes, and A.A. Sha'at, "Field and Laboratory Evaluation of the Mechanical Behavior of Unbound Granular Materials in Pavements," *Transportation Research Record 1095,* Transportation Research Board, National Research Council, Washington, D.C., 1993, pp. 88–97.

Khogali, W.E.I. and E.H.H. Mohamed, "Mechanistic Classification of Unbound Materials," *Transportation Research Record: Journal of the Transportation Research Board, 2016,* Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 39–48.

Knutson, R.M. and M.R. Thompson, "Resilient Response of Railway Ballast," *Transportation Research Record 651,* Transportation Research Board, National Research Council, Washington, D.C., 1977, pp. 31–39.

Kolisoja, P., T. Saarenketo, H. Peltoniemi, and N. Vuorimies, "Laboratory Testing of Suction and Deformation Properties of Base Course Aggregates," *Transportation Research Record: Journal of the Transportation Research Board, 1787,* Transportation Research Board of the National Academies, Washington, D.C., 2002, pp. 83–89.

Kumar, D.S. and W.R. Hudson, *Use of Quarry Fines for Engineering and Environmental Applications,* Special Research Report for the National Stone Association, Center for Transportation Research, The University of Texas-Austin, Oct. 1992, 130 pp.

Kuo, S.S., H.S. Mahgoub, and J.E. Ortega, *Use of Recycled Concrete Made with Florida Limestone Aggregate for a Base Course in Flexible Pavement,* Florida Department of Transportation Final Report for Contract BC 409, State Materials Office, Gainesville, 2001, 212 pp.

Langer, W.H., *Aggregate Resource Availability in the Conterminous United States, Including Suggestions for Addressing Shortages, Quality, and Environmental Concerns,* Open File Report 2011-1119, U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C., 2011.

Lekarp, F., U. Isacsson, and A. Dawson, "State of the Art. I: Resilient Response of Unbound Aggregates," *Journal of Transportation Engineering,* Vol. 126, No. 1, 2000a, pp. 66–75.

Lekarp, F., U. Isacsson, and A. Dawson, "State of the Art. II: Permanent Strain Response of Unbound Aggregates," *Journal of Transportation Engineering,* Vol. 126, No. 1, 2000b, pp. 76–83.1.

Lukanen, E.O., *Application of AASHO Road Test Results to Design of Flexible Pavements in Minnesota,* Final Report FHWA/MN-80/09, Office of Materials and Research, Minnesota Department of Transportation, St. Paul, 1980, 83 pp.

Maher, M.H. and W. Popp, Recycled Asphalt Pavement as a Base and Subbase Material, ASTM STP 1275, American Society of Testing and Materials, New Orleans, 1997, pp. 42–53.

Manning, D. and J. Vetterlein, *Exploitation and Use of Quarry Fines,* Mineral Solutions, Ltd., Manchester, U.K., 2004, 55 p.

Maree, J.H., C.R. Freeme, N.J.W. van Zyl, and P.F. Savage, "The Permanent Deformation of Pavements with Untreated Crushed-Stone Bases as Measured in Heavy Vehicle Simulator Tests," *Proceedings of the 11th Australian Road Research Board Conference,* Melbourne, 1982.

Masad, E., T. Al-Rousan, J. Button, D.N. Little, and E. Tutumluer, *NCHRP Report 555: Test Methods for Characterizing Aggregate Shape, Texture, and Angularity,* Project 4-30A, Transportation Research Board of the National Academies, Washington, D.C., 2007, 93 pp.

Meininger, R.C. and S.J. Stokowski, "Wherefore Art Thou Aggregate Resources for Highways," FHWA-HRT-11-006, *Public Roads,* Vol. 75, No. 2, Sep./Oct. 2011 [Online]. Available: http://www.fhwa.dot.gov/publications/public roads/11septoct/06.cfm.

Mishra, D., E. Tutumluer, and A.A. Butt, "Quantifying Effects of Particle Shape and Type and Amount of Fines on Unbound Aggregate Performance Through Controlled Gradation," *Transportation Research Record: Journal of the Transportation Research Board, No. 2167,* Transportation Research Board of the National Academies, Washington, D.C., 2010a, pp. 61–71.

Mishra, D., E. Tutumluer, and Y. Xiao, "Particle Shape, Type and Amount of Fines, and Moisture Affecting Resilient Modulus Behavior of Unbound Aggregates," In *ASCE Geotechnical Special Publication No. 203, Paving Materials and Pavement Analysis, Proceedings Book of the ASCE GeoShanghai 2010 International Conference,* B. Huang, E. Tutumluer, I.L. Al-Qadi, J. Prozzi, and X. Shu, Eds., Shanghai, China, June 35, 2010, 2010b, pp. 279–287.

Mishra, D. and E. Tutumluer, "Field Performance Evaluations of Illinois Aggregates for Subgrade Replacement & Subbase—Phase II," Draft Final Report, Illinois Center for Transportation (ICT) R27-81 Project, University of Illinois Urbana–Champaign, Nov. 2011, 62 pp.

Mishra, D., *Aggregate Characteristics Affecting Response and Performance of Unsurfaced Pavements on Weak Subgrades,* Ph.D. Thesis, University of Illinois at Urbana–Champaign, 2012.

Muethel, R.W., *Calcium Carbonate Precipitate from Crushed Concrete,* Michigan Department of Transportation, Lansing, 1989.

Ooi, P.S.K., F. Rajabipour, A. Shafaatian, and S. Joo, "Forensic Investigation of Distressed Pavement Supported on a Base Course Containing Recycled Concrete Aggregate," *Transportation Research Record: Journal of the Transportation Research Board, No. 2253,* Transportation Research Board of the National Academies, Washington, D.C., 2011, pp. 22–31.

Ovik, J.M., B. Birgisson, and D.E. Newcomb, *Characterizing Seasonal Variations in Pavement Material Properties for Use in a Mechanistic-Empirical Design Procedure,* Final Report MN-RC-2000-35, Minnesota Department of Transportation, St. Paul, Dec. 2000, 191 pp.

Pan, T., E. Tutumluer, and J. Anochie-Boateng, "Aggregate Morphology Affecting Resilient Behavior of Unbound Granular Materials," *Transportation Research Record: Journal of the Transportation Research Board, 1952,* Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 12–20.

Paute, J.L., P. Hornych, and J.P. Benaben, "Comportement mécanique des graves non traitées," *Bulletin de Liaison des Laboratoires des Ponts et Chaussées 190,* 1994, pp. 27–38.

Puppala, A.J., S. Saride, S.K. Sirigiripet, and R. Williammee, "Evaluation of Cemented Quarry Fines as a Pavement Base Material," *ASCE Geotechnical Special Publication 177,* Geo-Congress, New Orleans, La., Feb. 2008, pp. 312–319.

Rao, C., E. Tutumluer, and I.T. Kim, "Quantification of Coarse Aggregate Angularity Based on Image Analysis," *Transportation Research Record: Journal of the Transportation Research Board, 1787,* Transportation Research Board of the National Academies, Washington, D.C., 2002, pp. 17–24.

Rathje E.M., et al., *Recycled Asphalt Pavement and Crushed Concrete Backfill: Results from Initial Durability and Geotechnical Tests,* Center for Transportation Research Report 4177-2, The University of Texas at Austin, 2002, 70 pp.

Roberts, F.L., P.S. Kandhal, E.R. Brown, D.-Y. Lee, and T.W. Kennedy, *Hot Mix Asphalt Materials, Mixture Design, and Construction,* 2nd ed., NAPA Education Foundation, Maryland, 1996, pp. 102–114.

Rohde, L., W.P. Nunez, and J.A.P. Ceratti, "Electric Arc Furnace Steel Slag Base Material for Low-Volume Roads," *Transportation Research Record: Journal of the Transportation Research Board, 1819,* Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 201–207.

Rowshanzamir, M.A., *Resilient Cross-Anisotropic Behavior of Granular Base Materials Under Repetitive Loading,* Ph.D. Thesis, School of Civil Engineering, University of New South Wales, Australia, 1995.

Rupnow, T.D., V.R. Schaefer, and D.J. White, "An Investigation of the Use of Limestone Screenings in Roadway Construction," CD-ROM, *Proceedings of 89th Annual Meeting of the Transportation Research Board,* Jan. 10–14, 2010, Washington, D.C.

Sadecki, R.W., G.P. Busacker, K.L. Moxness, K.C. Faruq, and L.G. Allen, An Investigation of Water Quality in Runoff from Stockpiles of Salvaged Concrete and Bituminous Paving, Report No. MN/PR-96/31, Minnesota Department of Transportation, St. Paul, 1996.

Saeed, A., J.W. Hall, Jr., and W. Barker, *NCHRP Report 453: Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers,* Transportation Research Board, National Research Council, Washington, D.C., 2001, 55 pp.

Saeed, A., *NCHRP Report 598: Performance-Related Tests of Recycled Aggregates for Use in Unbound Pavement Layers,* Transportation Research Board of the National Academies, Washington, D.C., 2008, 53 pp.

Seyhan, U. and E. Tutumluer, "Anisotropic Modular Ratios as Unbound Aggregate Performance Indicators," *Journal of Materials in Civil Engineering,* Vol. 14, No. 5, 2002, pp. 409–416.

Snyder, M.B. and J.E. Bruinsma, "Review of Studies Concerning Effects of Unbound Crushed Concrete Bases on PCC Pavement Drainage," *Transportation Research Record 1519,* Transportation Research Board, National Research Council, Washington, D.C., 1996, pp. 51–58.

Stroup-Gardiner, M. and T. Wattenberg-Komas, *NCHRP Synthesis 435: Recycled Materials and Byproducts in Highway Applications, Volume 6: Reclaimed Asphalt Pavement, Recycled Concrete Aggregate, and Construction Demolition Waste,* Transportation Research Board of the National Academies, Washington, D.C., 2013.

Taha, R., G. Ali, A. Basma, and O. Al-Turk, "Evaluation of Reclaimed Asphalt Pavement Aggregate in Road Bases and Subbases," *Transportation Research Record 1652,* Transportation Research Board, National Research Council, Washington, D.C., 1999, pp. 264–269.

Tamirisa, R., *Study of Highway Base/Subbase Aggregates That Cause Depositions of Calcareous "Tufa" in Drains,* Master's thesis, Department of Civil Engineering, University of Toledo, Ohio, 1993.

Tao, M., M.N. Louay, N.D. Munir, Z. Zhang, and Z. Wu, "Application of Shakedown Theory in Characterizing Traditional and Recycled Pavement Base Materials," *Journal of Transportation Engineering,* Mar. 2010, Vol. 136, No. 3, pp. 214–222.

Thom, N.H., *Design of Road Foundations,* Ph.D. Thesis, University of Nottingham, Department of Civil Engineering, May 1988, 307 pp.

Thom, N.H. and S.F. Brown, "Effect of Moisture on the Structural Performance of a Crushed-Limestone Road Base," *Transportation Research Record 1121,* Transportation Research Board, National Research Council, Washington, D.C., 1987, pp. 50–56.

Thom, N.H. and S.F. Brown, "The Effect of Grading and Density on the Mechanical Properties of a Crushed Dolomitic Limestone," *Proceedings of the 14th ARRB Conference,* Part 7, 1988, pp. 94–100.

Thompson, M.R. and Q.L. Robnett, "Resilient Properties of Subgrade Soils," *Transportation Engineering Journal,* ASCE, Vol. 105, No. TE1, 1979.

Thompson, M.R., and T. LaGrow, "A Proposed Conventional Flexible Pavement Thickness Design Procedure," Transportation Engineering Series No. 55, Civil Engineering Studies, University of Illinois at Urbana–Champaign, 1988.

Tutumluer, E. and U. Seyhan, "Neural Network Modeling of Anisotropic Aggregate Behavior from Repeated Load Triaxial Tests," *Transportation Research Record 1615,* Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 86–93.

Tutumluer, E. and U. Seyhan, "Effects of Fines Content on the Anisotropic Response and Characterization of Unbound Aggregate Bases," *Proceedings of the 5th International on Unbound Aggregates in Roads* (*UNBAR5*), *Unbound Aggregates in Road Construction,* A.R. Dawson, Ed., A.A. Balkema, University of Nottingham, U.K., 2000, pp. 153–161.

Tutumluer, E. and T. Pan, "Aggregate Morphology Affecting Strength and Permanent Deformation Behavior of Unbound Aggregate Materials," *Journal of Materials in Civil Engineering,* ASCE, Vol. 20, No. 9, Sep. 2008, pp. 1–11.

Tutumluer, E., D. Mishra, and A. Butt, *Characterization of Illinois Aggregates for Subgrade Replacement and Subbase,* Final Report, Illinois Center for Transportation (ICT) R27-1 Project, University of Illinois, Urbana–Champaign, Sep. 2009, 90 pp.

Uthus, L., E. Tutumluer, I. Horvli, and I. Hoff, "Influence of Grain Shape and Surface Texture on the Deformation Properties of Unbound Aggregates in Pavements," *International Journal of Pavements,* Vol. 6, No. 1–3, Jan.–May–Sep. 2007, pp. 75–88.

Van Niekerk, A.A., *Mechanical Behavior and Performance of Granular Bases and Sub-Bases in Pavements,* Ph.D. Thesis, Delft University of Technology, Delft, the Netherlands, 2002.

Willett, J.C., *2008 Stone (Crushed). U.S. Geological Survey Mineral Commodity Summaries,* pp. 158–159 [Online] Available: http://minerals.usgs.gov/minerals/pubs/commodity/stone_crushed/mcs-2008-stonc.pdf [accessed Jan. 8, 2013].

Willett, J.C., *2010 Minerals Yearbook: Stone, Crushed* [advance release], U.S. Geological Survey, U.S. Department of the Interior, Washington, D.C., Nov. 2011.

Wilburn, D.R. and T.G. Goonan, "Aggregates from Natural and Recycled Sources: Economic Assessments for Construction Applications-A Materials Flow Analysis," *U.S. Geological Survey Circular 1176,* U.S. Department of the Interior, Washington, D.C., 1998.

Woodbridge, M.E., "Use of Soft Limestone for Road-Base Construction in Belize," Seventh International Conference on Low-Volume Roads, Baton Rouge, La., 1999.

Woolf, D.O., "Results of Physical Tests of Road Building Aggregates: To January 1951," Physical Research Branch, Bureau of Public Roads, 1952.

Wu, Y., F. Parker, Jr., and K. Kandhal, *Aggregate Toughness/Abrasion Resistance and Durability/Soundness Tests Related to Asphalt Concrete Performance in Pavements,* NCAT Report No. 98-4, National Center for Asphalt Technology, Auburn, Ala., 1998.

Xiao, Y., E. Tutumluer, and J. Siekmeier, "Mechanistic-Empirical Evaluation of Aggregate Base and Granular Subbase Quality Affecting Flexible Pavement Performance in Minnesota," *Transportation Research Record: Journal of the Transportation Research Board, No. 2227,* Transportation Research Board of the National Academies, Washington, D.C., 2011, pp. 97–106.

Xiao, Y. and E. Tutumluer, *Best Value Granular Material for Road Foundations,* Final Report, Submitted to Minnesota Department of Transportation, St. Paul, Dec. 2011, 179 pp.

CHAPTER THREE

GRANULAR BASE AND SUBBASE CONSTRUCTION PRACTICES

INTRODUCTION

Aggregate storage, transportation, and construction practices are critical to ensuring adequate performance of constructed UAB and subbase layers under loading. Improper material handling and construction procedures often lead to aggregate segregation and/or degradation, ultimately resulting in a poorly compacted aggregate layer. Because unbound aggregate layers function primarily through interparticle load transmission at aggregate contact points, such poorly compacted layers may undergo excessive shear deformation, leading to pavement failure.

This chapter comprises an overview of common construction and material handling practices adopted by transportation agencies as far as UAB and subbase layers are concerned. Extensive review of published literature was conducted to identify different methods identified by researchers in the past as being adequate or inadequate for aggregate base and subbase construction. A survey of U.S. state and Canadian provincial agencies was conducted to gather information on the state of the practice on this topic, and an analysis of the findings presented to highlight areas where significant improvements are still needed. Moreover, applications of nonstandard or unconventional pavement types using unbound aggregate layers and related construction practices, such as the inverted pavement concept of a granular layer over a stiff layer at depth, are described in this chapter. The overall objective is to identify gaps in knowledge concerning the "effective practices" for UAB and subbase layer construction, along with research needs to address these gaps.

IMPORTANCE OF STANDARDIZED CONSTRUCTION SPECIFICATIONS

A "pocket-sized" handbook published by the NSSGA (1989) contains important guidelines for UAB construction. Similarly, different transportation agencies have adopted different guidelines to help the construction of "good quality" base and subbase layers. Apart from providing the contractors with a definite set of guidelines to be followed during construction, these guidelines help the field engineers with QA of constructed pavement layers. However, the survey of state and Canadian provincial transportation agencies conducted under the scope of this synthesis study indicated that only 37% of the responding agencies (17 of 46) currently have specific guidelines regarding the transportation and storage (stockpiling) of aggregate materials for base and subbase construction. Approximately 25 agencies reported not having any such guidelines, whereas the remaining four agencies indicated the presence of generic guidelines without specific instructions.

AGGREGATE STORAGE AND CONSTRUCTION PRACTICES AFFECTING CONSTRUCTED LAYER PERFORMANCE

To fulfill the overall objectives of this synthesis study, it is important to first present a summary of material handling and construction practices that have been identified as adequate or inadequate as far as ensuring the construction of good quality unbound aggregate pavement layers. Inadequate material handling and construction practices may lead to aggregate segregation and/or degradation affecting the gradation or particle size distribution of the constructed aggregate layer. The following sections discuss different material storage and construction practices that may lead to the problems of aggregate segregation and degradation.

Aggregate Stockpiling as a Source of Segregation

The Aggregate Handbook defines aggregate segregation as the separation of one size of particles from a mass of particles of different sizes, caused by the methods used to mix, transport, handle or store the aggregate in the plant under conditions favoring nonrandom distribution of the aggregate sizes (Barksdale 1991). Certain practices magnify the segregation problem and thus are best restricted by transportation agencies. One possible source of segregation is during the formation of conical stockpiles by dumping material using a conveyor belt. As the aggregate is transported by a conveyor belt, vibration and motion of the belt causes the fine particles to settle to the bottom of the material stream, whereas coarse particles remain at the top. These coarse particles have a higher velocity at the end of the conveyor, and are thrown a greater distance to the stockpile. In addition, the coarser particles hit the front face of the stockpile with a greater momentum and roll down the outer edge of the pile, creating overrun (an accumulation of particles at the pile's bottom edge or toe). Fine particles, which have settled against the surface of the conveyor belt, tend to cling to the belt and drop to the back face of the pile. The resulting stockpile is segregated, with coarse particles settled at the toes, and fine particles in the center portion of the pile.

"Material overrun," particles (regardless of size) moving down the side of the stockpile, is another major source of segregation in stockpiles. As the material moves down the side of the stockpile, larger particles tend to move down to the bottom (owing to higher momentum), whereas finer materials tend to settle into the side of the pile. Such spatial distribution of aggregate particles of different sizes at different portions of the stockpile results in pronounced segregation. Figure 24 shows the spatial distribution of different aggregate particle sizes in a segregated stockpile (Nohl and Domnick 2000).

Materials with a large variation in particle size usually undergo higher degrees of segregation as a result of improper stockpiling practices. Usually aggregate materials in which the ratio of the largest to the smallest particle size exceeds 2:1 are likely to experience segregation problems during stockpiling (Nohl and Domnick 2000). From in-depth investigation of aggregate stockpiling practices, Miller Warden Associates (1964) observed that flat-mixed piles formed by the use of a crane bucket was the only stockpiling method that resulted in an insignificant amount of segregation. The most commonly used truck dumping method, although economical, was found to cause significant segregation of aggregates. Majidzadeh and Brahma (1969) studied different stages in the aggregate handling process, such as (1) initial material fabrication, (2) producer stockpile, (3) truck transportation, and (4) job-site stockpile, to establish the severity of segregation problem at these different stages and also observed that the segregation problem increased as the material approached the job site from the production plant.

Creating stockpiles using the "windrow concept" is one of the alternatives available for storage of materials where segregation is a likely problem. Involving the creation of "miniature stockpiles" in layers, windrow stockpiles can be built effectively using a telescoping conveyor that can move laterally as well as along the direction of the conveyor to create the stockpile in layers. Although individual "miniature stockpiles" in a windrow pile still undergo segregation, such stockpiles are said to have better "segregation resolution" because the segregation pattern repeats itself in smaller intervals. Figure 25 shows schematics of (a) the configuration of a windrow pile formed using a telescoping conveyor and (b) segregation resolution in a windrow pile (Nohl and Domnick 2000).

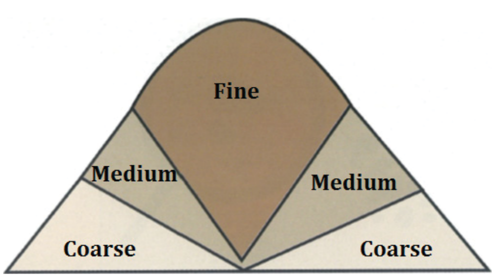


FIGURE 24 Spatial distribution of particle gradation in a stockpile (modified from Nohl and Domnick 2000).

Stockpiling Practices by Different Agencies

Different transportation agencies adopt different stockpiling practices to minimize aggregate segregation. Specifications are often provided to aggregate manufacturers and contractors mentioning the desired storage and stockpiling practices. For example, the New Hampshire Department of Transportation standard specifications on base courses (http://www.nh.gov/dot/org/projectdevelopment/highwaydesign/specifications/documents/2010_Division_300.pdf) include the following requirements for aggregate stockpiling:

> Stockpiles shall be constructed in layers that minimize segregation. The desired optimum thickness of layers is 6 ft. (1.8 m) and in no instance shall the layer be more than 10 ft. (3 m). Each layer shall be completed before the next layer is started. Construction of stockpiles by direct use of a fixed conveyor belt system or by dumping over a bank will not be permitted.

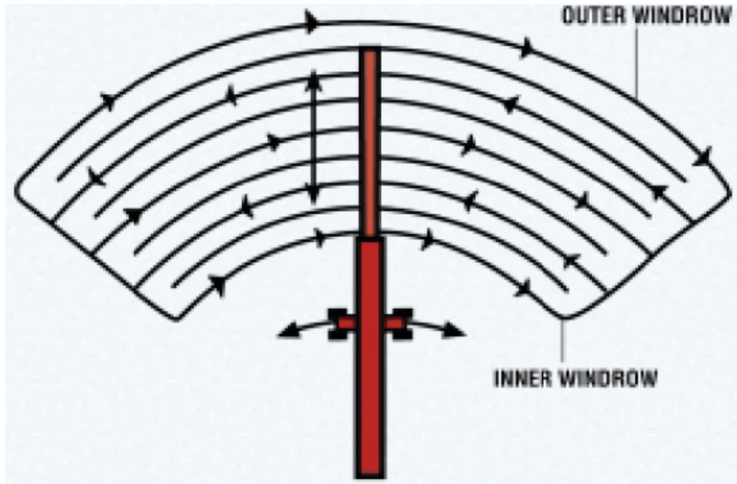
Similarly, stockpiling practices recommended by the Alabama Department of Transportation (http://www.dot.state.al.us/mtweb/Testing/testing_manual/doc/pro/ALDOT175.pdf) include

- Stockpiles need to be placed on firm, well-drained ground that is free of any material that could cause contamination.
- Stockpiles should be built in layers of uniform thickness and not in cone-shaped piles that result in segregation of piles.
- After the first layer of the stockpile is placed, it is important that heavy transporting equipment not be allowed to run on top of this layer because this tends to degrade the aggregate by grinding the particles together, also contaminating the aggregate with mud and other deleterious substances from the wheels or tracks of the vehicle.
- If the stockpile is to be constructed in more than one layer in height, the aggregate should be dumped in a small pile at the base of the stockpile and then moved over to the stockpiled layer in place by a crane equipped with a clamshell, front-end loader or bulldozer equipped with large pneumatic tires.

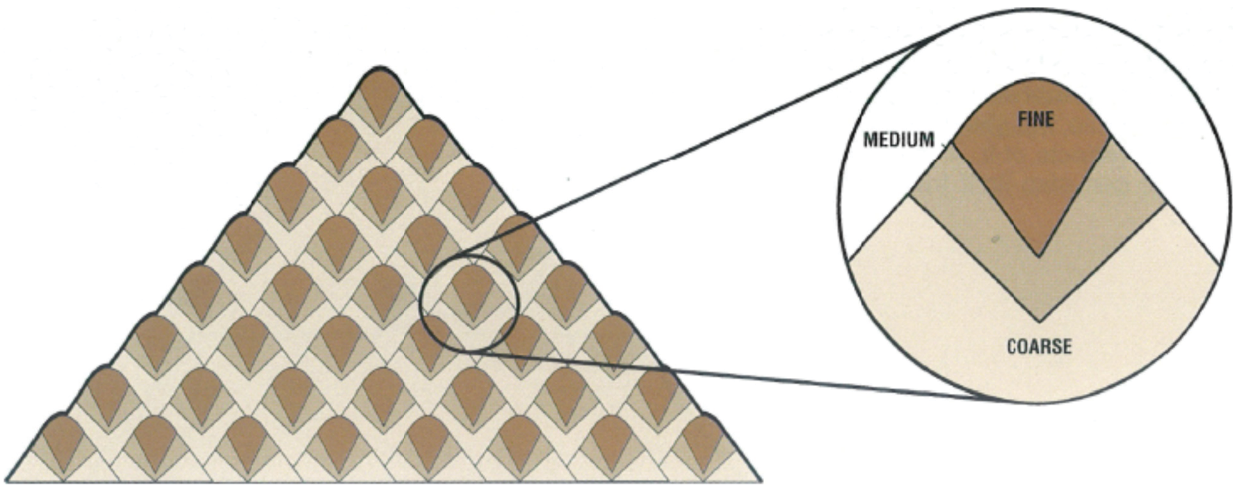
The standard operating procedures recommended by the GDOT (http://www.dot.ga.gov/doingbusiness/Materials/Documents/StudyGuide9_22_04.pdf) provide graphical representations of the prohibited and recommended practices as far as aggregate stockpiling and aggregate sampling from different types of stockpiles are concerned (see Figure 26).

Construction Practices as a Source of Segregation

Different construction practices can contribute significantly to aggregate segregation and therefore should be controlled through the agency specifications. White et al. (2004) observed that aggregate trimming operations are likely to contribute the most to the segregation problem as they shake the aggregate, causing fine particles to migrate to the bottom of the layer. Subsequent removal of the top aggregate by the trimmer leaves the fine aggregate behind, resulting in uneven spatial distribution



(a)



(b)

FIGURE 25 (*a*) Windrow configuration and (*b*) segregation resolution in a windrow pile (Nohl and Domnick 2000).

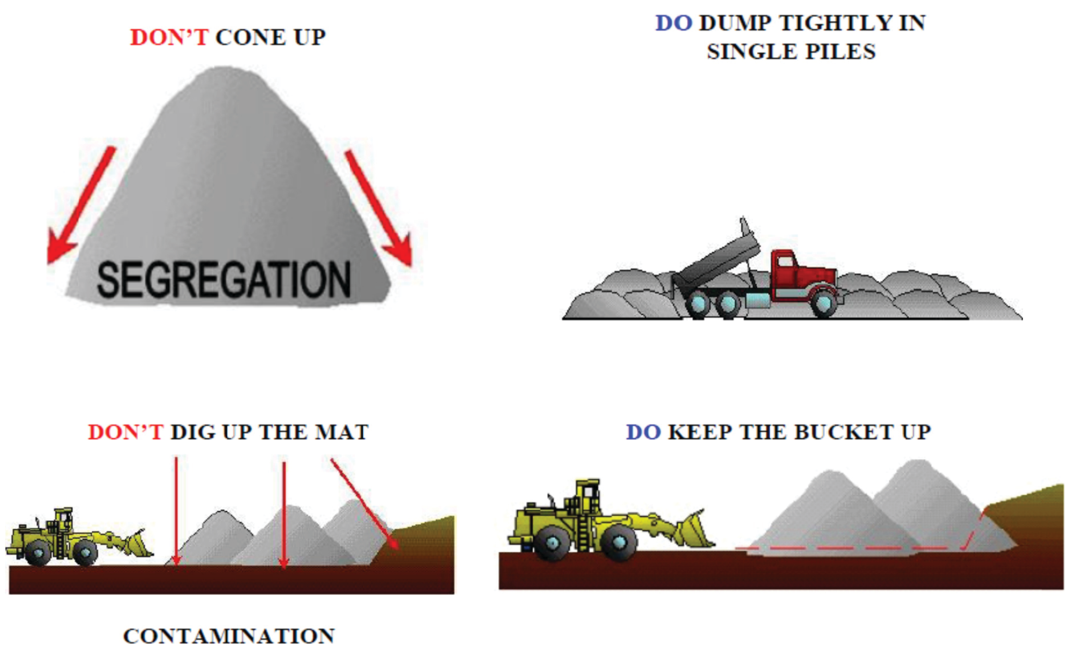


FIGURE 26 Example specifications regarding aggregate stockpiling (Georgia DOT).

of aggregate particle sizes in the constructed layer. They also observed that low moisture in the aggregate mostly corresponded to increased segregation as a result of poor adhesion between finer and larger particles.

Through in situ testing of full-scale unbound aggregate test sections, they suggested changes to construction operations to limit spatial variations in constructed layer properties. These changes include (1) limiting movement of aggregate by primarily transporting aggregate transversely, rather than longitudinally, and (2) moistening the aggregate before trimming to reduce fines migration. Williamson and Yoder (1967) studied the achieved compaction levels in different rigid pavement subbase layer constructions in the state of Indiana and concluded that the lack of compaction could be attributed to nonuniform aggregate gradations in the constructed layer, which is an indicator of segregation during the construction process.

Aggregate Degradation and Possible Sources

Aggregate degradation is defined as the breakdown of an aggregate into smaller particles (Barksdale 1991). Aggregate degradation can occur during the process of aggregate stockpiling or during the placement and compaction of aggregate base and subbase layers. Formation of stockpiles by pushing of aggregates using dozers and handling of the aggregates during different stages of construction both may result in degradation of the larger particles into smaller fractions. Although the problem of degradation is not as severe for quarries excavating hard rock formations, the problem can be significant for operations dealing with "softer" parent rocks. Some quarries compensate for the potential degradation by producing aggregate sizes that are coarser than the target aggregate size. Moreover, different agencies impose different restrictions on the type of construction vehicle allowed to operate on aggregate stockpiles.

Compaction of constructed layers may impose heavy loads that cause aggregate degradation. Aughenbaugh et al. (1963) indicated that degradation is dominant in the top lift of an aggregate layer. Thus, the height of a layer during compaction may contribute toward nonuniform aggregate gradation resulting from degradation of individual particles. This is particularly critical for aggregate layers constructed with large lift thicknesses. To ensure adequate compaction at greater depths, high compactive energies need to be imparted on the layer surface. Such high compactive energies result in significant crushing of aggregate particles near the layer surface, changing the gradation and thus achieved density. Density-based compaction control techniques may give erroneous indications of layer compaction in such cases because density measuring devices, such as the nuclear density gauge, can measure the compaction level for only the upper few inches (typically 12 in. for a nuclear density gauge) and do not check the compaction levels for deeper sections in the layer. Thus, it is important to control the amount of energy imparted to the aggregate layer during the compaction process through adjusting the amplitude and frequency of impacts applied by vibratory rollers.

Key Lessons

- Aggregate segregation and deterioration can be minimized through proper stockpiling and construction practices.
- Stockpiling of aggregates using the windrow concept has been proven to be the most efficient practice as far as minimizing segregation is concerned.

CONSTRUCTION LIFT THICKNESS AND ITS EFFECT ON COMPACTABILITY

Background

One of the primary factors affecting the performance of UAB and subbase layers is the DOC. Aggregate materials received from the source are placed on the prepared subgrade and compacted to the design layer thicknesses. Generally, upon compaction an unbound aggregate layer loses approximately one-third of its loose placement depth (Barksdale 1991; Saunders 1997). Specifications require the compaction to be carried out immediately after placement of the aggregate material while the gradation and moisture content are still at the specified values (NSSGA 1989). Maximum allowable lift thicknesses are usually specified during the construction of unbound aggregate layers to ensure adequate compaction, which is critical to pavement layer performance under loading. Saunders (1997) reports that the maximum lift thicknesses specified by agencies most likely were established in the early days of highway construction, when only static rollers or limited vibratory rollers were available. Saunders also indicates that in view of the modern construction equipment, these maximum lift thickness thresholds most likely are on the conservative side. Wells and Adams (1997) successfully constructed aggregate base courses in single-lift depths of 10 and 12 in., which were greater than the 8-in. maximum allowed by North Carolina Department of Transportation (NCDOT) specifications. Similarly, Womack (1997) reports successful construction of aggregate base courses in Virginia with 10-in. thick lifts, which were greater than the maximum construction lift thicknesses specified by the Virginia Department of Transportation (VDOT) at that time.

Researchers have since focused on evaluating the effectiveness of layer compaction when aggregate layers are constructed with large lift thicknesses. Bueno et al. (1998) constructed test pads in Texas and Georgia using crushed limestone and crushed granite, respectively, with lift thicknesses ranging from 6 to 21 in. Three test strips were constructed in Georgia with different lift thicknesses. A "target test strip" was compacted in two lifts: one 178-mm (7-in.) compacted lift under a 152-mm (6-in.) compacted lift. Two other test sections (sections 1 and 2) were both compacted in one single lift to a final compacted thickness of 330 mm (13 in.). In-place density measurements indicated compaction levels of 102.5%, 103.9%, and 103.3% for the target Strip, Test Section 1, and Test Section 2, respectively, indicating that adequate compaction levels could be achieved even for higher construction lift thicknesses.

Constructing test pads in Texas, Bueno et al. (1998 and 1999) observed that higher densities could sometimes be achieved for 457-mm (18-in.) and 584-mm (23-in.) lifts compared with those achieved for 305-mm (12-in.) lifts. In general, dry densities were found to increase with depth into the compacted base, illustrating that the lower parts of the aggregate base course were being compacted even for greater lift thicknesses. This trend was supported by the shear wave velocity profiles obtained from Spectral analysis of surface waves (SASW) testing. From SASW data, they observed that stiffness of a graded aggregate base (GAB) course was sensitive to moisture variations and concluded this sensitivity likely was caused by changes of effective confining stress, which occur when the material is wetted or dried. Overall, findings from this study clearly indicate that thicker single lifts could be compacted equally well or sometimes better than are the thinner aggregate layers commonly constructed by agencies.

Allen et al. (1998) conducted a survey of all state transportation agencies and found that 12 of 36 responding states allowed a maximum lift thickness of 6 in. or less, one state allowed 7-in. lifts, and 16 states allowed 8-in. lifts. Only three states allowed thicker lifts (Washington, 9-in.; North Carolina, 10-in.; and Maine, 12-in.). The survey of state and Canadian provincial agencies conducted under the scope of the current synthesis study found that 11 of 46 responding agencies allowed construction lift thicknesses of 8 in. The current limits for construction lift thickness in North Carolina and Maine were the same as those reported by Allen et al. (1998). Note that as of 2012, 17 agencies still restricted the maximum lift thickness to 6 in. Figure 27 summarizes all the data collected from the survey respondent states and Canadian provincial agencies. From the figure it is evident that despite several research and trial studies demonstrating the effectiveness of aggregate layer construction with larger lift thicknesses, the current practices in state and Canadian provincial transportation agencies still use a conservative approach in this regard. Thus, more research and demonstration projects need to focus on the advantages and disadvantages (if any) of higher construction lift thicknesses. Such studies will also help harmonize the construction practices throughout the United States and Canada.

From successful implementation of thick single-lift aggregate base and subbase layer construction, Allen et al. (1998) recommended the following changes to unbound aggregate layer construction practices:

- **Equipment:** Mixing is to be accomplished by stationary plant, such as a pugmill, or by road mixing using a pugmill or rotary mixer. Mechanical spreaders should be used to avoid segregation and achieve grade control. Suitable vibratory compaction equipment should be employed.
- **Mixing and Transporting:** The aggregates and water should be plant mixed (stationary or roadway) to the range of optimum moisture plus 1% or minus 2% and transported to the job site so as to avoid segregation and loss of moisture.
- **Spreading:** The material is to be placed at the specified moisture content to the required thickness and cross section by an approved mechanical spreader. At the engineer's discretion, the contractor may choose to construct a 500-ft long test section to demonstrate achieving adequate compaction without particle degradation for lift thicknesses in excess of 13 in. The engineer may allow thicker lifts on the basis of the test section results.

Optimum Construction Lift Thickness

As observed from the survey results, no consensus exists among transportation agencies with regard to the maximum allowed construction lift thickness for UAB/subbase layers. From extensive review of literature conducted under the scope of this synthesis study, it was observed that most research studies and trial implementation projects could successfully compact 12-in. thick aggregate layers while achieving desired compaction levels. Thus, it is suggested that 12-in. aggregate lifts be standardized as "optimum construction practice" for UAB/subbase layers. Given adequate support conditions, construction of unbound aggregate layers in such thick lifts could sufficiently expedite

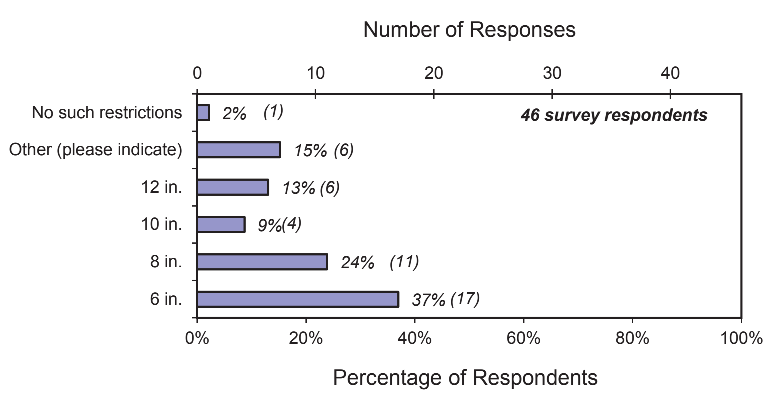


FIGURE 27 Maximum construction lift thickness allowed for unbound aggregate layers.

Key Lessons

- No common practice exists among transportation agencies as far as the maximum construction lift thickness of UAB/subbase layers is concerned.
- Maximum construction lift thickness value for UAB/subbase layers are best based on project "test-strip" sections using the specific materials and equipment.
- From extensive review of literature and state practices, this synthesis study suggests an optimum construction lift thickness of 12 in. for UAB/subbase layers. Note that this suggestion is based on the assumption that the UAB/subbase layer to be constructed is at least 12-in. thick. Moreover, the DOC achieved is contingent upon the use of adequate equipment by the contractor.

the construction process while ensuring adequate compaction levels. For aggregate layers being constructed over "firm" prepared subgrades (often represented by subgrade CBR > 8%), the compaction of 12-in. thick layers may be possible. However, reduced lift thicknesses may need to be adopted for "weaker" subgrade support conditions (subgrade CBR < 8%).

DOCUMENTED AGGREGATE BASE AND SUBBASE LAYER CONSTRUCTION PRACTICES

Figures 28 to 30 summarize agency responses to various currently adopted aggregate base and subbase layer construction practices. Figure 28 highlights that 52.2% of the respondent agencies allow construction of two functionally different aggregate layers on top of each other, such as an OGDL underlain by a dense-graded aggregate subbase, in pavements. Furthermore, 66.7% of the respondent agencies do not separate two unbound aggregate layers by any kind of constructed aggregate separation or filter layers (see Figure 29). Figure 30

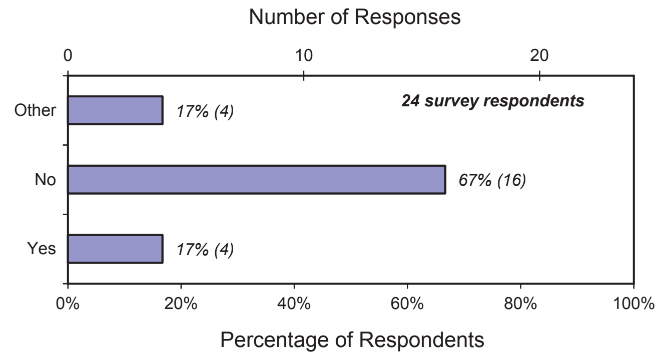


FIGURE 29 Agency responses to whether the two unbound aggregate layers are separated by any kind of constructed aggregate separation/filter layers.

indicates that 65.2% of the respondent agencies allow construction of unbound aggregate layers directly over or under pavement layers stabilized or treated with lime, fly ash, cement, or bitumen. Note that these findings have implications on some of the domestic and foreign innovative pavement construction practices, such as the construction of "inverted pavements."

INVERTED PAVEMENTS

Sustainable application of unbound aggregate structural layers in pavements would improve the designs of low, medium, and moderately high volume roads while reducing the dependence on asphalt (and thus crude oil) for pavement construction. Such an alternative is offered by an inverted pavement section that consists of an unstabilized crushed stone base or GAB sandwiched between a lower cement-stabilized layer and a thin upper asphalt concrete surfacing.

Conceptual Background

Conventional pavement systems rely on a combination of asphalt concrete or PCC and aggregate base components to transfer load to the subgrade. All three components use

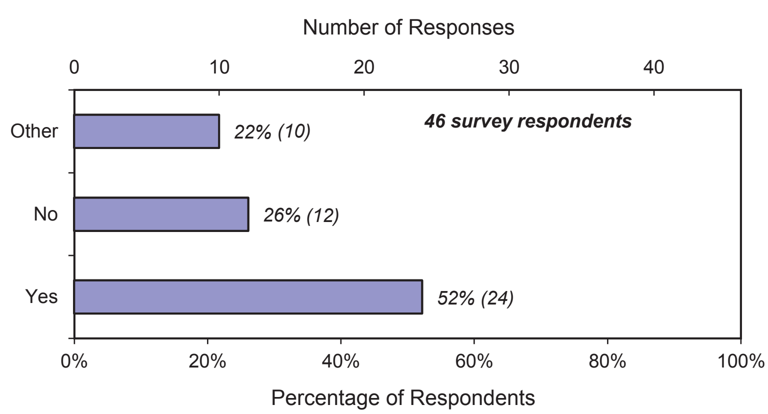


FIGURE 28 Agency responses to whether multiple unbound aggregate layers are allowed to be placed on top of each other.

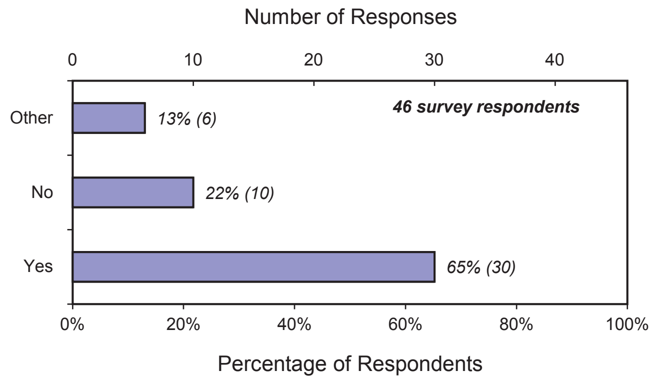


FIGURE 30 Agency responses to whether the construction of unbound aggregate layers is allowed over or under pavement layers stabilized or treated with lime, fly ash, cement, or bitumen.

aggregates as their primary constituent. Classic pavement design places higher modulus, more durable layers toward the surface. Inverted pavement is a composite system composed of asphalt top layer(s) and a well-compacted unbound crushed stone base layer over a stiffer bound subbase that is usually cement treated. Mechanistically, this configuration provides a stronger reaction platform than unbound subgrades or subbases, allowing increased granular base compaction during construction, and it also has the potential to take advantage of the compressive stresses induced in the granular aggregate base owing to the presence of the stiff underlying layer. This pavement design philosophy potentially offers economic advantages by requiring less asphalt concrete and placing the burden of strength and structural performance on relatively less expensive underlying layers.

Inverted pavements were first introduced in South Africa and involved the construction of thick crushed stone base layers over stabilized subbase layers. The superior performing crushed stone base layers used in South African inverted pavements are also known as "G1" base layers (Horne et al. 1997; Jooste and Sampson 2005; De Beer 2012). These pavements are also called stone interlayer pavements, G1-base pavements, inverted base pavements, sandwich pavements, and upside down pavements (Lewis et al. 2012). Figure 31 shows the layer configuration of a typical inverted pavement structure.

As shown in Figure 31, the HMA layers in inverted pavement sections often are very thin, so their contribution to the structural capacity of the pavements often is not significant. Primarily, these surface layers provide a smooth ride quality and protect the underlying pavement layers from water infiltration. The unbound aggregate layer is the primary load-bearing layer in inverted pavement structures. Summarizing the construction practices and layer configurations of these pavement systems, De Beer (2012) presented the following definition for inverted pavements:

> A structural pavement system, where the static modulus of the unbound base layer is lower compared with the supporting (mainly lightly cementitious) subbase layers. Unbound base layer (crushed rock) of extremely high bearing capacity is usually covered with 12 mm to 50 mm asphalt layer for sealing and functional properties.

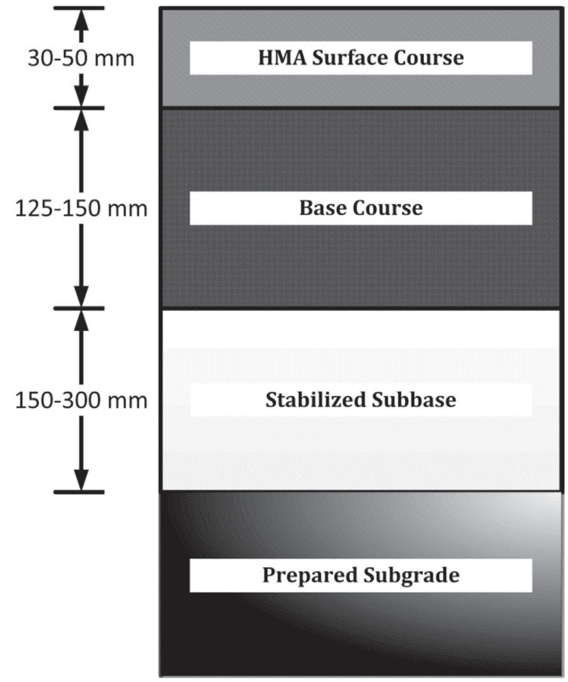


FIGURE 31 Layer configuration of a typical inverted pavement section.

Owing to the reduced thickness of the HMA surface layers, these pavement systems are cost-effective alternatives for high-performance pavement structures. The primary advantages of inverted pavements include (1) better compaction of unstabilized materials placed over the stabilized layers; (2) optimum use of unstabilized crushed stone; and (3) elimination or significant reduction in reflective cracking in the pavement structures (Barksdale and Todres 1983).

Response Mechanism

The UAB is primarily a structural load-carrying component in inverted pavement sections. When properly compacted, the UAB causes lateral dissipation of traffic-induced stresses through interparticle contact points. The stiff UAB and the cement-stabilized subbase combined result in a significant reduction in the vertical compressive stress levels on top of the subgrade, thus eliminating chances of pavement failure because of subgrade rutting. However, the UAB in an inverted pavement structure is subjected to considerably higher stresses to make the base layer prone to rutting, a potential failure mechanism for inverted pavement sections. Thus, construction procedures for inverted pavements aim to eliminate rut accumulation within the UAB layers through innovative compaction procedures.

The thin HMA surface typically considered in an inverted pavement section induces considerably high stress states within the aggregate base under wheel loading. Owing to the stress-hardening nature of unbound aggregates, these high stress states often lead to the aggregate layer developing high elastic modulus values, often on the order of 689 MPa or 100 ksi (Maree et al. 1982a, 1982b; O'Neil et al. 1992). Such high modulus levels achieved in the aggregate bases of inverted pavement sections would help better dissipate the traffic-induced stresses with depth. Moreover, the presence of the stiffer subbase layer in an inverted pavement section causes the neutral axis in bending to fall below the aggregate base layer. This results in the surface layers and the UAB layers performing mainly under compression. Accordingly, the stiffness profile in inverted pavement structures prevents the development of tensile stresses in the UAB even if a linear model is used to represent it (Cortes 2010). The stiff aggregate base layer also leads to a reduction in the tensile stresses at the aggregate base course-HMA surface interface, thus significantly reducing the chances for reflective cracking occurring in these pavement structures.

Material Specifications and Construction Procedure

Material Specifications

The aggregate material to be used in the base course of an inverted pavement structure is obtained from crushing of hard, sound, durable, and unweathered parent rock. All the faces of the aggregate particles are required to be fractured. South African G1 base specifications allow the material gradation to be adjusted only through the addition of fines produced from the crushing of the original parent rock.

Table 3 lists the (a) particle size distribution and (b) other material quality specifications used in South Africa for use in G1 base course applications (TRH 1985; Buchanan 2010).

Note that the gradation requirements listed in Table 3a are based on restricting the "*n*" values in the Fuller's or Talbot's equation (as defined in Equation 1) between 0.33 and 0.50. Note that in Equation 1, P is the percentage (%) of material by weight finer than the sieve size being considered; d is the sieve size being considered; D is the maximum aggregate particle size in the current matrix; and n is a parameter that adjusts the gradation curve for fineness or coarseness.

$$P = \left(\frac{d}{D}\right)^n \times 100 \qquad (1)$$

Construction Procedure

Compaction of the UAB layer in an inverted pavement structure is the most critical step during its construction to ensure that individual layers perform as desired. The UAB is constructed on top of a stabilized subbase, which provides a solid construction platform for the placement and compaction of the UAB layer and ensures that adequate density levels can be achieved. The DOC achieved in the UAB layer is dependent on the energy applied, as well as the initial and final gradations of the aggregate material used.

TABLE 3
RECOMMENDED PARTICLE SIZE DISTRIBUTION RANGE FOR SOUTH AFRICAN G1 BASE

(*a*)

Sieve Size (mm)	Sieve Size (in.)	Percent Passing	
		G1, 37.5 NMS	G1, 26.5 NMS
50	2.0		
37.5	1.5	100	
26.5	~1.0	84–94	100
19.0	¾	71–84	85–95
13.2	~1/2	59–75	74–84
4.75	#4	36–53	42–60
2.00	#10	23–40	27–45
0.425	#40	11–24	13–27
0.075	#200	4–12	5–12

(*b*)

Aggregate Material Property	Specified Threshold Values
Minimum 10% FACT[a]	110
Maximum aggregate crushing value[b]	29%
Liquid limit	<25
Linear shrinkage[c]	<2%
Plasticity index (PI)	<4

[a] 10% FACT (fines aggregate crushing value) is the force in kilonewtons required to crush a sample of aggregate passing the 13.2-mm and retained on the 9.5-mm sieve so that 10% of the total test sample will pass a 2.36-mm sieve.

[b] The aggregate crushing value (ACV) of an aggregate is the mass of material, expressed as a percentage of the test sample that is crushed finer than a 2.36-mm sieve when a sample of aggregate passing the 13.2-mm and retained on the 9.50-mm sieve is subjected to crushing under a gradually applied compressive load of 400 kN.

[c] The linear shrinkage of a soil for the moisture content equivalent to the liquid limit is the decrease in one dimension, expressed as a percentage of the original dimension of the soil mass, when the moisture content is reduced from the liquid limit to an oven-dry state.

Sources: TRH 1985; Buchanan 2010.

The compaction of unbound aggregate layers in the South African inverted pavement structures involves the following two phases: standard compaction phase and particle interlocking or slushing phase. The standard compaction phase is carried out using a combination of grid rollers, vibratory rollers, and pneumatic tire rollers. Commonly two to three passes of the grid roller are used to gently knead the aggregate layer into shape. Subsequently, the vibratory roller is used to compact the layer to 85% of apparent solid density. It is important to note that the amplitude and frequency of vibration need to be strictly monitored during this phase because too much vibration can easily lead to "de-densification" of the aggregate matrix. Moreover, extreme care is to be exercised to prevent the breakage of individual aggregate particles under the vibratory roller. The aggregate moisture content usually is maintained near the "optimum" conditions during this phase of compaction to aid the rearrangement of individual aggregate particles into a densely packed matrix. The fines fraction in the aggregate matrix plays a critical role during this phase by lubricating the aggregate contact points. Thus, it is important that the aggregate material used in the construction of these superior performing base layers contain the adequate amount of fines. A rule of thumb used in the construction of South African G1 base course layers is that OMC values less than 4% are indicative of too few fines in the aggregate matrix, whereas OMC values higher than 6% are indicative of too many fines (De Beer 2012).

The second phase of compaction involves consolidating the material under saturated conditions by expelling or "slushing" out the excess fines material from the matrix, allowing the larger particles to interlock into a "superdense" matrix. The fines serve as lubricants to ensure reorientation and interlocking of the larger particles into a superdense matrix. This "washing out" of the fines, accompanied by compaction, is continued until the water draining from the pavement becomes colorless and does not contain any trace of excess fines (De Beer 2012). A pneumatic tire roller passing over the aggregate layer without leaving any indentations is used as an indicator of the achievement of adequate compaction. South African specifications require achieved density to be greater than 88% of solid particle density (assuming solid rock for the equal volume with no voids).

It is usual to place the prime coat and HMA surfacing layer immediately after compaction of the UAB layer. This is primarily because the aggregate base layer is noncohesive in nature, and the aggregate matrix may get disturbed upon exposure to direct application of traffic loads and weathering.

Previous Findings on the Benefits of Inverted Pavements

The first application of inverted pavements in the United States can be traced back to 1954 in New Mexico (Barksdale and Todres 1983). These initial inverted pavement sections involved the overlaying of several badly broken concrete pavements with 152 mm (6 in.) of unstabilized granular base, and 51 mm (2 in.) of asphalt concrete. Johnson (1960) reported that after six years of heavy traffic, no reflection cracking or significant rutting had developed in the test sections.

Subsequently, two experimental roads were constructed in New Mexico in about 1960, consisting of a 76-mm (3-in.) asphalt concrete surfacing, 152-mm (6-in.) granular base, and a 152-mm (6-in.) granular subbase treated with 4% cement.

U.S. Army Corps of Engineers Experience

The U.S. Corps of Engineers studied the behavior of the various layers in flexible pavement structures having lime-stabilized and cement-stabilized subbases: that is, inverted base type structures (Ahlvin et al. 1971; Barker et al. 1973; Grau 1973). The objective of the study was to measure the mechanical response of full-scale pavement structures and compare the results against predictions from layered elastic theory and other available constitutive models. Two inverted base pavement structures were investigated, both composed of a 90-mm asphalt concrete layer, a 150-mm crushed limestone base, a 380-mm stabilized clay subbase, and a clay subgrade (CBR of 4%). The structures were subjected to traffic under controlled conditions while monitoring displacements and stresses at key locations (Ahlvin et al. 1971; Barker et al. 1973; Grau 1973). Linear elastic analyses failed to adequately predict the measured stresses and strains in different layers and the plastic subgrade deformation. The performance of the inverted pavement structures was found to be influenced by the stiffness and tensile strength of the cement-treated base. This study highlighted the importance of a comprehensive material characterization and numerical implementation through appropriate constitutive models. Furthermore, it urged the development of laboratory tests capable of simulating field conditions and the introduction of nonlinear models in numerical simulations (Barker et al. 1973).

Barksdale and Todres

Barksdale and Todres (1983) constructed 12 laboratory-scale instrumented pavement structures and cyclically loaded them to failure under controlled environmental conditions. Among conventional flexible pavement and full-depth asphalt pavement test sections, they also tested two inverted pavement test sections made of 89-mm thick asphalt concrete layers over 203-mm thick unbound aggregate layers (well-graded granitic gneiss), over a 150-mm thick cement-stabilized subbase, over a micaceous nonplastic silty sand subgrade. One inverted pavement section had a 152-mm (6-in.) thick cement stabilized crushed stone subbase; the other had a 152-mm (6-in.) thick cement-treated, silty sand subbase. It was found that the cement-treated base facilitated compaction in inverted structures leading to denser unbound aggregate layers (Barksdale 1984).

The pavement sections were subjected to a 28.9-kN cyclic load for the first 2×10^6 repetitions, followed by cyclic application of a 33.4-kN load until failure. Monitoring the performance of the test sections under loading, Barksdale and Todres observed that the two inverted pavement sections outperformed equivalent pavement structures in terms of lower resilient surface displacements, reduced transferred compressive stress onto the subgrade, and less tensile radial strain at the bottom of the asphalt concrete layer (Barksdale and Todres 1983; Avellandeda 2010). The superior mechanical performance of the inverted pavement structures was clearly reflected from the significantly higher number of load cycles to failure (3.6×10^6 and 4.4×10^6) compared with the best performing conventional flexible pavement section (Barksdale 1984; Tutumluer and Barksdale 1995).

Tutumluer and Barksdale

Tutumluer and Barksdale (1995) conducted numerical modeling of the two full-scale instrumented inverted pavement sections tested by Barksdale and Todres (1983), and made the following observations:

- Cement-stabilized inverted sections can successfully withstand large numbers of heavy loadings through
  - Lower vertical subgrade stresses owing to the "beam action" of the stiff base layer;
  - Lower tensile strain at the bottom of the asphalt layer; and
  - Lower resilient surface deflections.
- The upper portion of the cement-treated subbase and almost all of the unstabilized crushed stone base near the load were in horizontal compression. The bottom half of the subbase and a thin layer on top of the subgrade were in horizontal tension.
- Presence of the cement-stabilized layer beneath the aggregate base resulted in horizontal compressive stresses of magnitudes ranging from 0 to 110 kPa (0 to 16 psi) in the unstabilized crushed stone base. This was probably a major factor contributing to the lower permanent deformation and higher resilient moduli of these base layers as observed from laboratory testing.
- From sensitivity analyses conducted using the GT-PAVE finite element (FE) program, they observed that the optimum and economical inverted pavement section constructed over a weak subgrade would consist of an unstabilized aggregate base 152 mm (6 in.) thick and a 152-mm to 203-mm (6-in. to 8-in.) thick cement-stabilized subbase.

Lafarge Quarry Access Road —Morgan County, Georgia

Two 122-m (400-ft.) long inverted pavement test sections were constructed on a new access road at the Lafarge Building Materials quarry near Madison, Georgia, in 2001. Both test sections had a 200-mm (8-in.) thick cement-treated base layer, a 150-mm (6-in.) thick GAB layer, and a 75-mm (3-in.) thick HMA layer. The only difference between the two inverted pavement sections was in the construction of the GAB layer: the first section was constructed using the South African "slushing" technique, whereas the second section was constructed using conventional construction methods.

Terrell et al. (2003a, b) conducted miniaturized versions of traditional cross-hole and downhole seismic tests to determine the stiffnesses of each base layer. Horizontally propagating compression and shear waves were measured under four different loading conditions to determine Young's moduli and Poisson's ratios of the base. An increase in stiffness with an increase in load was measured. In addition, it was found that the Georgia and South Africa sections had similar stiffnesses. Surprisingly, the traditional section was found to be somewhat stiffer than the other sections. This higher stiffness was thought to be caused by a prolonged period of compaction before construction of the UAB layer, which essentially transforms the traditional section (Terrell et al. 2003b).

Comparing the performances of the two inverted pavement test sections with a conventional flexible pavement section subjected to the same loading, Lewis et al. (2012) made the following observations:

- The two test sections performed remarkably well for more than 10 years, without needing any maintenance or resurfacing;
- No significant rut accumulation was observed in the inverted pavement test sections, whereas the conventional pavement section exhibited both "minor" and "major" rutting problems;
- FWD testing conducted in 2007 indicated that the two inverted pavement sections had remaining service lives of 99.34% (conventional compaction) and 94.61% (compacted using the South African slushing method), respectively, whereas the conventional pavement section had a remaining service life of 67.92%.

FHWA International Scanning Tour

A scanning study of France, South Africa, and Australia sponsored by FHWA, AASHTO, and NCHRP investigated innovative programs for pavement preservation (Beatty et al. 2002). During the scanning tour, the team observed typical pavement structures used by the countries visited to ensure longer-lasting, better-performing pavement systems. Figures 32 and 33 show the typical pavement structures constructed by Australia and South Africa, respectively, as noted by Beatty et al. (2002). As can be seen from the figures, it is common practice in Australia and South Africa to use thick aggregate layers in conjunction with relatively thin HMA surface layers. The practice in Australia involves the use of multiple

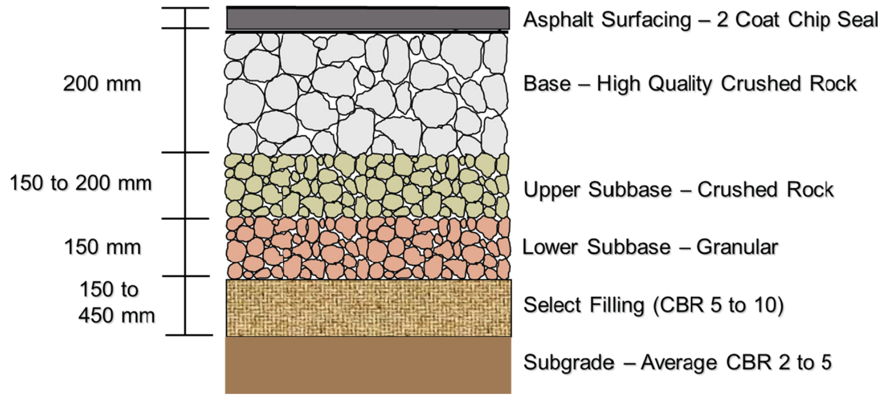


FIGURE 32 Typical heavy-duty pavement configuration in Australia (Beatty et al. 2002).

unbound aggregate layers in conjunction with a thin HMA surface layer, whereas the South African practice involves the construction of inverted pavement sections.

Application of Stone Interlayer Pavements in Louisiana

Stone interlayer pavement designs were introduced in Louisiana to reduce the problem of reflective cracking that is often observed in flexible pavements constructed using soil-cement bases (Rasoulian et al. 2000, 2001). Titi et al. (2003) compared the performances of stone interlayer pavements (3.5-in. HMA surface layer; 4-in. crushed limestone interlayer; 6-in. in-place cement-stabilized base course layer; and 12-in. lime-treated subgrade layer) with conventional flexible pavements with cement-treated bases (3.5-in. HMA surface layer, 8.5-in. in-place cement stabilized base course layer, and 12-in. lime-treated subgrade layer) constructed on State Highway LA-97 near Jennings, Louisiana. Both pavements were monitored for more than 10 years and were evaluated through pavement distress surveys, testing for roughness and permanent deformation, as well as evaluation of pavement structural capacity through dynamic nondestructive testing (NDT). The same two designs were also compared through accelerated pavement testing at the Louisiana Transportation Research Center. Through analyses of the field monitoring and accelerated testing data, Titi et al. (2003) reported that the stone interlayer pavements performed significantly better than did the conventional pavement designs with cement-treated base. From comparing the performances of the two pavement types, Titi et al. (2003) made the following primary observations:

- Both pavement types showed an increasing trend in crack accumulation with time. However, the rate of crack accumulation was significantly lower for the stone interlayer pavement sections.
- The average International Roughness Index value for the stone interlayer pavement was lower than that for the conventional flexible pavement after 10.2 years. This indicated smooth surface conditions and better ride quality for the stone interlayer pavement. This was attributed to the lower amount of reflective cracking in the stone interlayer pavement.
- The stone interlayer pavement could withstand about four times the number of load applications (1,294,800 ESALs) under accelerated testing compared with the conventional flexible pavement section (314,500 ESALs) before undergoing failure.
- From survival analyses of the two accelerated pavement sections, Metcalf et al. (1998) concluded that the dominant mode of failure (88%) for the stone interlayer pavement was rutting, whereas that for the conventional pavement was cracking.
- Through regression analyses of the long-term performance data of the two test sections along LA-97, it was concluded that the only mode that could lead to the failure of both of the modes was cracking. The regression analyses clearly established the superior performance of stone interlayer pavement sections.

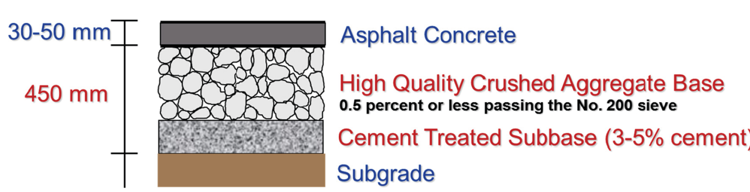


FIGURE 33 Typical pavement sections in South Africa constructed with high-quality crushed aggregate base layers (Beatty et al. 2002).

- The initial material cost for the stone interlay pavement was approximately 20% higher than that for the conventional pavement. However, considering the significantly higher number (300% higher) of load applications until failure, the stone interlayer pavement alternative indicated considerable savings when life-cycle costs were analyzed.

LaGrange Bypass Project, Troup County, Georgia

Encouraged by the positive results from the inverted pavement test sections in Morgan County, Georgia, in 2009 GDOT constructed another inverted pavement test section on the South LaGrange Loop in Troup County. The constructed inverted pavement test sections had (1) 150-mm (6-in.) thick stabilized subgrade; (2) 250-mm (10-in.) thick cement-treated base; (3) 150-mm (6-in.) thick GAB; (4) 50-mm (2-in.) thick Superpave binder course; and (5) 37-mm (1.5-in.) thick Superpave surface course (Lewis et al. 2012). The GAB was constructed using standard construction techniques at a moisture content of 100% to 120% of the OMC. Figure 34 shows a schematic of the inverted pavement sections constructed as part of this project.

Buchanan (2010) compared the life-cycle costs for the LaGrange Bypass inverted pavement sections with a rigid pavement designed to carry the same amount of traffic (the rigid pavement had a 9.5-in. thick PCC slab over a 10-in. thick GAB over a 6-in. thick prepared subgrade with a minimum soil support value of 5). Table 4 lists the comparative cost estimates over a 30-year life cycle as presented by Buchanan (2010). As can be seen from the table, the inverted pavement section results in net savings of $139,000 over a 30-year period.

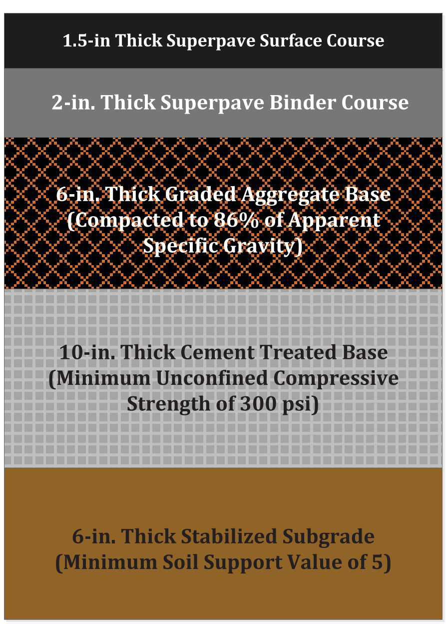


FIGURE 34 Schematic of inverted pavement section constructed in LaGrange Bypass Project, Troup County, Georgia.

TABLE 4
LIFE-CYCLE COST COMPARISON FOR LAGRANGE BYPASS INVERTED PAVEMENT SECTION WITH A RIGID PAVEMENT SECTION DESIGNED TO SUSTAIN THE SAME TRAFFIC LEVEL OVER A 30-YEAR PERIOD

Event	Cost ($/Lane-Mile)	
	Inverted Pavement	PCC Pavement
Installation cost	342,000	584,000
10 years of maintenance	101,000	
20 years of maintenance	123,000	
20–30 years of maintenance		121,000
30-year life-cycle cost	566,000	705,000
Net savings	139,000	

Avellandeda (2010) developed new field test methods to characterize the stress-dependent stiffness of UAB layers in these inverted pavement test sections and found that inverted pavement sections could exceed the structural capacities of flexible pavement designs and result in savings to 40% of the initial construction costs.

Lewis et al. (2012) reported that the test sections showed excellent structural capacities and long remaining lives upon FWD testing immediately after construction. Cortes (2010) conducted precompaction and postcompaction sieve analyses of aggregate samples collected from the GAB and reported inconclusive data about the extent of particle crushing. By digging trenches through the HMA layers to expose the GAB and subsequently processing the grain skeleton photographs through digital image analysis, Cortes found evidence of compaction-induced anisotropy in the GAB as the coarse aggregate particles were found to preferentially align their major axis parallel to the horizontal plane. Through FE analyses of the test sections, Cortes observed that both vertical and radial stresses in the UAB layer remained in compression throughout the layer depth.

Luck Stone Bull Run Project, Virginia

An application in 2010 of inverted pavement in Virginia involved a relocated road (Virginia Highway 659) bypassing the Luck Stone Bull Run Quarry. The project included collaboration between Luck Stone, Texas A&M University (ICAR), FHWA Office of Infrastructure R&D, Virginia DOT, and the Virginia Transportation Research Council. This section was designed using the ICAR model, and the materials characterization protocol was carried out at Texas A&M University and the Texas Transportation Institute and instrumented heavily through FHWA sponsorship. Discussing the benefits of this inverted pavement trial application, Weingart (2012) reported a potential for 22.3% cost savings compared with the construction of a conventional flexible pavement with equivalent structural and functional capacities; the estimated cost for construction of the conventional flexible pavement section was $21,311 per 100 linear ft, whereas that for the inverted pavement section was $16,555 per 100 linear ft.

Summary of Past Experience on Inverted Pavements

From extensive review of literature covering inverted pavement applications internationally as well as within the United States, it was observed that almost all applications of inverted pavements have resulted in favorable performance compared with conventional pavement structures. In addition to resulting in superior performance, inverted pavement sections often have led to significant cost savings over the life cycle of the pavement. Although the construction of pavements using thick unbound aggregate layers appears to be a common practice in countries such as Australia and South Africa, projects involving such pavements have been confined primarily to trial studies in the United States. Moreover, these trial projects have been confined to a limited number of states, with most other states showing resistance to the adoption of such innovative pavement construction practices. Possible explanations of why inverted pavements have not been constructed in the United States include (1) traditional pavement designs and construction practices using rather thick asphalt or concrete surface courses were still affordable; (2) details of foreign technology related to UAB compaction, such as the South African slushing technique, were not readily available; and (3) cement-treated subbase used in inverted pavements was considered a potential risk for pavement cracking, especially in northern climates. A conscious effort needs to be made to encourage the construction of inverted pavements in the United States to fully study any potential disadvantages, such as pavement distresses occurring as a result of cracking of the cement-treated subbase, perhaps as a result of shrinkage and exposure to freeze-thaw conditions. In addition, in colder climates further evaluation of related pavement design considerations is needed. According to the Portland Cement Association, it is possible to limit the percent cement used in inverted subbases (such as to 2% to 3%) to potentially mitigate these cracking problems. The successful construction, ongoing documentation, and technology transfer of the superior performances of the inverted pavement trials no doubt will have a positive impact on such sustainable alternatives to pavement design.

Current State of Practice on Alternative Base Course Construction

One of the objectives of the current synthesis study was to gather information on the state of practice in the United States and Canada regarding the application of alternative UAB/subbase layers, such as inverted pavement sections. Accordingly, the survey of state and Canadian provincial transportation agencies included questions regarding construction practices such as the South African slushing technique. Only two states (New Mexico and Rhode Island) reported the use of alternative construction techniques. New Mexico DOT reported an ongoing project involving the construction of inverted pavement sections that will use the South African slushing technique for compaction of the UAB layer. Rhode Island DOT indicated that the agency used the "test strip" method to determine the maximum achievable density value for an UAB/subbase layer through repeated compaction of the same spot until no noticeable increase in the density was achieved. The DOC achieved during construction of UAB/subbase layers was then compared with the maximum density values obtained from the test strips. No other state reported the use of innovative construction practices.

Key Lessons

- Inverted pavements involve the construction of a "high quality" crushed stone base layer over a stabilized subbase course.
- With the aggregate base layer functioning as the primary structural component, inverted pavements offer a long-lasting, economical alternative to conventional pavement construction.
- Construction of inverted pavements and similar pavements utilizing thick crushed aggregate base layers is a common practice in countries such as South Africa, Australia, and France.
- All inverted pavement applications in the United States have resulted in equal or better performance compared with equivalent conventional pavement sections.
- A conscious effort is required in the United States and Canada to encourage the construction of alternative pavement structures with thick UAB layers as the primary structural component.

SUMMARY

This chapter presents an overview of current practices as far as material handling and construction practices for UAB and subbase layers are concerned. Extensive review of published literature was conducted to gather information on different procedures and practices identified by researchers to be adequate/inadequate for pavement layer construction. Aggregate segregation and degradation are identified as two major concerns affecting aggregate gradation, and different practices that magnify these problems are listed. A survey of state and Canadian provincial transportation agencies indicated that only 37% of the responding agencies (46 total respondents) currently have specific guidelines governing aggregate storage, transportation, and stockpiling practices.

The current state of the practice regarding construction lift thicknesses indicates a significant gap between the knowledge gained through research and trial projects and current agency specifications. Different research studies establishing the effectiveness of greater lift thicknesses during construction are summarized in this chapter and the need for harmonizing such practices among transportation agencies was established.

Finally, this chapter discusses the concept of inverted pavements as an alternative application of UAB layers. The concept behind this application was described, as were the response mechanism and construction procedures. Different research studies emphasizing the effectiveness of inverted pavements are highlighted, and the need for further exploration in this area was established. The next chapter discusses the different methods used for the characterization of unbound aggregate materials and layer design.

REFERENCES

Ahlvin, R.G., W.J. Turnbull, J.P. Sale and A.A. Maxwell, *Multiple-Wheel Heavy Gear Load Pavement Tests,* U.S. Army Corps of Engineers, Vicksburg, Miss., 1971, 216 pp.

Allen, J.J., J.L. Bueno, M.E. Kalinski, M.L. Myers, and K.H. Stokoe II, *Increased Single-Lift Thicknesses for Unbound Aggregate Base Courses,* ICAR Report No. 501-5, International Center for Aggregate Research, The University of Texas at Austin, 1998.

Aughenbaugh, N.B., R.B. Johnson, and E.J. Yoder, *Degradation of Base Course Aggregates during Compaction,* School of Civil Engineering, Purdue University, West Lafayette, Ind., 1963.

Avellandeda, D.D.C., *Inverted Base Pavement Structures,* Ph.D. Thesis, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, 2010.

Barker, W.R., W.N. Brabston and F.C. Townsend, *An Investigation of the Structural Properties of Stabilized Layers in Flexible Pavement Systems,* U.S. Army Corps of Engineers, 165, Vicksburg, Miss., 1973.

Barksdale, R.D. and H.A. Todres, *A Study of Factors Affecting Crushed Stone Base Performance,* School of Civil Engineering, Georgia Institute of Technology, Atlanta, 1983.

Barksdale, R.D., "Performance of Crushed Stone Base Courses," *Transportation Research Record 954,* Transportation Research Board, National Research Council, Washington, D.C., 1984, pp. 78–87.

Barksdale, R.D., *The Aggregate Handbook,* National Stone Association, Washington, D.C., 1991.

Beatty, T.L., et al., *Pavement Preservation Technology in France, South Africa, and Australia,* Office of International Programs, Federal Highway Administration, U.S. Department of Transportation, and the American Association of State Highway and Transportation Officials, Alexandria, Va., 2002.

Buchanan, S., Inverted Pavements-What, Why, and How? AFTRE Industry Education Webinar, Aggregates Foundation for Technology, Research, and Education, Alexandria, Va., June 1, 2010.

Bueno, J.L., K.H. Stokoe, and J.J. Allen, "A Study on the Feasibility of Compacting Unbound Graded Aggregate Base Courses in Thicker Lifts that Presently Allowed by State Department of Transportation," Report No. ICAR 501-2, International Center for Aggregates Research, University of Texas at Austin, 1998.

Bueno, J.L., K.H. Stokoe II, J.J. Allen, and M.E. Kalinski, "Evaluation of Constructing Increased Single-Lift Thicknesses of Unbound Aggregate Bases Case Study in Georgia," *Transportation Research Record: Journal of the Transportation Research Board, No. 1673,* 1999, pp. 95–102.

Cortes, D.D., "Inverted Base Pavement Structures," Ph.D. Dissertation, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, 2010.

De Beer, M., "South African G1 Base Course-Inverted Pavement," 2012 Transportation Research Board Mineral Aggregates Committee (AFP70) Meeting Presentation, 91st Annual Meeting of the Transportation Research Board, Jan. 22–26, 2012, Washington, D.C.

Grau, R.W., "Evaluation of Structural Layers of Flexible Pavements," Miscellaneous Paper, S-73-26, Waterways Experiment Station, 1973.

Horne, D., et al., *South African Pavement and Other Highway Technologies and Practices,* Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 1997.

Johnson, V.W., "Comparative Studies of Combinations of Treated and Untreated Bases and Subbase for Flexible Pavements," Bulletin 289, Highway Research Board, National Research Council, Washington, D.C., 1960, pp. 44–61.

Jooste, F.J., and L. Sampson, "The Economic Benefits of HVS Development Work on G1 Base Pavements," Department of Public Transport, Roads, and Works, 2005.

Lewis, D.E., K. Ledford, T. Georges, and D.M. Jared, "Construction and Performance of Inverted Pavements in Georgia," Paper No. 12-1872, Poster Presentation in Session 639, 91st Annual Meeting of the Transportation Research Board, Jan. 22–26, 2012, Washington, D.C.

Majidzadeh, K. and C.S. Brahma, *Statistical Analysis of Aggregate Size Distribution,* The Ohio State University Department of Civil Engineering and Transportation Research Center, 1969.

Maree, J.H., N.J.W. van Zyl, and C.R. Freeme, "Effective Moduli and Stress Dependence of Pavement Materials as Measured in Some Heavy-Vehicle Simulator Tests," *Transportation Research Record 852,* Transportation Research Board, National Research Council, Washington, D.C., 1982a.

Maree, J.H., C.R. Freeme, N.J.W. van Zyl, and P.F. Savage, "The Permanent Deformation of Pavements with Untreated Crushed-Stone Bases as Measured in Heavy Vehicle Simulator Tests," *Proceedings of the 11th Australian Road Research Board Conference,* Melbourne, 1982b.

Metcalf, J.M., S. Romanoschi, L. Yongqi, and M. Rasoulian, *Construction and Comparison of Louisiana's Conventional and Alternative Base Courses under Accelerated Loading, Interim Rep. 1, Phase 1,* Louisiana Transportation Research Center, Baton Rouge, 1998.

Miller Warden Associates, "Effects of Different Methods of Stockpile Sampling Interim Report," *Proceedings of Highway Research Board,* Highway Research Board, National Research Council, Washington, D.C., 1964.

National Stone, Sand and Gravel Association (NSSGA), *Stone Base Construction Handbook,* NSSGA, Alexandria, Va., 1989.

Nohl, J., and B. Domnick, "Stockpile Segregation," *Technical Paper T-551,* Superior Industries, Morris, Minn., 2000.

O'Neil, D.J., J.P. Mahoney, and N.C. Jackson, *An Evaluation of Granular Overlays in Washington State,* Final Technical Report, Report No. FHWA-SA-92-042, Washington State Department of Transportation, Olympia, 1992.

Rasoulian, M., B. Becnel, and G. Keel, "Stone Interlayer Pavement Design," *Transportation Research Record 1709,* Transportation Research Board, National Research Council, Washington, D.C., 2000, pp. 60–68.

Rasoulian, M., H. Titi, M. Martinez, B. Becnel, and G. Keel, *Long Term Performance of Stone Interlayer Pavement,* Louisiana Transportation Research Center Report 29, Baton Rouge, 2001.

Saunders, C.H., "Increasing the Single Lift Thickness for Aggregate Base: Base Placement and Compaction of Increased Lift Thickness of Aggregate Base on Test Projects," Paper Presented at the 5th Annual Symposium of the International Center for Aggregate Research (ICAR), 1997.

Terrell, R.G., B.R. Cox, F.Y.U.H. Menq, J.J. Allen, and K.H. Stokoe, II, "Stiffness of Unbound Aggregate Base Layers in Inverted Flexible Pavements," International Center for Aggregates Research 11th Annual Symposium: Aggregates–Asphalt Concrete, Bases and Fines, Austin, Tex, Apr. 27–30, 2003a.

Terrell, R.G., B.R. Cox, K.H. Stokoe II, J.J. Allen, and D. Lewis, "Field Evaluation of the Stiffness of Unbound Aggregate Base Layers in Inverted Flexible Pavements," *Transportation Research Record: Journal of the Transportation Research Board, No. 1837,* Transportation Research Board of the National Academies, Washington, D.C., 2003b, pp. 50–60.

Titi, H., M. Rasoulian, M. Martinez, B. Becnel, and G. Keel, "Long-Term Performance of Stone Interlayer Pavement," *Journal of Transportation Engineering,* Vol. 129, No. 2, 2003, pp. 118–126.

TRH, *Guidelines for Road Construction Materials, Draft Technical Recommendation for Highways (TRH) 14*, Pretoria, South Africa, 1985.

Tutumluer, E. and R.D. Barksdale, "Inverted Pavement Response and Performance," *Transportation Research Record 1482,* Transportation Research Board, National Research Council, Washington, D.C., 1995, pp. 102–110.

Weingart, R., "Inverted Base: The Virginia Experience," 2012 Transportation Research Board Mineral Aggregates Committee (AFP 70) Meeting Presentation, 91st Annual Meeting Transportation Research Board, Jan. 22–26, 2012, Washington, D.C.

Wells, G.E., and R.C. Adams, "Increased Single-Lift Depths for Aggregate Base Course in Highway Construction: Secondary Road 1117 Relocation, Richmond County, North Carolina," presented at the 5th Annual Symposium of the International Center for Aggregate Research (ICAR), 1997.

White, D.J., P. Vennapusa, and C.T. Jahren, *Determination of the Optimum Base Characteristics for Pavements,* Iowa DOT Project TR-482, Center for Transportation Research and Education, Iowa State University, Ames, 2004.

Williamson, T.G., and E.J. Yoder, *An Investigation of Compaction Variability for Selected Highway Projects in Indiana,* Publication FHWA/IN/JHRP-67/05, Joint Highway Research Project, Indiana Department of Transportation and Purdue University, West Lafayette, 1967.

Womack, W.W., "Increasing the Lift Thickness for Aggregate Base: Virginia Department of Transportation Perspective," presented at the 5th Annual Symposium of the International Center for Aggregate Research (ICAR), 1997.

CHAPTER FOUR

UNBOUND AGGREGATE BASE CHARACTERIZATION FOR DESIGN

INTRODUCTION

This chapter presents an overview of unbound aggregate material characteristics and structural layer behavior as the primary structural component in flexible pavement systems, as well as subbase layers under concrete pavement slabs. A thorough review of different aggregate test procedures and characterization methods commonly used to model granular pavement layer responses and permanent deformation behavior is imperative to facilitate better designs of pavement systems and ultimately ensure adequate performance under repeated loading.

The main load transfer mechanism governing unbound aggregate structural layer behavior under loading is highlighted. Important aggregate physical properties affecting granular layer strength, modulus, and permanent deformation behavior are discussed in detail in this chapter. Commonly used models to characterize the elastic or resilient, as well as permanent, deformation behavior of unbound aggregate materials are discussed with a review of typical pavement stress states and initial loading conditions affecting the primary features of aggregate repeated-load behavior. Both empirical and M-E methods developed for designing unbound aggregate pavement systems are summarized with historical perspective by listing their advantages and limitations. State-of-the-art approaches, including the stress-dependent and anisotropic models from the recent ICAR research findings, are described in detail for proper characterization of unbound aggregate pavement layers. Accordingly, different mechanisms contributing to the failure of pavement systems with unbound aggregate layers are reviewed to emphasize the importance of aggregate material quality governing pavement performance. Finally, permeable and open-graded aggregate layers are discussed to review and summarize the most important climatic (that is, moisture and temperature) conditions influencing designs of unbound aggregate pavement systems.

LOAD TRANSFER IN GRANULAR MATERIALS

The mechanism of load transfer in granular materials was first experimentally studied by Dantu (1957) with the help of photoelastic models. From the experiments performed, it was concluded that the stresses in granular materials were not uniformly distributed but were concentrated along load-carrying particle chains. Later Oda (1974) described other experiments in which photoelastic rods were loaded biaxially. Forces across individual particle contacts were monitored by counting the resulting interference fringes.

Based on experimental studies, the stresses in particulate media are not transferred in a uniform manner but are concentrated along continuous columns of particles. The particles in between the columns provide only lateral support but do not carry much load. At a critical load, a column will fail and the internal structure will be rearranged. Formation of a new column takes place if particles in that region are favorably oriented. The deformation of a particulate mass under increasing load is then mostly the continual collapse and generation of adjacent chains of load-carrying particles. The predominant orientation of particle contacts is in the direction of the major principal stress.

Similar results on the load transfer and deformation characteristics of granular materials were obtained by Dobry et al. (1989). Using the discrete element approach (Cundall and Strack 1979), Dobry et al. (1989) modeled granular soil as random arrays of 531 elastic, rough spheres of two different sizes. Numerical simulations of these arrays under monotonic and cyclic loading were compared with typical experimental results from triaxial compression tests on a medium-dense uniform quartz sand. From the numerical simulations, they observed that triaxial deviator stresses were clearly transmitted by a limited number of "stiff chains" or irregular columns of grains aligned in generally the vertical direction.

Therefore, according to the experimental and numerical findings, the deformation pattern of an unbound aggregate layer is directly related to load transfer by shear in the columns of particles supported under confinement. The orientation of such columns is primarily in the direction of the principal stresses and also is affected by the assembly of the grains and their shape.

UNBOUND GRANULAR MATERIAL BEHAVIOR UNDER REPEATED LOADING

Unbound aggregate layers in pavements are subjected to repeated load applications as a result of traffic. They undergo both elastic (commonly known as resilient for pavement applications) as well as plastic (permanent) deformations with every load repetition. Figure 35 presents a schematic of typical unbound aggregate behavior under repeated loading with the help of a stress-strain diagram. Note that the relative

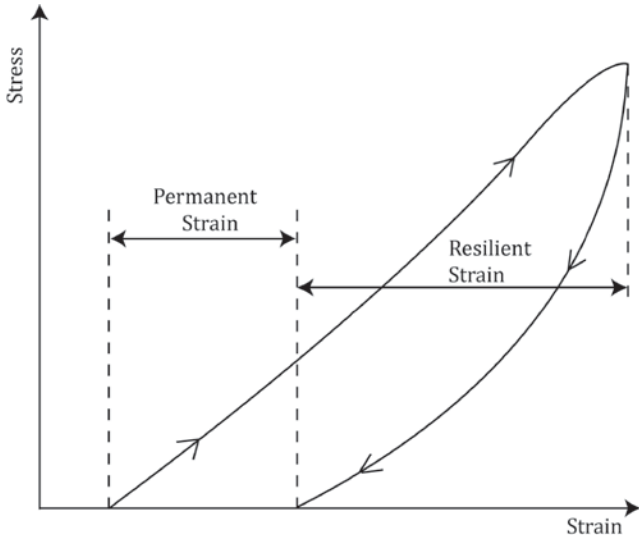


FIGURE 35 Strains in granular materials during one cycle of load application.

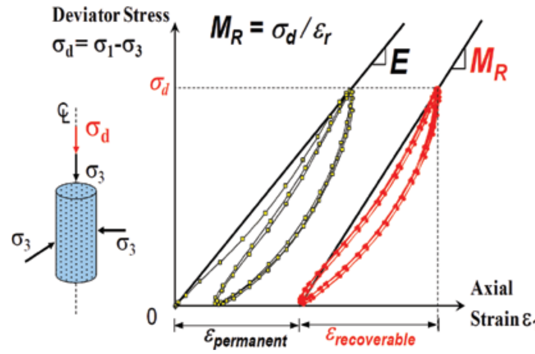


FIGURE 36 Resilient modulus defined as the elastic modulus of a deformed material.

magnitudes of elastic and plastic components of the total strain depend on several different factors, including traffic load levels and speed of operation, thickness and quality of overlying pavement layers (if any), characteristics of aggregates used in construction of the aggregate layer, and subgrade conditions.

In a constructed granular layer, the accumulation of permanent deformation as a result of each load repetition gradually decreases with increased number of load applications. Once the layer has been well compacted to achieve a densely packed matrix, all the subsequent load applications ideally would result in deformations that are mostly elastic in nature. The resilient and permanent deformations of an unbound granular layer can be attributed to different mechanisms. Werkmeister (2003) summarized the Hertz contact theory and suggested that resilient deformation in granular materials is caused primarily by "temporary" deformation of individual grains, whereas permanent deformation takes place because of relative movement of the particles with respect to each other. The initial rapid accumulation of permanent deformation typically corresponds to the rearrangement of particles during initial compaction and subsequent loading of the pavement layers. After adequate "shakedown" (that is, particle reorientation and rearrangement into a dense matrix) of the material is reached under this initial loading phase, the pavement layers show predominantly resilient deformation provided that the load levels remain below permissible limits.

RESILIENT RESPONSE OF UNBOUND AGGREGATE LAYERS

Ideally, pavement layer response under traffic loading should be purely elastic, and thus no accumulation of permanent deformation would occur during its service life. Accordingly, mechanistic-based pavement design approaches traditionally have focused on the elastic or resilient response of unbound aggregate layers to predict the critical pavement responses under traffic loading. Figure 36 shows a schematic of typical hysteretic response exhibited by unbound aggregate materials under repeated loading. The most important input property for characterizing repeated load behavior of unbound aggregate layer in pavement analysis has been the "resilient modulus." Defined as a secant modulus representing hysteretic stress-strain behavior of materials, the resilient modulus (M_R) is a critical material property needed for M-E pavement design methods (Puppala 2008).

As shown in Figure 36, the resilient modulus (M_R) of a material is defined as the elastic modulus after the material has accumulated a certain amount of permanent deformation. The difference between elastic or Young's modulus (E) and the resilient modulus (M_R) of a material is clearly highlighted in the figure. Equation 2 can be used to determine the resilient modulus of a material from repeated-load triaxial test results. Note that in the equation, σ_d represents the deviator stress or repeated wheel load stress, and ε_r represents the recoverable strain.

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{2}$$

Key Lessons

- Particle-to-particle interlock is critical to the dissipation of stresses in unbound aggregate layers under loading.
- Constructed unbound aggregate layers are best compacted into a densely packed matrix to ensure all deformations under vehicle loading are primarily resilient in nature and no significant permanent deformation accumulation occurs.

STRESS STATES IN UNBOUND AGGREGATE LAYERS UNDER LOADING

Pavement stresses are mainly composed of two parts: initial in situ stresses and stresses resulting from moving wheel loads. The initial in situ stresses, static in nature, are the overburden and residual stresses. The initial stresses typically are lower

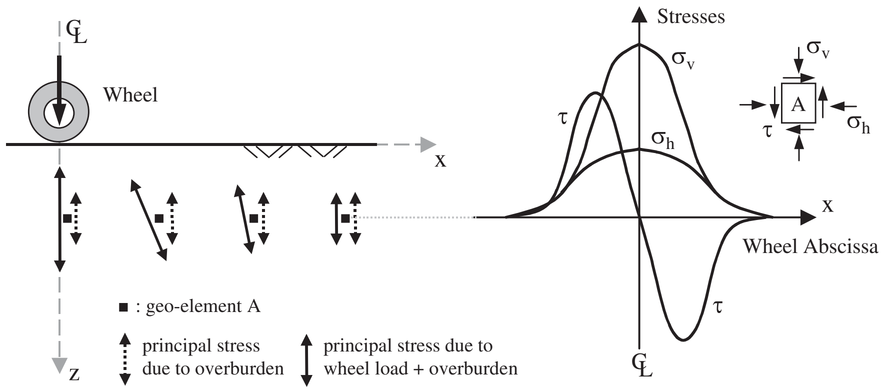
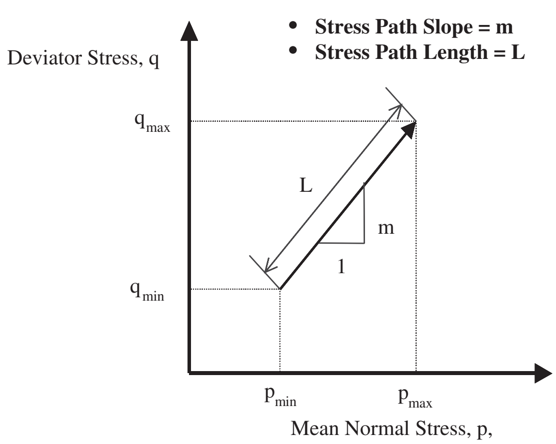


FIGURE 37 Stress states and rotation of principal stresses experienced by the aggregate layer beneath a rolling wheel load.

at shallow depths than at greater depths. Compaction-induced residual stresses that are compressive in nature can often exist in the unbound aggregate layers and contribute to the static stress states (Uzan 1985; Barksdale et al. 1997). On the other hand, traffic loading resulting from moving wheel loads induces much higher dynamic stresses than do the static ones. For example, the dynamic vertical stresses become the highest underneath the wheel where shear loading is nonexistent on a representative pavement element, but at some radial distance away from the wheel, applied vertical stresses decrease and the shear stresses reach their maximum values. In summary, a pavement element constantly experiences a combination of varying magnitudes of static and dynamic vertical (compressive) and shear stresses, depending on the depth in the pavement layer and the radial offset from the wheel load.

A known limitation of repeated-load triaxial tests is that the principal stress rotation and the constantly rotating fields of stresses under moving wheel loads are not possible to simulate in a continuous fashion. However, principal stress rotation may cause increased rates of shear and volumetric strains during cyclic loading relative to equivalent stress paths without stress rotation. In the case of aggregate bases, the cyclic component of load imposes a change (increment) of stress state, which is not co-axial with the stress state under the static (overburden) load. This is illustrated in Figure 37. The major principal stress caused by overburden is always aligned in the vertical direction, regardless of the location of a moving wheel. However, the incremental stresses imposed by a wheel load are not co-axial with this system, and as a result, the total principal stresses rotate as the wheel load passes.

Figure 38 illustrates the concept of stress path loading related to stress path slope (*m*) and stress path length (*L*) on a *q-p* diagram (Kim 2005). Static overburden stresses correspond to q_{min} and p_{min}, whereas dynamic traffic load reaches to q_{max} and p_{max} following a constant stress path slope (*m*). Analyses of test data often require defining geomaterial behavior in terms of these principal stresses considering a mean normal stress component (*p*) influencing volume change and the deviator stress component (*q*) affecting shear behavior for shape change and distortion (Kim and Tutumluer 2005). In general, the stress path slope ($m = \Delta q/\Delta p$) for the standard constant confining pressure (CCP) tests (characterized by the application of an all-around constant confining pressure while the vertical deviator stress is pulsed) takes a constant value of 3.0. For variable confining pressure (VCP) tests characterized by pulsing of the confining pressure in phase with the axial deviator stress, the stress slope varies generally from −1.5 to 3. VCP tests offer the capability to apply a wide combination of stress



$$p = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3}$$

$$q = \sigma_1 - \sigma_3$$

FIGURE 38 Concept of stress path loading showing slope and length (Kim 2005).

paths by pulsing both cell pressure, σ_3, and vertical deviator stress, σ_d. Various stress paths cause different loading effects on pavement elements, which are not yet fully studied and understood to explain permanent deformation accumulation.

Key Lessons

- The directions of principal stresses imposed on an unbound aggregate pavement layer undergo constant rotation under a moving wheel load.
- Such rotation of the principal stress directions as well as the associated loading patterns are best simulated during laboratory testing of aggregates to obtain realistic estimates of unbound aggregate layer performance under loading.

COMPACTION-INDUCED RESIDUAL STRESSES

In the initial stages of new pavement construction, heavy loads are applied to granular layers causing large deformations by compaction equipment. These layers are subjected to larger stresses during construction than they may ever experience during the service life of the pavement structure. The largest vertical and lateral stresses are caused in the uppermost lift as compaction progresses. After the compaction is completed, field measurements indicate compressive residual lateral stresses are locked in the granular bases (Barksdale and Alba 1993). These residual stresses developed as a result of compaction of unbound aggregates should be considered in determining the initial stress state of granular bases.

Proper compaction of granular pavement layers is required to ensure adequate strength and stability of the layer. The particles, when subjected to compaction, rearrange themselves by translating and rotating to become locked in a final position. After the externally applied compaction stress is removed, this final stage is not a stress-free state, but rather a residual stress state. The residual stress state includes the effects of both confinement and aggregate interlock. Depending on the pore size distribution in the aggregate matrix, as well as the compaction moisture content, suction-induced negative pore pressures may exist in the newly constructed aggregate layer.

The initial stress states used in the analysis of pavements usually is determined only by geostatic stresses attributable to body weight and are ignored in most linear elastic pavement analyses. A comprehensive granular base model needs to include both overburden stresses and the horizontal residual stresses. Several researchers have experimentally analyzed the residual stresses produced in granular bases (Stewart et al. 1985; Uzan 1985; Selig 1987; Zeilmaker and Henny 1989; Barksdale and Alba 1993). According to the research performed by Uzan (1985), Stewart et al. (1985), and Zeilmaker and Henny (1989), these horizontal residual stresses were measured to be as high as 2 to 5 psi in cohesionless granular materials. Barksdale and Alba (1993) also reported 3 psi horizontal residual stresses in the upper 6-in. (152-mm) portion of a 12-in. (305-mm) thick granular base obtained from field measurements.

Based on experiments, Broms (1971), Ingold (1979) and Uzan (1985) employed a limit equilibrium approach to predict compaction-induced lateral stresses. The vertical stress under the compaction equipment was determined assuming a line loading (Holl 1941) and a semi-infinite homogeneous elastic halfspace (Boussinesq 1885). The lateral stresses developed were limited to the maximum compaction loading and unloading conditions applied to a pavement in accordance with the classic earth pressure theory for frictional materials:

1. Under the loading of compaction equipment, horizontal stresses start to increase according to the active state when the limit equilibrium is reached and horizontal compression develops in the granular layer:

$$\sigma_h = K_a \sigma_v$$

where σ_v and σ_h are the vertical and horizontal stresses, and K_a is the coefficient of active lateral earth pressure, which usually is expressed in terms of the friction angle || as:

$$K_a = \tan^2\left(45 - \phi/2\right)$$

2. After the compaction is completed, during unloading, the vertical stresses decrease. When the limit equilibrium is reached, horizontal stresses also decrease according to the passive state, and vertical stresses finally reduce down to the overburden stresses:

$$\sigma_h = K_p \sigma_v$$

where K_p is the coefficient of passive lateral earth pressure, which is usually expressed in terms of the friction angle || as:

$$K_p = \tan^2\left(45 + \phi/2\right).$$

Using the method of analysis, Uzan (1985) observed that a maximum vertical stress of 61 psi reached during compaction yielded a horizontal residual stress of about 6 psi. This residual stress may be higher depending on the friction angle (||) and load intensity (Tutumluer and Thompson 1998). The importance of considering compaction-induced residual stresses in the analysis and design of unbound aggregate layers is discussed later in this chapter.

CONCEPT OF CROSS-ANISOTROPY

The behavior of a granular medium at any point depends on the arrangement of particles, which usually is determined by aggregate characteristics, construction methods, and loading conditions. In the case of unbound aggregate pavement layers, an apparent anisotropy is induced during construction by aggregate placement and then loading from the compaction equipment. Thus, the granular layer becomes stiffer in the vertical direction than in the horizontal direction, even before the wheel load on the pavement imposes further anisotropic loading.

Most geomaterials, such as naturally deposited soils, exhibit a rotational symmetry about their vertical axes called the "axis of symmetry." The material properties are then the same in all directions on the plane perpendicular to the axis of symmetry. These materials are known as "cross-anisotropic" materials. An isotropic material has the same material properties in all directions. A cross-anisotropic material has different properties in the horizontal and vertical directions. The stress-strain conditions in such a material can be defined using the following five material properties (as illustrated in Figure 39): (1) stiffness in the vertical direction M_R^z, (2) stiffness in the radial (horizontal) direction M_R^r, (3) shear modulus in the vertical direction G_R^z, (4) Poisson's ratio for strain in the horizontal direction as a result of a vertical direct stress ν_z, and (5) Poisson's ratio for strain in any horizontal direction as a result of a horizontal direct stress ν_r.

Equation 3 shows the constitutive relationship for an elastic cross-anisotropic material in terms of the five independent material parameters.

$$\begin{Bmatrix} \varepsilon_v \\ \varepsilon_h \\ \varepsilon_h \\ \gamma_{vh} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_v} & -\frac{\nu_{vh}}{E_h} & -\frac{\nu_{vh}}{E_h} & 0 \\ -\frac{\nu_{hv}}{E_v} & \frac{1}{E_h} & -\frac{\nu_{hh}}{E_h} & 0 \\ -\frac{\nu_{hv}}{E_v} & -\frac{\nu_{hh}}{E_h} & \frac{1}{E_h} & 0 \\ 0 & 0 & 0 & \frac{1}{G_{vh}} \end{bmatrix} \begin{Bmatrix} \sigma_v \\ \sigma_h \\ \sigma_h \\ \tau_{vh} \end{Bmatrix} \quad (3)$$

where

E_h is the modulus of elasticity in the horizontal direction;
E_v is the modulus of elasticity in the vertical direction;
G_{vh} is the shear modulus;

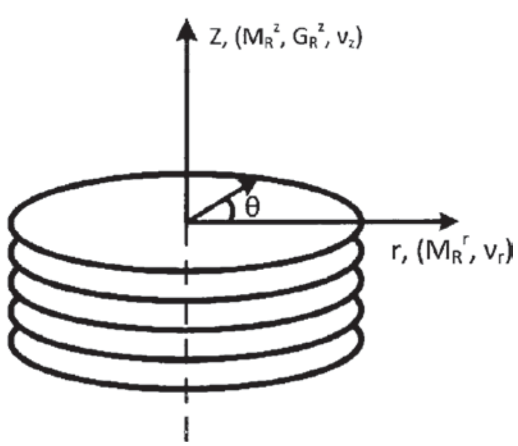


FIGURE 39 Stratified anisotropic material under axial symmetry.

ν_{vh} and ν_{hv} are the out-of-plane Poisson's ratio; and
ν_{hh} is the in-plane Poisson's ratio.

The remaining parameters are not independent, as was proven by Love (1944), and is shown in Equations 4 and 5.

$$\frac{\nu_{hv}}{E_v} = \frac{\nu_{vh}}{E_h} \quad (4)$$

$$G_{hh} = \frac{E_h}{2(1+\nu_{hh})} \quad (5)$$

Key Lessons

- Compaction of unbound aggregate layers results in preferential orientation of individual aggregate particles, which ultimately leads to "cross-anisotropic" behavior.
- Such compaction and stress-induced anisotropy is best considered during the design and analysis of pavement systems with UAB and subbase layers.

METHODS TO CHARACTERIZE UNBOUND AGGREGATE LAYER BEHAVIOR

The recent NCHRP Project 4-23, *NCHRP Report 453: Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers,* summarized the most important tests that relate to the performance of aggregates in unbound pavement layers (Saeed et al. 2001). Among the tests highlighted, the shear strength tests (triaxial tests conducted on wet and dry samples and CBR test) and stiffness test (resilient modulus conducted on wet and dry samples) are the most relevant for characterizing the strength, modulus, and permanent deformation behavior of unbound aggregate pavement layers.

California Bearing Ratio

The CBR test (AASHTO Test Method T-193; ASTM Test Method ASTM D 1883) is an empirical test method. In this test, the aggregate is compacted into a 6-in. diameter mold to form a specimen 4.6 in. high. The maximum particle size permitted is ¾ in. Specimen conditioning usually consists of a 96-hour soaking period to simulate wet pavement conditions. Soaking is particularly important if a significant quantity of fines (passing No. 200 sieve or less than 0.075 mm) material is present. The specimen is then penetrated at a loading rate of 0.05 in./minute with a piston having an end area of 3 square inches. The specimen remains in the mold throughout the testing process. The CBR is calculated by dividing the piston pressure at 0.1 or 0.2 in. penetration by standard reference values of 1,000 psi for 0.1 in. penetration and 1,500 psi for 0.2 in., multiplied by 100 to give the CBR value expressed as a percent. These standard values represent the pressures observed for a high-quality, well-graded, crushed stone reference material. Accordingly, high-quality, dense-graded crushed stone commonly has CBR values in excess of 80, whereas well-graded gravel (AASHTO classification A-1-a; Unified Classification GW) may have CBR values ranging from 30 to 80. Note that testing of angular crushed stones in the laboratory often results in CBR values significantly higher than 100.

Note that many base course aggregate specifications require CBR values in excess of 80 and subbase specifications require minimum CBR values in the range of 20% to 50%. CBR values often are presented together with moisture-density test results to indicate the change in CBR behavior above and below the OMC. Swell measurements if taken for the sample also precede the CBR testing. Figure 40 shows typical CBR and swell test results obtained from an unsoaked molded sample and a sample of the same material that was allowed to soak for 96 hours. Note that unsoaked specimens are likely to give high CBR values, often higher than 100, on the dry side of OMC.

CBR is not a fundamental material property and thus is unsuitable for direct use in mechanistic and M-E design procedures. However, it is a relatively easy and inexpensive test to perform, it has a long history in pavement design, and it is reasonably well correlated with more fundamental properties such as resilient modulus. Consequently, it continues to be used in practice.

Most CBR testing is laboratory-based; thus, the results will be highly dependent on the representativeness of the samples tested. It is also important that the testing conditions be clearly stated: for example, CBR values measured from as-compacted samples at optimum moisture and density conditions can be significantly greater than CBR values measured from similar samples after soaking. For field measurement, care is to be taken to make certain that the deflection dial is anchored well outside the loaded area. Field measurement is made at the field moisture content, whereas laboratory testing typically is performed for soaked conditions, so soil-specific correlations between field and laboratory CBR values are often required.

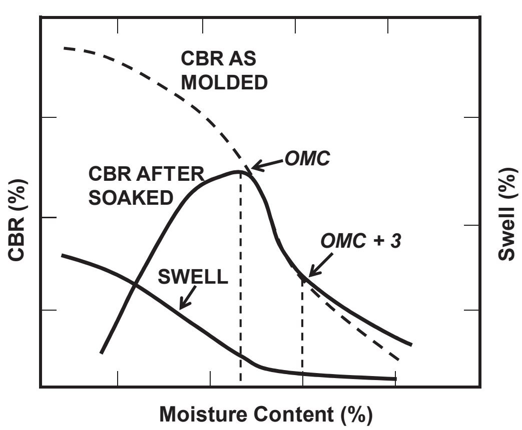


FIGURE 40 Presentation of CBR and swell measurements in relation to specimen moisture content.

TABLE 5
TYPICAL FIELD CALIFORNIA BEARING RATIO (CBR) VALUES FOR DIFFERENT SOIL CLASSES

Unified Soil Classification System (USCS) Soil Class	Field CBR (%)
GW	60–80
GP	35–60
GM	40–80
GC	20–40
SW	20–40
SP	15–25
SM	20–40
SC	10–20
ML	5–15
CL	5–15
OL	4–8
MH	4–8
CH	3–5
OH	3–5

Source: Christopher et al. (2010).

Table 5 lists typical field CBR values for different Unified Soil Classification System classifications as obtained from Christopher et al. (2010) with reference to original work by the U.S. Army Corps of Engineers (1953).

Static Triaxial Testing

Strength is defined as the maximum level of stress that material can sustain before it fails or excessively deforms. Strength properties of a granular material can be best determined from static triaxial testing with monotonically increasing loading. A cylindrical test specimen is prepared at a target density and moisture content and is then encased in a membrane. The specimen is subjected to a constant all-around confining pressure (σ_3) and then loaded under an increasing axial stress until failure. Because the axial stress is in addition to the confining stress already on the specimen, it is called the deviator stress: $\sigma_d = \sigma_1 - \sigma_3$. The total axial stress is called σ_1. Usually three triaxial tests are conducted over a range of confining pressure levels representative of probable in-service conditions. Confining pressures used typically vary from 3 to 40 psi. Axial strain rates used in triaxial testing are typically 1% to 2% strain per minute. Triaxial test data are then interpreted to determine the cohesion (c) and angle of internal friction (ϕ) of the material tested. The parameters c and ϕ define the shear strength of the material, which is given by the Mohr-Coulomb equation:

$$\tau_{max} = c + \sigma_n \tan\phi \qquad (6)$$

where

τ_{max} = Shear strength
c = Cohesion
σ_n = Normal stress on specimen failure plane
ϕ = Angle of internal friction.

Considering vehicles usually move across a pavement very quickly, triaxial shear tests at the University of Illinois were performed at a rapid shearing rate, which is more representative of usual loading conditions than is the conventional slow triaxial shear test. Three different samples are tested at confining pressures of 5, 10, and 15 psi to determine the shear strength properties, friction angle, and cohesion of the aggregate materials. In rapid shear tests, a high loading rate of 1.5 in./s is applied, instantly causing 12.5% deformation in a 12-in. high specimen; the loading rates in such tests are higher than those in conventional triaxial shear tests. Figure 41 shows the deformed shape of an aggregate specimen after completion of the test.

Because of the high loading rate, the University of Illinois rapid shear strength test gives slightly higher peak stress results than do the conventional shear strength tests (see Figure 42). Although not conservative, the rapid shear tests are believed to better simulate the conditions of the actual pavement layer under the dynamic application of a moving wheel load.

FIGURE 41 Deformed sample after completion of University of Illinois Rapid Shear Monotonic Triaxial Strength Test.

Repeated Load Triaxial Testing

Repeated load triaxial testing has received major emphasis as a means for evaluating in the laboratory the modulus–deformation characteristics of granular materials and subgrade soils. Both resilient modulus and permanent deformation accumulation can be quantified based on the appropriate repeated-load testing data. Resilient modulus testing requires pneumatic or servohydraulic loading, a data acquisition system with feedback control, a personal computer with an integrated software package, modern equipment, a good technician, and careful equipment calibration. The equipment must be capable of producing load pulse duration of approximately 25 to 150 ms. The load pulse is generally repeated 15 to 30 times a minute. Specimen deformation over the entire length (or in some cases a portion of the specimen) typically is measured

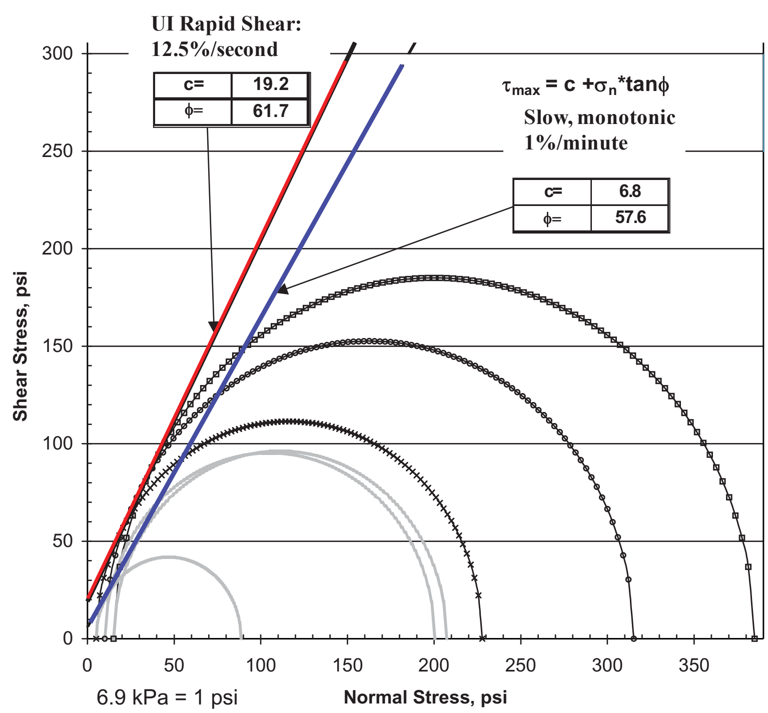


FIGURE 42 Conventional slow and rapid shear strength test results on a crushed stone.

with "externally" or "internally" mounted linear variable differential transformers (LVDTs).

The modulus and permanent deformation tests are performed on a cylindrical shaped solid specimen subjected to repeated axial compressive stresses. The specimen is subjected to a constant or variable (pulsed) all-around confining pressure to simulate the field stress condition. The cyclic application of the deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) distinguishes the modulus and permanent deformation tests from the static triaxial tests. Air or fluid is commonly used to provide the all-around confining stress, and the vertical deviator stress is applied with a servopneumatic or servohydraulic actuator onto a specimen placed between top and bottom platens (see Figure 43).

The conventional repeated-load triaxial tests use a simple and convenient arrangement to apply stresses in the vertical and horizontal directions. Little friction between the loading system and specimen is generated during the test. The stress state in a sample remains fairly uniform, especially in the middle one-third section. Most importantly, the simplicity and the lower cost are the reasons the conventional repeated-load triaxial test setup is widely used for aggregate characterization. Yet, in this type of a conventional triaxial test, only an all-around CCP is applied while the vertical deviator stresses are pulsed. Special triaxial testing devices with the capability of pulsing confining stresses offer an advanced material characterization by simulating various dynamic stress states under experienced moving wheel loads. Modulus and permanent deformation tests that consider the application of such realistic stress states often are referred to as VCP tests.

Figure 44 shows the principal stresses applied in a repeated load triaxial test apparatus. The typical stress states

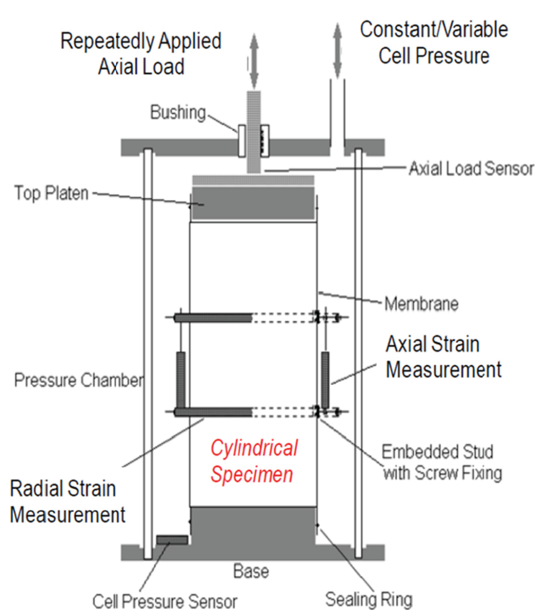


FIGURE 43 Repeated load triaxial testing apparatus.

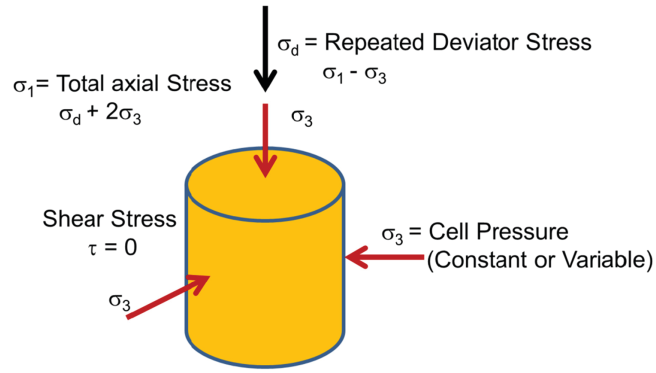


FIGURE 44 Stresses applied on a cylindrical specimen in repeated load triaxial testing.

applied on the specimen are according to the CCP conditions with cell pressure not pulsed in the triaxial chamber. The VCP-type, repeated-load triaxial tests, on the other hand, offer much wider loading possibilities by better simulating actual field conditions because in the pavement structure the confining stress acting on the material is cyclic in nature. The inherent differences between the CCP and VCP tests are such that in the VCP tests (1) the confining pressure is also cycled in phase with the axial deviator stress and (2) the axial specimen deformations generally are larger owing to the lack of a constant all-around confinement on the specimen.

A Strategic Highway Research Program (SHRP) Testing Protocol (P46—Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils) was developed in the United States for conducting standard M_R tests on unbound aggregate materials. SHRP P46 was used in testing the various granular material and subgrade soil samples collected in support of the SHRP (FHWA) LTPP program. A "round robin" type evaluation was conducted with the SHRP P46 Protocol. The results were very helpful in a priori M-E design activities. SHRP P46 was first approved as an AASHTO Interim Method of Test (AASHTO T 294-92 I, Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils—SHRP Protocol P46), then carried the designation T294-94 in the 1995 AASHTO specifications. Another standard for resilient modulus (AASHTO T 292), which was originally developed in 1991, was still active until 2003. A new test standard (AASHTO T 307) was introduced in 1999, leading to the existence of two resilient modulus test specifications (AASHTO T 292 and AASHTO T 307) until 2003. AASHTO T 292 was withdrawn in 2003, and AASHTO T 307 became the only standard for resilient modulus testing. AASHTO recommends the T307-99 (2003) repeated load triaxial test as a standard test for resilient characterization of pavement materials in the United States.

In general, CCP-type triaxial test conditions are used for M_R testing of granular materials in the United States according to the SHRP P46 and AASHTO T307-99 test protocols. The test specimens are subjected to 15 stress states in which the pulsed dynamic stresses (σ_d) range from 21 to 276 kPa in the axial direction, and the confining pressures (σ_3) range from 21 to 136 kPa. All applied stress states are in general below the failure stress conditions and applied following a CCP test condition with a loading stress path slope of $m = 3.0$.

The conditioning stage requires applying a confining pressure of 103.4 kPa and a minimum of 500 (up to 1,000) repetitions of a load equivalent to a deviator (maximum axial) stress of 103.4 kPa. Considering that the conditioning test data are often used for permanent deformation characterization, this stress state, which only corresponds to a conditioning stress ratio (σ_1/σ_3) of 2, may not be high enough to properly shake down granular materials before M_R testing. The M_R testing stage requires applying 100 repetitions of the corresponding cyclic stress using a haversine-shaped load pulse and recording the average recovered vertical deformations for each LVDT separately for the last five cycles.

No doubt the findings from the NCHRP 1-28 project (Barksdale et al. 1997), SHRP LTPP studies (*LTPP Materials Characterization: Resilient Modulus of Unbound Materials—LTPP Protocol P46 Laboratory Startup and Quality Control Procedures,* FHWA-RD-96-176), and the recent NCHRP 1-28A study on "Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design" greatly helped in preparing and updating the current SHRP TP P46 and the AASHTO T307-99 (2003) test protocols, which are adopted for routine use in the MEPDG. It is also apparent that resilient testing procedures for granular materials are still undergoing development and refinement.

In Europe, the final draft European standard for M_R testing by the European Committee for Standardization (CEN) is the EN 13286-7 (2004), "Unbound and Hydraulically Bound Mixtures—Test Methods—Part 7: Cyclic Load Triaxial Test for Unbound Mixtures. This European Standard specifies test procedures for determining the resilient and permanent behavior of unbound mixtures under conditions that simulate the physical conditions and stress states of these materials in pavement layers subjected to moving loads. These procedures allow determining mechanical properties that can be used for performance ranking of materials and for calculating the structural responses of pavement structures. Testing procedures similar to those of EN 13286-7 adopted in the United Kingdom can be found in the British standard BS EN 13286-7 (2004). Note that the European standard specifies test methods to characterize both the resilient and permanent deformation behavior of unbound aggregates, whereas the AASHTO T 307 focuses only on evaluating the resilient behavior. Moreover, the European standard incorporates both VCP (method A) and CCP (method B) loading conditions, whereas AASHTO T 307 uses only CCP conditions.

Need for Permanent Deformation Testing

Rutting or accumulation of permanent deformation is the primary damage/distress mechanism of UAB/subbase layers in pavements. Accordingly, rutting resistance is a major performance measure for designing pavements with granular base/subbase layers. Granular base/subbase permanent deformation may contribute significantly to the overall flexible pavement surface ruts. Low-strength granular materials generally are more susceptible to higher permanent deformation accumulation. However, a properly compacted UAB/subbase layer comprising crushed particles adequately prevents settlement and any lateral movement in the layer through high shearing resistance and contributes significantly to the dissipation of wheel load stresses. The NCHRP 4-23 study identified shear strength of unbound aggregates as one of the most significant mechanistic properties influencing pavement performance (Saeed et al. 2001). Moreover, shear strength property, rather than "resilient modulus" (M_R), always has been shown to better correlate with unbound aggregate permanent deformation behavior for predicting field rutting performance (Thompson 1998; Tao et al. 2010; Xiao et al. 2012).

Although the influence of stress state on unbound aggregate resilient modulus is relatively well understood, its influence on the actual performance—rutting, cracking, roughness—of flexible pavements is less clearly known in practice. The design domains in which the influence of stress state is significant are also poorly defined. Note that it is not uncommon to have two different aggregate materials with very poor and excellent rutting characteristics possess similar high modulus properties from laboratory M_R testing (Mishra and Tutumluer 2011). Accordingly, it is never possible to evaluate aggregate base course rutting performances from just the M_R tests conducted on aggregates for modulus characterization and mechanistic pavement analysis. This is because computed elastic responses, such as the vertical resilient strain (e_v) within an aggregate base/subbase layer, can never be properly correlated with their permanent strain/deformation independent of the material's shear strength. Furthermore, permanent deformation accumulation of a particular layer also depends significantly on the level of wheel load stress applied in relation to the aggregate material's strength under confinement, which is often represented by the stress/strength ratio (the percentage of strength that is reached upon loading at that same layer confinement) and closely linked to the material's "shakedown" limits (Werkmeister et al. 2004).

Accordingly, repeated-load triaxial tests need to be conducted on unbound aggregate materials to study the accumulation of permanent deformation under loading. Such tests can be used for different purposes, such as ranking of materials, evaluation of maximum allowable stress levels, and predicting permanent deformation accumulation in pavement layers (CEN 2004). The European Standard (CEN 2004) makes use of three different "shakedown zones," as defined by Werkmeister (2003), to rank unbound aggregate materials based on the permanent deformation test results. These shakedown zones correspond to certain load-deformation behavior trends identified by significantly different permanent strain accumulation rates under repeated loading. For a detailed discussion on shakedown zones, the reader is directed to the work by Werkmeister (2003). *Note that none of the test procedures currently available in the United States (AASHTO T 307 or NCHRP 1-28A) covers permanent deformation testing and characterization of unbound aggregates.*

Key Lessons

- The CBR test is a commonly used index test to estimate the shear strength of unbound aggregates.
- Although triaxial tests for shear strength, resilient modulus, and permanent deformation behavior give more realistic estimates of unbound aggregate behavior under loading, conducting such tests requires significant investments in equipment and personnel training.
- Test procedures for conducting resilient modulus tests on aggregates, AASHTO T 307 and NCHRP 1-28A, have been available for more than a decade. These specifications can adequately capture the stress-dependent nature of unbound aggregates and are ready to be implemented in practice.
- Although permanent deformation behavior has been established as a more direct indicator of pavement performance compared with resilient modulus, no standard test procedure is available in the United States or Canada governing the testing of aggregates for permanent deformation.
- New research efforts should be focused on developing harmonized protocols for quantifying the permanent deformation behavior of aggregates.

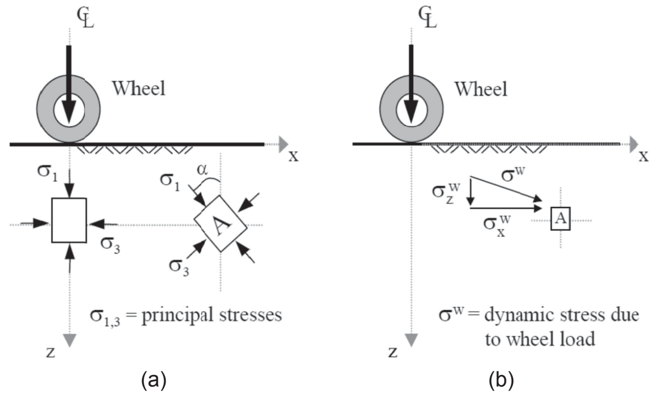


FIGURE 45 Stress conditions in a granular base to consider for advanced aggregate characterization: (*a*) Rotation of principal stress directions. (*b*) Stresses in extension loading (Seyhan and Tutumluer 1999).

Innovative Devices for Advanced Triaxial Characterization of Unbound Aggregates

Traditional triaxial testing equipment, operating under CCP conditions, cannot simulate the rotation of principal stress directions experienced by a pavement element under moving wheel loads. Such equipment is only capable of applying one constant stress path representing the stress states immediately under the wheel loading. However, as discussed, because of the moving nature of the wheel load, the major principal stress often is not aligned in the vertical direction and rotates in the direction of the applied load, as shown in Figure 45a. Thus, the total principal stress on a pavement element rotates as the wheel passes.

Advanced triaxial test devices operating under VCP conditions offer the capability to apply different combinations of stress paths by pulsing both the confining (cell) pressure and the vertical deviator stress. Such stress path loading tests better simulate actual field conditions because the confining pressures acting on a representative soil element in a pavement structure also are cyclic in nature (see Figure 45b). Typically, at a distance away from the centerline of loading, the horizontal component of dynamic wheel load can become greater in magnitude than the vertical component. In that event, an extension type of loading is more critical on top of the base. Advanced triaxial testing devices can simulate such loading conditions because of their ability to apply confining pressures (σ_3 for a cylindrical specimen) that are larger in magnitude that the vertical stresses (σ_1 for a cylindrical specimen).

Several researchers have found that the resilient strains measured from CCP tests are smaller than those from VCP tests even under similar peak stress levels (Shackel 1974; Allen and Thompson 1974). Allen and Thompson (1974) also found that the Poisson's ratio values obtained from CCP tests were significantly higher than those obtained from VCP tests. Brown and Hyde (1975) suggested that similar resilient moduli could be obtained from CCP and VCP tests by ensuring that the confining stress in the CCP test was equal to the mean value of stress used in the VCP test. However, Nataatmadja (1989) observed that the resilient moduli obtained from CCP tests were higher than those from VCP conditions, even when the stress states followed a pattern similar to that proposed by Brown and Hyde (1975). Thus, it is apparent that characterization of unbound granular materials under CCP conditions may only overestimate the resilient moduli, thus leading to inadequate structural design of pavements.

Similarly, repeated load triaxial testing of unbound aggregates incorporating stress path rotation usually results in higher permanent deformation accumulations compared with tests conducted under constant stress path loading. Accelerated loading tests carried out at the University of Nottingham in the United Kingdom showed that moving wheel loading or actual trafficking is more damaging to a pavement system than is running a repeated plate loading test on the same pavement system (Brown and Brodrick 1999). They also observed that bidirectional loading causes more severe permanent deformation development when shear stress reversals owing to oscillating wheel loads are considered. Similar findings were also reported from a full-scale pavement experiment undertaken in France to study the behavior and performances of unbound granular materials as pavement granular layers (Hornych et al. 2000). The permanent strains accumulated in the granular layers under moving wheel loading were about *three* times as large as those under cyclic plate loads.

Tutumluer and Kim (2003) studied typical airport granular base/subbase course materials at various densities through single and multiple stress path laboratory tests. They observed that multiple stress path tests always resulted in much higher permanent volumetric and shear strains than those of the single path tests. Thus, their findings indicate that actual traffic loading, simulated by the multiple path tests, can cause greater permanent deformations or rutting damage, especially in the loose base/subbase, than can dynamic plate loading or a constant confining pressure-type laboratory test. Figure 46 shows the stress states applied by Tutumluer and Kim (2003) to evaluate the effects of multiple stress path tests on permanent deformation accumulation in unbound aggregate specimens.

From advanced triaxial testing incorporating loading of the aggregate specimen using multiple stress paths, they observed that the specimen axial strains (ε_1) were considerably lower in magnitude than the radial (ε_3) strains owing to the proper specimen compaction effort in the vertical direction and the VCP type multidirectional stress pulsing. They also observed that both the volumetric and deviatoric strains obtained from the multiple stress path tests were consistently higher than the ones from the single path tests. The maximum or peak values of all multiple stress path permanent strains at the elevated load cycles were significantly higher than those from the single path test procedure. Their findings suggested that moving wheel load effects should be properly accounted for in laboratory testing to better predict unbound aggregate layer performance under typical highway and airport pavement loads (Tutumluer and Kim 2003).

It is therefore important to use advanced triaxial testing devices capable of incorporating variable confining pressure conditions to properly account for the effects of moving wheel loads on unbound aggregate behavior. However, such test devices are usually very expensive, so their use has been limited. Some of the most well-known advanced triaxial testing devices, developed through research, capable of better simulating the stress conditions in a pavement structure are discussed here.

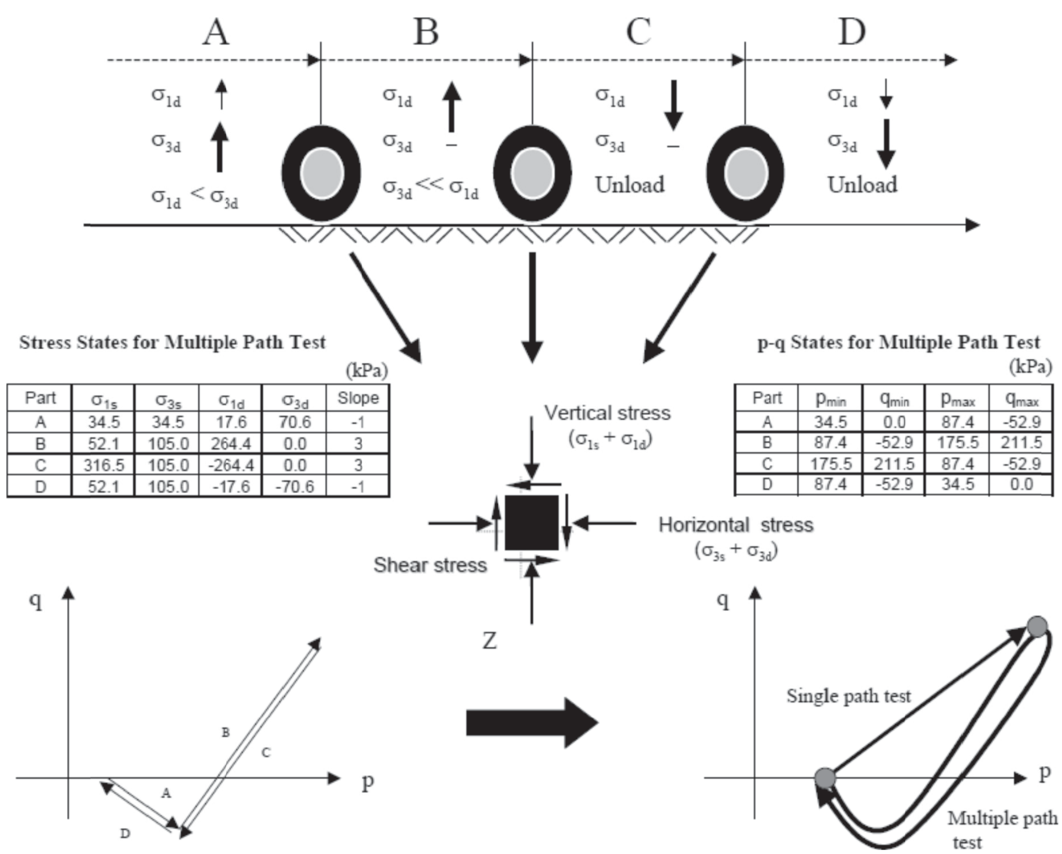


FIGURE 46 Concept map of multiple stress path tests compared to single path tests (Tutumluer and Kim 2003).

K-Mould

Semmelink and De Beer (1995) introduced a sophisticated laboratory test system, called the K-Mould device, for the rapid determination of the elastic and shear properties of pavement materials as developed by CSIR in South Africa. In the K-Mould device, an axial load is applied to soil/aggregate specimen contained in a segmented, thick-walled cylinder (see Figure 47). The segments are held in place with springs whose stiffnesses are chosen to simulate typical lateral stiffnesses of, for example, aggregates in granular bases. Lateral stress is mobilized by an elastic support system with a stiffness that can be varied between 15 and 60 MPa and designed to simulate in situ conditions. This provides a state somewhere between K_0 (zero lateral strain) and unconfined. Thus, the lateral stress depends on the total lateral strain mobilized by the applied vertical stress against the horizontally mounted radial disc springs. It was shown that K-mold was relatively rapid and cost-effective for determining the engineering properties, such as the modulus, friction angle ϕ, and the cohesion intercept, *c*. Both elastic and permanent deformation properties of unbound granular materials and soils were studied, including quantification of the stress sensitivity and anisotropic nature of these materials.

According to an assessment by the European COST Action 337 (2002) research program study, the K-Mould possibly may offer an alternative test to the cyclic/repeated load triaxial test, but considerable developmental work is required before this would be suitable for routine application. However, it has the potential to assess modulus and permanent deformation behavior characteristics, and sample preparation may be simpler.

University of Illinois FastCell

An innovative laboratory cyclic/repeated load triaxial testing device, the University of Illinois FastCell (UI-FastCell), was introduced by Tutumluer and Seyhan (1999) to have provisions for applying static and dynamic stresses in both vertical and radial direction by the use of the two independently controlled stress channels. This advanced triaxial setup was a custom-designed system superior to the commercially available rapid triaxial tester equipment, RattCell, manufactured by the Australian Industrial Process Controls (IPC), Ltd. company. With UI-FastCell, higher magnitudes of radial stresses can be pulsed by the use of a triaxial chamber filled with hydraulic oil. The UI-FastCell was designed mainly for the purpose of determining in the laboratory the anisotropic and dynamic properties of unbound aggregates through stress path testing in VCP conditions. Because it is not possible to reorient the granular samples in the triaxial cell, applying and switching of the various stress states on the same specimen allowed for adequately determining the inherent and load-induced anisotropy. The device is also suitable for simulating field stress conditions in the laboratory and for studying the effects of principal stress rotation as a result of moving wheel loads that involve a change in total shear stress direction.

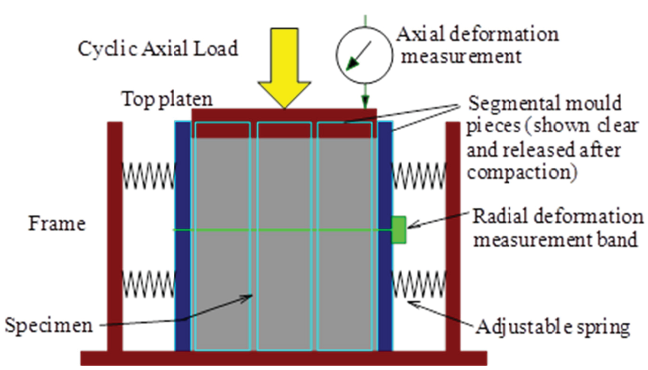


FIGURE 47 Schematic of K-Mould testing equipment (Semmelink and De Beer 1995).

The UI-FastCell uses a fluid/air interface to minimize compressibility effects when conducting tests in which the horizontal stress on a specimen must be cycled. This is useful for investigating anisotropic effects and the response to loading in which a 90° rotation of planes of principal stress is important. The cell also provides a capability for on-specimen displacement measurements, which eliminate problems associated with compliance of the machine used to load the specimen. When on-specimen vertical displacements are used as well, end effects are eliminated.

Figure 48a shows an unbound aggregate specimen (6-in. diameter, 6-in. height) being prepared in a split mold for testing using the University of Illinois FastCell. Figure 48b shows the specimen setup under the loading ram of the UI-FastCell, with the confining chamber in a raised position. Figure 48c shows a picture of the confinement cell lowered down around the specimen for the testing position. An air actuator applies the axial pressure, and the confining pressures are cycled through a hydraulic fluid within the rubber membrane. The driving cylinders on the back of the confining cell (not shown in the picture) include an air-fluid interface that provides fast application and switching of the dynamic loading. Some of the advanced features of the UI-FastCell, as discussed by Tutumluer and Seyhan (1999), are as follows:

- Measurement of on-specimen vertical and radial displacements and axial force and displacement external to cell;
- A bladder-type horizontal confinement chamber with a built-in membrane that is inflated to apply variable confining pressures during vertical cyclic loading;
- Ability to independently cycle either vertical or radial loading/confining pressures in phase or out of phase, in compression or extension type loading;
- Ability to reverse principal loading direction on the same specimen with applied radial pulse stresses exceeding the vertical ones.

The UI-FastCell and the IPC RattCell advanced triaxial devices are fundamental research tools when compared with

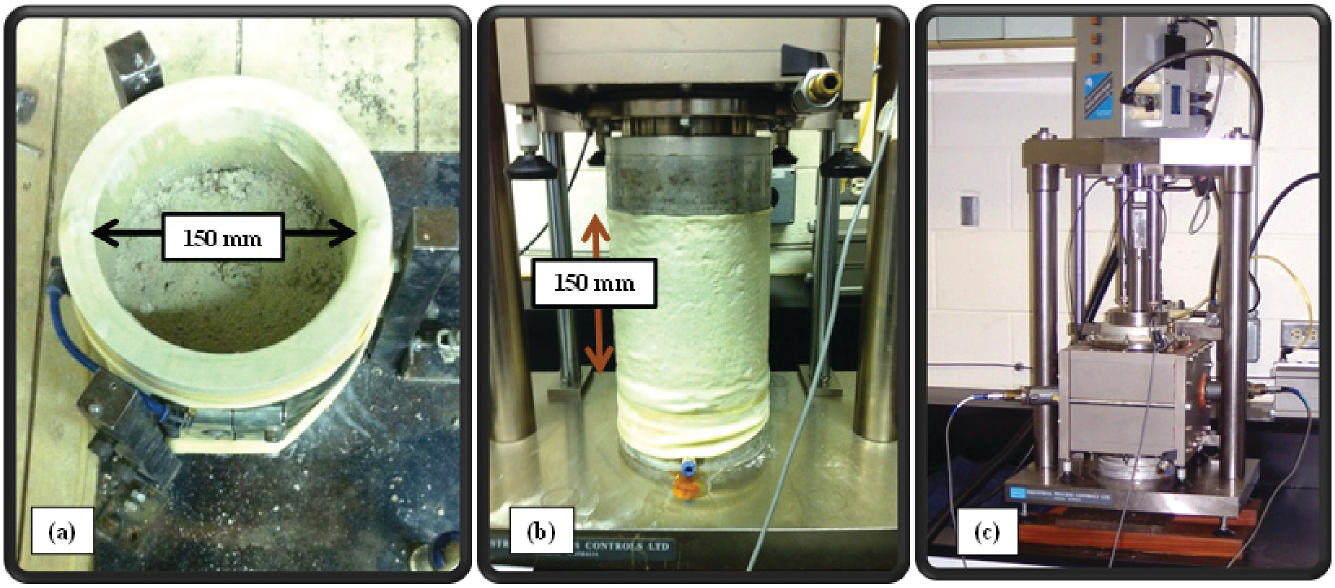


FIGURE 48 University of Illinois—FastCell.

the conventional repeated load/cyclic triaxial testing equipment. Using the UI-FastCell, the following important laboratory testing considerations can be addressed (Tutumluer and Seyhan 1999): (1) the aggregate specimen can be anisotropically consolidated (K_0 condition in the field); (2) various stress paths experienced under a rolling wheel load can be adequately applied; (3) anisotropic aggregate resilient moduli can be conveniently obtained by pulsing vertical and radial stresses; and (4) different orientations of principal stresses can be achieved by independently applying vertical and radial stresses (that is, major principal stress direction is not limited to only 0 or 90 degrees with the horizontal).

Springbox

The Springbox equipment (Edwards et al. 2005) is a suitable tool for testing unbound granular and some weak hydraulically bound mixtures (see Figure 49). It consists of a steel box containing a cubical sample of unbound aggregate material, of edge dimension 170 mm, to which a repeated load can be applied over the full upper surface. One pair of the box sides is fully restrained, and the other is restrained through elastic springs, giving a wall stiffness of 10–20 kN per mm. The equipment enables a realistic level of compaction to be applied to the test material by means of a vibrating hammer and also includes a facility to introduce water to the sample or drain water from its underside. Loading takes the form of repeated vertical load applications of controlled magnitude at a frequency of at least 1 Hz and no greater than 5 Hz. The load capacity is equivalent to a vertical stress of at least 150 kPa. Measurements of both vertical and horizontal (spring restrained) deflections can be made with two measurement transducers for each measure. In the case of vertical deflection measurement, the equipment allows the transducers to make direct contact with the specimen through holes in the loading platen. The stiffness modulus of the material can be calculated from the averaged deflections measured over a series of loading patterns.

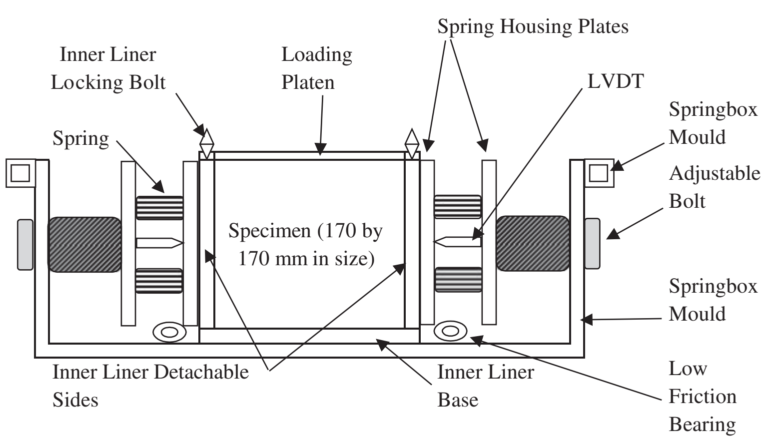


FIGURE 49 Schematic of the Springbox testing equipment (Edwards et al. 2005).

The Springbox device provides a relatively rapid and economic test method for determining modulus behavior of aggregate materials in an accelerated fashion, ranking materials susceptibility to permanent deformation, and assessing materials durability and moisture sensitivity. The Springbox test method is currently included in the first revision of the 2009 Design Guidance for Road Pavement Foundations (draft HD25) in the United Kingdom.

Key Lessons

- Several researchers have developed innovative devices to simulate the actual stress conditions induced in an unbound aggregate layer under moving wheel loads.
- Despite the ability of these devices to better predict pavement behavior under loading, their use in practice is not feasible because of equipment cost and personnel training requirement concerns.

Interpretation of Repeated Load Triaxial Test Data

Data collected during repeated load testing of unbound aggregates can be analyzed to calculate the resilient modulus (M_R) and permanent deformation (δ_p) values as indicators of aggregate layer performance under loading. Figure 50 presents a schematic of typical deformation behavior of unbound granular materials under repeated loading. As can be seen from the figure, with increasing number of load applications, the material rapidly accumulates permanent deformation under the first few cycles. This can be attributed to the rearrangement of individual particles in the aggregate matrix. However, as the number of load applications increases, the rate of accumulation of permanent deformation gradually decreases, and all the deformation corresponding to each loading cycles is resilient (recoverable) in nature. The initial reorientation of particles often is said to correspond to the compaction and construction phases in a pavement layer. Thus, for an in-service pavement, all the deformation under vehicle loading ideally would be recoverable in nature. The resilient modulus of a material is usually calculated after all the particle reorientation has taken place.

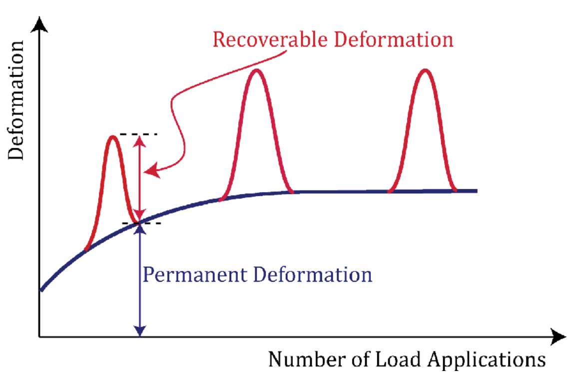


FIGURE 50 Behavior of unbound granular materials under repeated loading.

In the case of conventional triaxial tests (CCP conditions), the resilient modulus (M_R) and the Poisson's ratio (ν) can be obtained from the measured recoverable strains using axisymmetric stress-strain relations as follows:

$$\varepsilon_1 = \frac{\sigma_d}{M_R} \tag{7}$$

$$\varepsilon_3 = -\nu\frac{\sigma_d}{M_R} \tag{8}$$

In the case of advanced triaxial tests conducted under VCP conditions, both the vertical and horizontal stresses are pulsed. Thus, the resilient modulus (M_R) and the Poisson's ratio (ν) need to be obtained from the measured recoverable strains using axisymmetric stress-strain relations as follows:

$$\varepsilon_1 = \frac{\sigma_{1d}}{M_R} - 2\nu\frac{\sigma_{3d}}{M_R} \tag{9}$$

$$\varepsilon_3 = -\nu\frac{\sigma_{1d}}{M_R} + (1-\nu)\frac{\sigma_{3d}}{M_R} \tag{10}$$

where σ_{1d} and σ_{3d} are the pulsed stresses in the vertical and horizontal directions, respectively; and ε_1 and ε_3 are the recoverable strains in the axial and radial directions, respectively. In using the earlier equations to obtain resilient parameters as constants, an assumption is made that the material behaves linearly and elastically for any individual stress state.

In addition, interpretations of anisotropic moduli with both σ_{1d} and σ_{3d} are that the pulsed stresses require the consideration of vertical modulus (M_R^V), horizontal modulus (M_R^h), in-plane Poisson's ratio (ν_h), and out-of-plane Poisson's ratio (ν_V) to be obtained from the measured recoverable strains using axisymmetric stress-strain relations as follows:

$$\varepsilon_1 = \frac{\sigma_{1d}}{M_R^V} - 2\nu_V\frac{\sigma_{3d}}{M_R^h} \tag{11}$$

$$\varepsilon_3 = -\nu_V\frac{\sigma_{1d}}{M_R^V} + (1-\nu_h)\frac{\sigma_{3d}}{M_R^h} \tag{12}$$

Current Resilient Modulus Models

Resilient moduli of granular materials increase with increasing stress states (stress-hardening), especially with confining pressure and/or bulk stress and slightly with deviator stress (Lekarp et al. 2000a). Resilient behavior of unbound aggregate materials can be reasonably characterized by using stress-dependent models that express the modulus as nonlinear functions of stress states. Such a characterization model must include in the formulation the two triaxial stress conditions (that is, the confining pressure σ_3 and the deviator stress σ_d or the applied mean pressure p and the deviator stress q) to account for the effects of both confinement and shear loading. The model parameters traditionally are obtained from the multiple regression analyses of the repeated load triaxial test data. Currently available models to predict the resilient modulus of granular materials are extensively discussed in Appendix D.

Current Permanent Deformation Models

Constitutive relationships often need to be developed to properly describe permanent deformation accumulation in unbound granular materials with number of load applications. A summary of the different models proposed by many researchers to predict permanent strain as a function of load and material property-related factors appears in Appendix E.

HISTORICAL DEVELOPMENT IN UNBOUND AGGREGATE CHARACTERIZATION FOR PAVEMENT DESIGN

More and more sophisticated geotechnical concepts have been introduced into AASHTO pavement design guides with the release of each successive version. Christopher et al. (2010) presented an extensive overview of the geotechnical inputs used in different AASHTO pavement design methods. The following sections present a summary of the discussion presented by Christopher et al. (2010).

1961 Interim Pavement Design Guide

The AASHO 1961 *Interim Design Guide* used the concept of structural number (*SN*) to account for the contribution of individual layers to pavement structural capacity.

$$SN = a_1D_1 + a_2D_2 + a_3D_3 \tag{13}$$

where D_1, D_2, and D_3 are the thicknesses (inches) of the surface, base, and subbase layers, respectively, and a_1, a_2, and a_3 are the corresponding layer coefficients. For the materials used in the AASHO road test, the values for the layer coefficients were fixed at 0.44, 0.14, and 0.11, respectively. Because the parameters in the design equation were primarily based on the materials used in the AASHO road test, there was no scope for geotechnical material input in the design procedure.

1972 Interim Pavement Design Guide

The 1972 *Interim Design Guide* (AASHO 1972) attempted to extend findings from the AASHO Road Test to foundation, material, and environmental conditions different from those at the test site. Several new features for the flexible and rigid pavement design were introduced, along with a rudimentary overlay design procedure.

Guidelines were given for estimating the structural layer coefficients a_1, a_2, and a_3 for materials other than those used in the AASHTO road test. These guidelines were developed based on a survey of state highway agencies regarding the values for the layer coefficients that they were using in design for various materials. For example, the recommended range of a_2 values (for untreated base layers) was from 0.05 to 0.18. Similarly, the structural layer coefficients for subbase (a_3) could range from 0.05 to 0.14. Each agency was recommended to rely on past experience to establish appropriate layer coefficient values. The 1972 *Guide* also introduced an empirical soil support scale to account for the different environmental conditions experienced based on geographical locations.

1986 Pavement Design Guide

The 1986 *Pavement Design Guide* introduced the concept of resilient modulus in a rational attempt to better characterize subgrade soil and unbound aggregate materials. The structural layer coefficients for base (a_2) and subbase (a_3) were estimated through correlations with resilient modulus. However, these relations for the structural layer coefficients were largely empirical and based primarily on engineering judgment with only limited amounts of data.

Drainage coefficients were also introduced into the structural number (*SN*) expression to accommodate different drainage conditions. Accordingly, the *SN* expression given in Equation 13 was modified to incorporate drainage coefficients (m_i), as given here:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \tag{13a}$$

where m_2 and m_3 are the drainage coefficients for the base and subbase layers, respectively. The empirical values for m_i, which are specified in terms of quality of drainage and the estimated percentage of time the layer will be near saturation, range from 0.4 to 1.4.

Similarly, the rigid pavement design procedure in the 1986 *Pavement Design Guide* incorporated seasonal adjustments to the effective modulus of subgrade reaction. This effective modulus of subgrade reaction was a function of seasonally adjusted values for the subgrade and subbase resilient modulus. A drainage coefficient (C_d) was also introduced to account for drainage conditions under rigid pavements.

1993 Pavement Design Guide

The 1993 design guide was similar to the 1986 *Pavement Design Guide* as far as the design of new flexible and rigid pavement structures were considered. The primary emphasis of this design guide was on rehabilitation design.

NCHRP 1-37A Pavement Design Guide

The MEPDG developed through NCHRP Project 1-37A (2004) incorporated new models of material behavior with the recognition that all pavement materials are exposed to and significantly affected by environmental, or climatic, factors.

Input Levels

A refinement in the MEPDG was the use of a hierarchical design approach. Such an approach provided the designer with several levels of "design efficacy" that could be related to the class of highway under consideration or to the level of reliability of design desired. A chosen higher level of design output implied that the inputs were also of a higher level. In keeping with the hierarchical approach, materials characterization was comprised of three levels, with Level 1 indicative of a design approach philosophy of the highest practically achievable reliability and Levels 2 and 3 of successively lower reliability.

The Level 1 analysis was a "first-class" or advanced design procedure to provide for the highest practically achievable level of reliability. For the unbound aggregate resilient modulus (M_R) characterization, the laboratory-determined inputs (k_1, k_2, and k_3 parameters) also were of the highest practically achievable level and generally required site-specific data collection or testing.

Level 2 inputs provided an intermediate level of accuracy and were closest to the typical procedures used with earlier editions of the AASHTO *Guide.* This level was recommended when resources or testing equipment were not available for tests required for Level 1. Level 2 inputs typically were user-selected, possibly from an agency database, could be derived from a limited testing program, or could be estimated through empirical correlations, such as resilient modulus and CBR relationships.

Level 3 inputs provided the lowest level of accuracy. This level was recommended for design where there were minimal consequences of early failure (e.g., lower-volume roads). Inputs typically were user-selected values or typical averages for the region. One example would be the use of default unbound material resilient modulus values identified from aggregate material soil classification.

The MEPDG used resilient modulus (M_R), obtained from the NCHRP 1-28A and AASHTO T307, as the primary material property for all unbound pavement layers and subgrade soils. Furthermore, the Level 1 inputs for the granular base, subbase, and subgrade required the characterization of the nonlinear, stress-dependent M_R behavior for these layers.

MEPDG Level 2 presented a correlation for determining modulus as a function of the layer coefficient as follows: M_R (psi) = 30,000 $(a_i/0.14)^3$. A major disadvantage of this equation was that the layer coefficient has to be determined or estimated before estimating the resilient modulus. Other empirical correlations used for Level 2 design were:

$$M_R = 1155 + 555(R) \text{ psi} \tag{14}$$

$$M_R = 2555(CBR)^{0.64} \tag{15}$$

MEPDG Level 3 resilient modulus inputs for a new design were obtained from soil property correlations and nomographs

DARWin-ME

The recent release of the DARWin-ME, the software package implementing the Mechanistic Empirical Design Procedure, does not consider the stress dependence of unbound aggregate resilient modulus. The earlier implementation of the M-E pavement design procedure in the public domain MEPDG software explicitly included stress dependence of unbound resilient moduli as Level 1 inputs, but this capability has been removed from the new DARWin-ME software implementation. Instead, DARWin-ME uses empirical correlations corresponding to Level 2 of the MEPDG to compute the layer resilient modulus values. To ensure proper characterization of unbound aggregate layers and accurate prediction of pavement base/subbase layer performance under loading, stress dependence of unbound aggregate modulus needs to be incorporated into future versions of DARWin-ME.

Key Lessons

- Resilient modulus of aggregates is used as a critical input in M-E pavement design methods.
- Current AASHTO pavement design methods do not consider the stress-dependence of unbound aggregate resilient modulus.
- It would be useful to incorporate the stress dependence of unbound aggregate materials into future releases of DARWin-ME, the current AASHTO mechanistic-empirical pavement design procedure.

STATE OF THE PRACTICE IN UNBOUND AGGREGATE CHARACTERIZATION AND DESIGN

Background

The survey of state and Canadian provincial transportation agencies conducted under the scope of this synthesis study aimed to assess the state of the practice in unbound aggregate material characterization for design of pavement layers. Important findings from the survey are discussed in this section.

When asked about the personnel responsible for the testing/characterization of unbound aggregate materials for use in pavement base/subbase layers, 39 of 46 (~85%) respondents indicated that the design was done by the agency geotechnical/materials laboratory. Only one agency (Wisconsin) delegated the characterization of such materials to a university laboratory under a research subcontract. Several agencies specify a constant modulus value and AASHTO 1993 layer coefficient

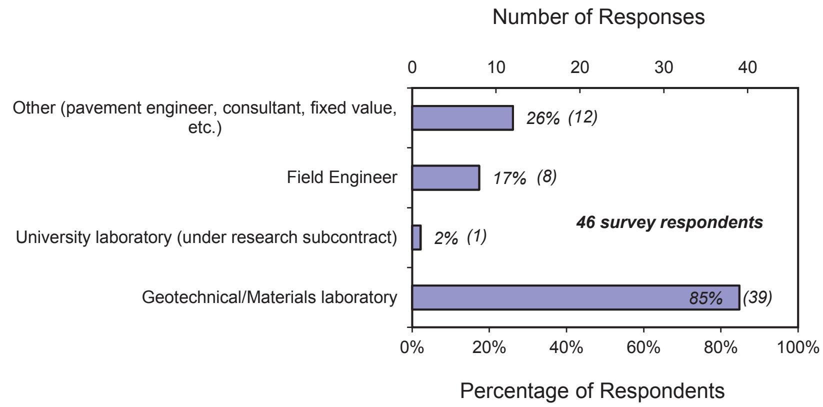


FIGURE 51 Personnel responsible for characterization and design of unbound aggregate pavement layers in the surveyed transportation agencies.

for the aggregate materials irrespective of their source. For example, the practice in Oregon is to use a constant aggregate layer modulus of 20,000 psi and a layer coefficient (AASHTO 1993) of 0.10. The Alberta (Canada) transportation agency also reported a practice of specifying a constant layer coefficient for UAB/subbase layers irrespective of the aggregate sources. Figure 51 shows the distribution of personnel responsible for aggregate material characterization for design, in different agencies surveyed under the scope of this synthesis effort.

Fourteen of 46 respondents reported the use of repeated-load triaxial tests for resilient modulus characterization. Twelve agencies do not conduct any test to characterize the strength, modulus, and permanent deformation characteristics of unbound aggregate materials. The use of strength index tests such as CBR or Hveem stabilometer appears to be a common practice among transportation agencies (41% of respondents, as shown in Figure 52).

FWD testing appears to be the most common practice among transportation agencies for strength, deformation, and modulus characterization of in-service unbound aggregate pavement layers; more than 65% of respondents reported its use (see Figure 53). Only one agency (Maryland) indicated the use of GeoGauge, whereas four states (Maryland, Louisiana, Oklahoma, and Indiana) reported the use of light weight deflectometers (LWD). Seventeen (17) agencies reported not measuring the strength, modulus, or deformation characteristics of in-service aggregate layers. Several agencies use density as the only indicator of constructed aggregate layer quality.

Twenty-seven of 46 respondents conduct laboratory/field tests to characterize aggregate materials for use in granular base and subbase layers on a project-need basis (see Figure 54). Six agencies do not conduct any laboratory/field tests on aggregates.

When asked about the method used to design pavements with UAB/subbase layers, 28 agencies (~61%, see Figure 55) reported using the AASHTO 1993 procedure. Four agencies use the AASHTO 1972 design guide, whereas 14 have adopted the MEPDG into their agency specifications.

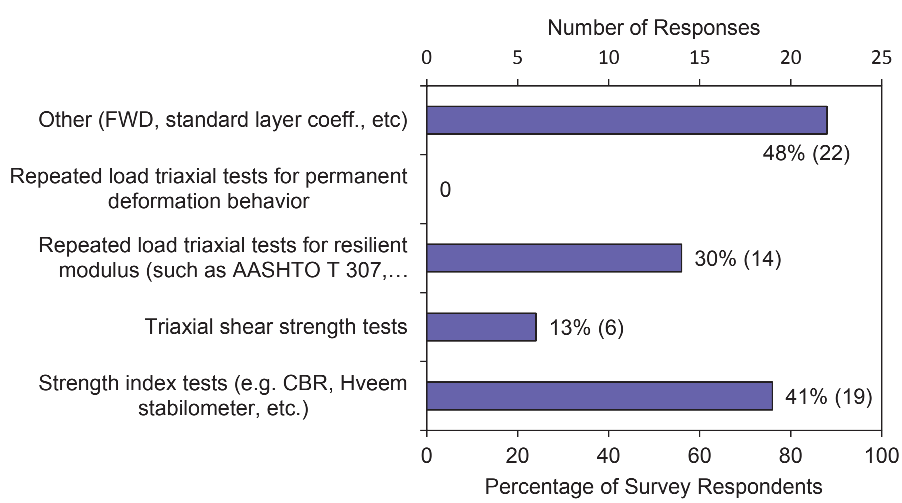


FIGURE 52 Different test methods conducted by transportation agencies for strength, deformation, and modulus characterization of unbound aggregate materials used in base and subbase course applications.

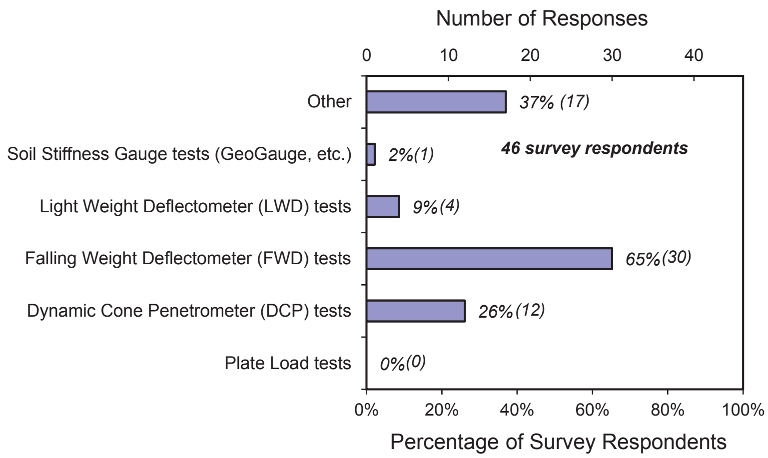


FIGURE 53 Field tests conducted by transportation agencies for strength, deformation, and modulus characterization of in-service unbound aggregate pavement layers.

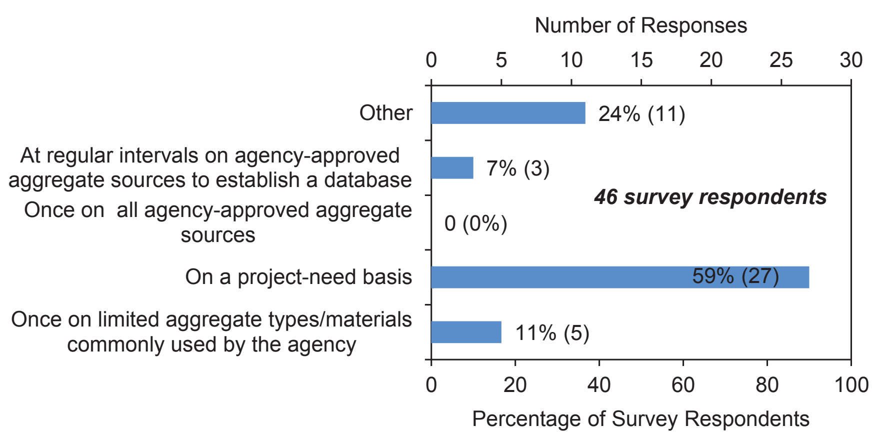


FIGURE 54 Frequency of laboratory/field tests conducted to characterize aggregate materials for use in granular base and subbase layers.

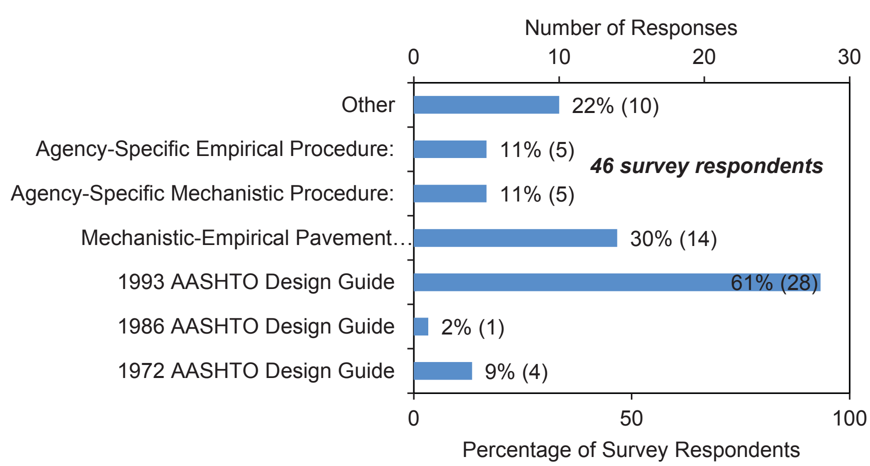


FIGURE 55 Different methods used by agencies to design pavements with UAB and subbase layers.

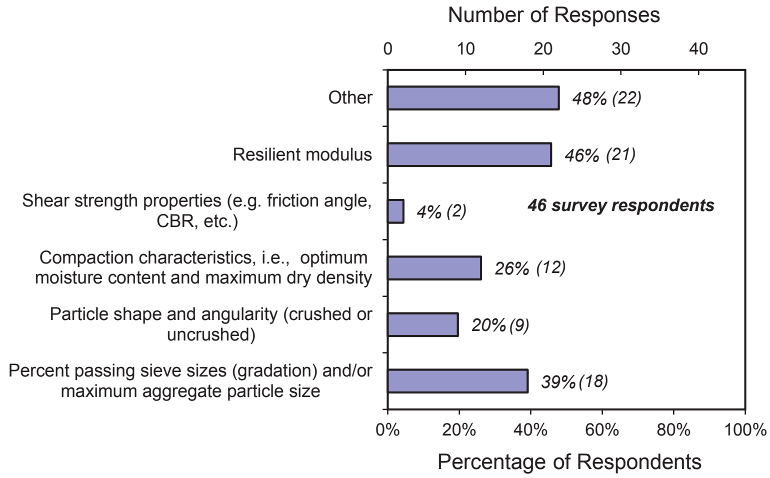


FIGURE 56 Aggregate properties/characteristics used by agencies as inputs for the design of pavements with UAB and subbase layers.

Resilient modulus appears to be the most commonly used aggregate property (used by 21 of 46 respondents) serving as an input for the design of pavement structures (see Figure 56). Only two agencies (Utah and Saskatchewan province in Canada) reported using aggregate shear strength as an input for pavement design. Twenty-two agencies reported other practices, such as the use of AASHTO-specified layer coefficients for designing pavements with unbound aggregate layers without using any material specific property. Twenty-six of 46 respondents assign a single modulus value to the entire aggregate layer without considering the stress-dependency of aggregate materials (see Figure 57). Ten agencies do not use modulus in the pavement design process, whereas only one agency (Oklahoma) incorporates the anisotropy of aggregate layers into its pavement design procedure.

Only 10 agencies conduct resilient modulus testing in the laboratory to determine the modulus of unbound aggregate materials for use in granular base and subbase layers. Twenty-two use empirical correlations to predict the resilient modulus from index properties such as CBR or aggregate gradation parameters. Although the use of in-place modulus measurement using FWD and LWD appears to be fairly common (used by 30% of the respondents, as shown in Figure 58), several agencies do not test unbound aggregate materials for resilient modulus and adopt generic values during their pavement design procedures.

Conclusions from Survey of Transportation Agencies

The survey of state and Canadian provincial transportation agencies indicated that there is a wide variety in agency practices as far as unbound aggregate material characterization and layer design is concerned. A significant gap appears to exist between the state of the art and state of the practice concerning unbound aggregate material characterization and pavement layer design procedures. Although it is widely recognized that aggregate shear strength, resilient modulus, and permanent deformation characteristics affect the performance of base and subbase layers in pavement systems, several agencies do not use these properties in their pavement design procedures.

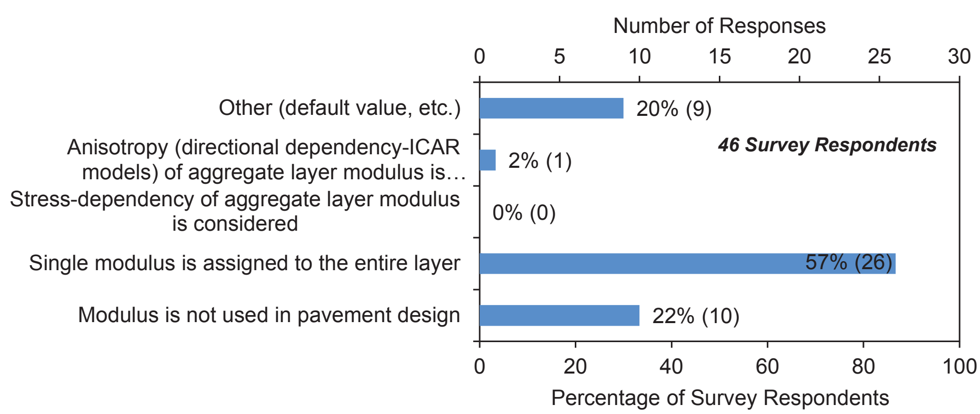


FIGURE 57 Different approaches used by agencies for assigning resilient modulus values to UAB and subbase layers.

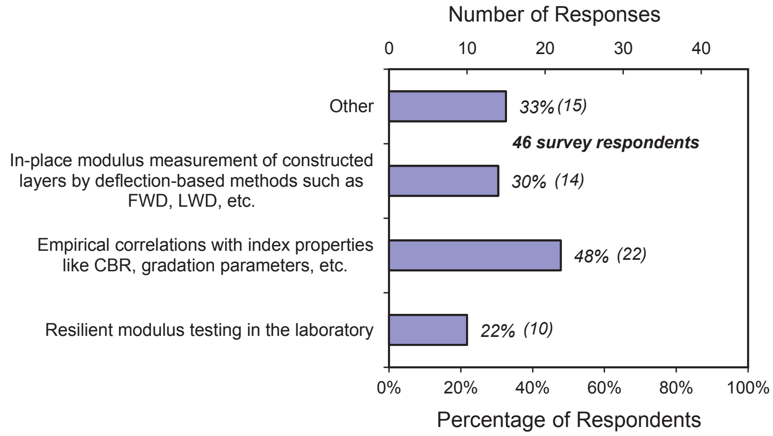


FIGURE 58 Methods to determine the resilient modulus of unbound aggregate materials for use in granular base and subbase layers.

The initial survey of transportation agencies conducted during this synthesis study reflected that 14 of the responding agencies conducted resilient modulus (M_R) testing on unbound aggregates before using them in pavement base/subbase layers. A follow-up survey of these agencies was conducted to gather further information on the prevalent state of the practice as far as M_R testing is concerned. Conversations with agency personnel indicated that although these agencies have the equipment to conduct M_R testing on unbound aggregates, performing this test is not a common practice. Although some of these agencies frequently conduct M_R testing on subgrade soils, using a constant modulus value for the base/subbase layer appears to be the preferred alternative. Difficulties associated with specimen preparation, obtaining reliable data, and personnel training appeared to be the most common factors responsible for the agencies not conducting M_R tests of unbound aggregates. The agencies that do occasionally conduct M_R tests on aggregates appeared to be unsure regarding making use of the test data in their respective pavement design procedures. Detailed findings from the follow-up survey of these 14 agencies are documented in Appendix F of this report.

A harmonized approach may need to be developed that will recommend to state and provincial transportation agencies appropriate laboratory tests and design methods for unbound aggregate pavement layers.

Key Lessons

- A significant gap appears to exist between the state of the art and the state of the practice concerning unbound aggregate material characterization and pavement layer design procedures.
- Resilient modulus testing of unbound aggregates is not a common practice among U.S. state and Canadian provincial transportation agencies.
- Several agencies possess the required equipment for conducting resilient modulus tests on aggregates. However, difficulties associated with specimen preparation, obtaining reliable "good quality" data, and personnel training have resulted in agencies often assigning constant modulus values to UAB/subbase layers during pavement design.
- Agencies would benefit from conducting resilient modulus testing on locally available aggregates for better consideration of unbound aggregate properties in pavement design.

STATE-OF-THE-ART METHODS FOR UNBOUND AGGREGATE LAYER CHARACTERIZATION AND DESIGN

Proper consideration of compaction-induced residual stresses in granular materials would no doubt more appropriately model the behavior of unbound aggregate layers in pavements. The stress path approach by Uzan (1985) and experiments performed by Selig (1987) and Zeilmaker and Henny (1989) are useful for estimating the magnitudes of residual stresses existing in the granular layers because of compaction or preloading of the pavement layers. Knowing these residual (locked-in) horizontal stresses is essential for determining the most accurate initial stress state to evaluate correctly the resilient modulus values used in any mechanistic pavement analysis.

Stress Path Testing

To better characterize unbound aggregate layer modulus and deformation behavior, it is important to properly simulate in the laboratory actual field loading conditions. The pavement in the field usually is loaded by moving wheel loads, which at any time impose varying magnitudes of normal and shear

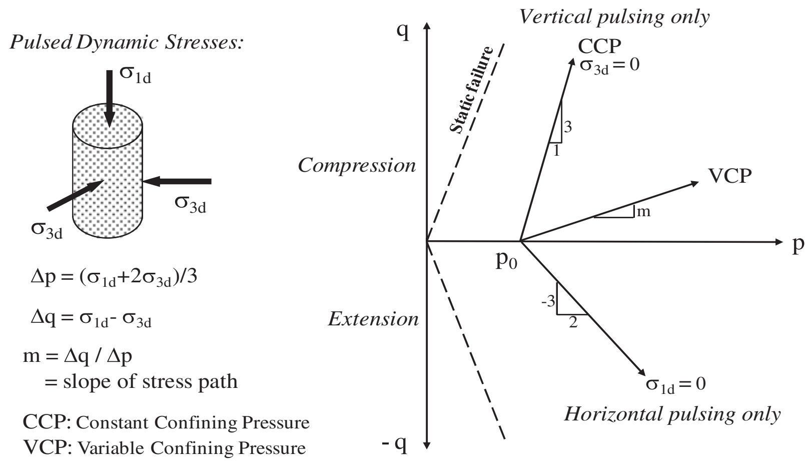


FIGURE 59 Stress paths that can be studied in an advanced triaxial setup such as UI-FastCell.

stresses in the aggregate layer, as reflected by the rotation of the principal stresses. This type of loading cannot be ideally simulated in the laboratory by the CCP-type repeated-load triaxial tests, such as the AASHTO T307-99. It is possible to apply only one constant stress path ($m = \Delta q/\Delta p = 3$; see Figure 59) in the CCP tests. However, the VCP-type repeated-load triaxial tests offer the capability to apply a wide combination of stress paths by pulsing both cell pressure and deviator stress (see Figure 59). Such stress path loading tests better simulate actual field conditions because in the pavement structure the confining stresses acting on the material are cyclic in nature.

Stress path tests can be performed by switching stress states from CCP to VCP, depending on the stress path slope that is subject to be studied. The VCP tests require pulsing of the stresses in both vertical and horizontal directions. For example, to test a specimen with a stress path slope of $m = 1.5$, horizontal dynamic stresses should be one-fourth of the vertical dynamic stresses in magnitude; that is, $\sigma_{1d} = 4*\sigma_{3d}$ (see Figure 43). Furthermore, extension stress states correspond to conditions of an approaching or departing wheel load at a distance with horizontal dynamic stresses exceeding the vertical ones. Kim and Tutumluer (2005) showed that under a moving wheel load, an extension-compression-extension stress cycling occurs to involve shear stress reversals.

Directional (Anisotropic) Modulus Testing

Unbound aggregate pavement layers exhibit higher stiffness characteristics in the vertical wheel loading direction than in the horizontal direction. This directional dependency is a special type of anisotropy, known as cross-anisotropy, caused by the preferred orientation of the aggregate to which both the shape characteristics of the aggregate and the compaction and traffic loading contribute. A cross-anisotropic representation has different material properties (i.e., elastic modulus and Poisson's ratio) assigned in the horizontal and vertical directions. Thus, for a cylindrical triaxial sample, a realistic assignment of in-plane and out-of-plane moduli is achieved under axial symmetry with the axial modulus increasing relative to the radial one.

Tutumluer and Seyhan (1999) considered the extreme stress conditions that may exist in the base layer of a flexible pavement structure under a moving wheel load. Then, considering these extreme compression and extension loading conditions, deviator stresses are pulsed either in the vertical or horizontal directions only (see Figure 43). If the tested specimen is made up of a material that is truly isotropic in behavior, the moduli determined from the two-extreme loading conditions should be similar in magnitude. Accordingly, the laboratory findings of Tutumluer and Seyhan (1999) from four aggregates tested using UI-FastCell indicated definite directional dependency (anisotropy) of aggregate moduli. The resilient moduli computed in the vertical and radial pulsing directions using a consistent set of isotropic stress-strain equations varied pronouncedly with the applied stress states. The vertical moduli typically were higher than the horizontal moduli for most aggregates tested, except for sandy gravel having a significant amount of P_{200} fines.

The research project 502 conducted at the ICAR focused on determining structural issues of unbound aggregate layers for a proper representation in a mechanistic-based design of flexible pavements (Adu-Osei et al. 2001; Tutumluer et al. 2001). The research team developed models for the resilient and permanent deformation behavior from the results of advanced triaxial tests conducted at the Texas Transportation Institute (TTI) using the IPC RattCell and at the University of Illinois using the UI-FastCell. The ICAR research team also developed a resilient modulus testing protocol, which although significantly different from the AASHTO T307-99 protocol, is not more complicated. The studies mainly indicated that the UAB material is to be modeled as nonlinear and cross-anisotropic to account for stress sensitivity and the significant differences between vertical and horizontal moduli and Poisson's ratios. With anisotropic modeling, a more realistic stress distribution could be achieved in UABs.

A recent state-of-the-art paper summarized the most significant work accomplished in the past 15 years in the area of anisotropic and stress-dependent modulus behavior of UABs used in flexible pavements (Tutumluer 2009). Findings of past research studies on both the laboratory and field validations of the anisotropic aggregate behavior were discussed in detail. The most important result of properly accounting for the anisotropic stiffnesses of compacted granular base/subbase layers is that critical pavement design parameters, such as vertical deviator stress and strain on top of the base course and the subgrade, are predicted to be typically higher than those computed when traditional isotropic pavement models are used. Note that these critical pavement responses are directly related to the degree and rate of permanent deformation in the base course and subgrade layers, and this is the substantial proportion of the overall pavement rutting in low- to medium-volume roads with thin asphalt surfaces. Therefore, traditional isotropic design approaches run the risk of under-designing flexible pavements or over-estimating the number of design axle loads the pavement can withstand.

Effect of Cross-Anisotropy on Pavement Analysis and Design

Traditional mechanistic pavement design methods use linear elastic programs that consider only isotropic material properties in granular base layers to predict deflections, stresses, and strains in the pavement structure. However, the assignment of a single modulus to the entire layer does not correctly model base stiffness owing to stress variations in both vertical and horizontal directions. This is one of the reasons the linear elastic programs predict significant tensile stresses at the bottom of the base layer in most cases. However, because unbound aggregate layers are not capable of withstanding tensile stresses, such predicted stress states are not realistic representations of the actual stress states in an unbound aggregate pavement layer.

Barksdale et al. (1989) observed from instrumented test sections that a linear cross-anisotropic model of an UAB was at least equal to, and perhaps better for, predicting general pavement response than the simplified contour model proposed by Brown and Pappin (1981), which requires elaborate testing. Tutumluer and Barksdale (1995) modeled the same test sections employing cross-anisotropic resilient properties in the base layer and using the nonlinear model proposed by Uzan (1985) to represent resilient modulus. Considerably lower horizontal tensile stresses were predicted in the granular base when the horizontal resilient modulus was equal to 15% of the vertical resilient modulus. Using this anisotropic modeling approach, reasonably good agreement was achieved with measured values of the resilient behavior for as many as eight response variables at the same time.

Karasahin et al. (1993) also reported results of a study in which the applicability of various resilient constitutive models of granular material was investigated for use in unbound base layers. An anisotropic volumetric-deviatoric model by Elhannani (1991) was found to give the best results for modeling the resilient behavior for the following two loading conditions: (1) only the deviator stress was cycled, and (2) both deviatoric and confining pressures were cycled in a triaxial test.

Early work in characterizing the anisotropic modulus properties of unbound aggregate layers used in flexible pavements was carried out at the Georgia Institute of Technology and the University of Illinois (Tutumluer 1995; Tutumluer and Thompson 1997a). Anisotropic modeling of a typical flexible pavement resulted in the magnitudes of both the horizontal and shear stiffnesses throughout the base being only small fractions of the vertical stiffness (Tutumluer 1995; Tutumluer and Thompson 1997a). Unlike the results of the isotropic-type analysis, the horizontal stiffnesses were found to be much lower when compared with the vertical values. These stiffnesses were not assumed in the base layer but predicted by the nonlinear stress-dependent models obtained directly from the triaxial specimen behavior. Both the important effects of load-induced directional stiffening and the dilative behavior of granular materials under applied wheel loading were successfully modeled using a cross-anisotropic approach (Tutumluer 1995; Tutumluer and Thompson 1997a).

Tutumluer and Thompson (1997b) modeled conventional flexible pavements using the GT-PAVE FE program and observed that, unlike the findings of isotropic analyses, a certain set of aggregate types and properties used in the granular layer typically resulted in horizontal stiffnesses varying between 3% and 21% of the vertical and the shear stiffnesses between 18% and 35% of the vertical throughout the base. As shown in Figure 60, the horizontal stiffness ratios (M_R^h/M_R^V) were low under the wheel load, 0.08 to 0.12 from the contour lines near the centerline, and increased radially away from the centerline to reach a value of 1 at approximately 6 load radii, which corresponds to the isotropic case. These stiffnesses were not assumed in the base layer but predicted by the anisotropic, nonlinear stress-dependent models developed from triaxial test data. The effects of compaction-induced residual stresses locked in granular bases also were of significance, especially when calculating horizontal stiffnesses. Such stresses offset any low-magnitude tensile stresses and provided adequate confinement radially away from the wheel load (Tutumluer and Thompson 1997a, b; Garg et al. 1998). A procedure was also established for estimating cross-anisotropic properties from repeated load triaxial tests with only vertical deformation measurements (Tutumluer and Thompson 1997a; Tutumluer 1998).

Using the UI-FastCell, a large stress excursion analysis was conducted to characterize unbound aggregate layer behavior under various stress path loadings. Seyhan et al.

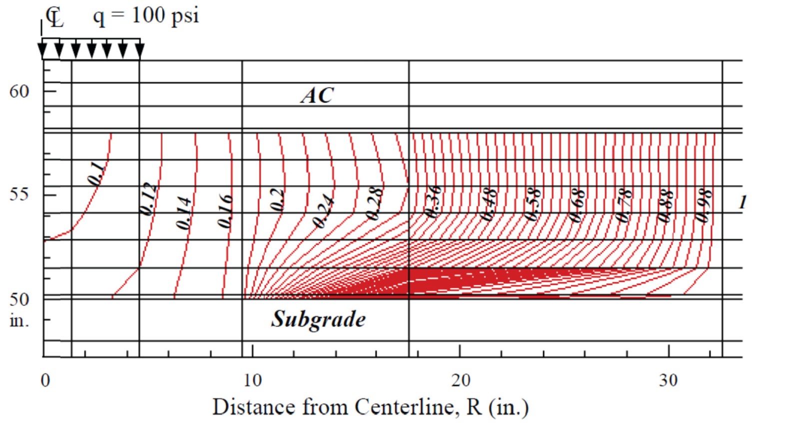


FIGURE 60 Horizontal stiffness ratio (M_R^h/M_R^V) distribution throughout the base in the presence of 20.7 kPa horizontal residual stresses (after Tutumluer and Thompson 1997b).

(2005) presented a new methodology for determining cross-anisotropic aggregate base properties (i.e., directional dependency of moduli and Poisson's ratios as inputs into mechanistic pavement analysis considering effects of actual traffic or moving wheel loading). The proposed materials characterization requires conducting constant stress path triaxial tests and incrementally varying loading stress path slopes at similar stress states that are representative of various moving wheel loading conditions in the laboratory. In accordance, cross-anisotropic aggregate properties were determined by varying slightly the stress path slopes during testing and then by employing an error minimization approach to interpret the test results. Crushed aggregate specimens were prepared and tested to obtain cross-anisotropic properties at five different stress path slopes representative of various moving wheel-load–induced compression and extension pavement stress states. Vertical resilient moduli were commonly found to be larger than horizontal ones, and critically low vertical resilient moduli were obtained for some extension states (Seyhan et al. 2005).

Simplified Procedure for Determining Anisotropic Model Parameters

Based on the data presented by Hicks (1970), Allen (1973), and Crockford et al. (1990), Tutumluer and Thompson (1998) established a procedure for estimating cross-anisotropic properties from repeated load triaxial tests in which only vertical deformations were measured (the standard procedure, i.e., AASHTO T307-99). To characterize the typical variations of horizontal and shear stiffness ratios, they analyzed a conventional flexible pavement section with anisotropic stiffness models used in a 203-mm thick granular base. The models were obtained from the multiple regression analyses of 50 triaxial test results on different aggregates obtained from the works of Hicks (1970), Allen (1973), and Crockford et al. (1990). Three stress-dependent M_R models were used to completely define the resilient granular material behavior in vertical, horizontal, and shear planes, as follows:

$$M_R = K_A p_a \left({}^{I_1}\!/_{p_a} \right)^{K_B} \left({}^{\tau_{oct}}\!/_{p_a} \right)^{K_C} \qquad (16)$$

where $\mathrm{M_R}$ is the resilient modulus; $I_1 = \sigma_1 + \sigma_2 + \sigma_3 = \theta$ = first stress invariant or bulk stress; $\tau_{oct} = 1/3 \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$ = octahedral shear stress; p_a = atmospheric pressure (100 kPa or 14.7 psi); K_A, K_B, and K_C are material constants obtained from regression analyses of repeated-load triaxial test data. The three cross-anisotropic moduli (M_R^V, M_R^h, and G_R) were modeled using the same formulation, and the model parameters used were as follows:

	Coefficient	I_1 Exponent	τ_{oct} Exponent
Horizontal resilient modulus (M_R^h)	K_1	K_2	K_3
Vertical resilient modulus (M_R^v)	K_4	K_5	K_6
Resilient shear modulus (G_R)	K_7	K_8	K_9

Therefore, the stiffness ratios, (M_R^h/M_R^v) and (G_R/M_R^v), could be expressed in terms of the coefficients, (K_1/K_4) and (K_7/K_4), respectively. Tutumluer and Thompson (1998) observed that the constant terms in the stiffness ratio models (K_1/K_4 or K_7/K_4) were good approximations for the horizontal and shear stiffness ratios (M_R^h/M_R^v and G_R/M_R^v) predicted by the FE analyses under the wheel load. Figure 61 shows the variations of the constant terms in the shear (K_7/K_4) and horizontal (K_1/K_4) stiffness ratio models obtained from tests performed on a variety of crushed and partially crushed aggregates and gravel. Although somewhat scattered, the data points plotted at various saturation levels clearly indicated an increasing trend of K_7/K_4 (thus G_R/M_R^v) with K_1/K_4 (thus M_R^h/M_R^v). The dotted lines plotted around the data define the lower and upper bounds for

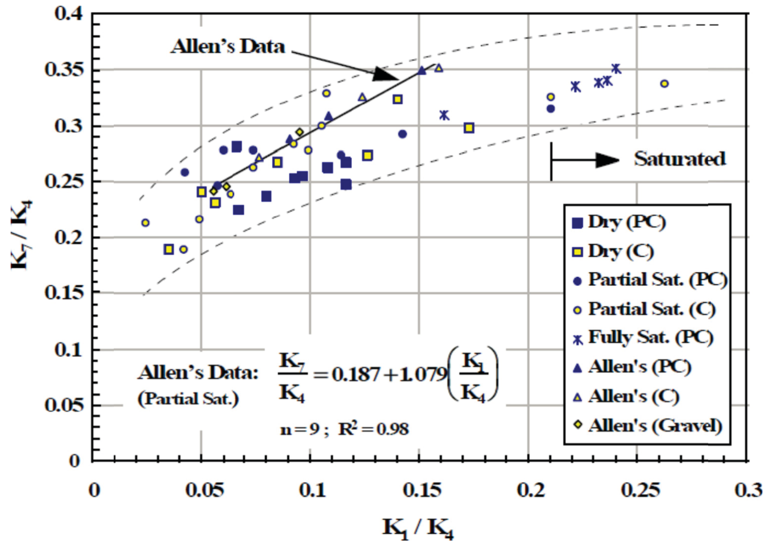


FIGURE 61 Variation of constant ratios in horizontal and shear stiffness ratio models (after Tutumluer and Thompson 1998). C = crushed; PC = partially crushed.

a typical variation of K_7/K_4 with K_1/K_4 from triaxial test results for which the horizontal and shear stiffnesses proportionally increase or decrease. Accordingly, a granular material with high shear and horizontal stiffnesses would have a reduced tendency to lateral spreading under wheel loads.

Figure 61 (Tutumluer and Thompson 1998) also shows a linear relationship found to exist between the constant shear ratio K_7/K_4 and the constant horizontal ratio K_1/K_4 for a consistent set of nine test results reported by Allen (1973). The standard estimated error (SEE) in the equation (see Figure 61) was given as 0.00636 for K_7/K_4. To estimate horizontal and shear model parameters, Tutumluer and Thompson (1998) proposed an additional equation relating the shear model constant parameter K_7 with the vertical model parameters K_4, K_5, and K_6, as follows (1 psi = 6.89 kPa):

$$K_7(psi) = -90.92 + 0.27K_4 + 305.34K_5 + 158.22K_6$$
$$(R^2 = 0.94,\ \text{SEE} = 178\ \text{psi}) \qquad (17)$$

Figure 62 shows for the 50 test results the deviator stress exponents (K_3-K_6 or K_9-K_6) plotted with the bulk stress exponents (K_2-K_5 or K_8-K_5) as obtained from the horizontal and shear stiffness ratio models. In both plots, the data points are generally centered on the equality line, indicating that they are equal in magnitude but opposite in sign. Overall, these plots indicate that when the deviator and bulk stresses take similar values, K_1/K_4 and K_7/K_4 primarily determine the stiffness ratios.

According to the earlier outlined simplified procedure by Tutumluer and Thompson (1998), these steps can be followed to estimate the shear and horizontal model parameters when the experimentally determined vertical modulus models (i.e., K_4, K_5, and K_6 are established from conventional repeated load triaxial test results) are known:

1. Use Equation 17 to compute K_7;
2. Compute the constant ratio K_7/K_4;

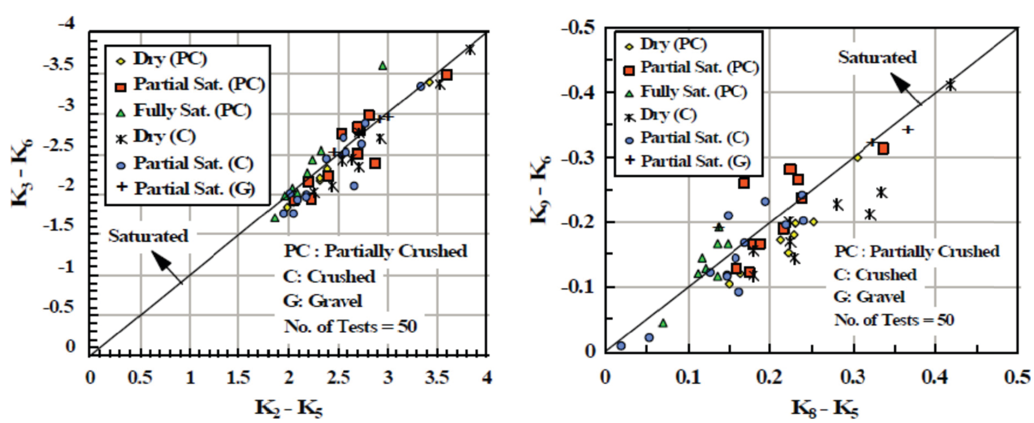


FIGURE 62 Variation of stress exponents in the horizontal and shear stiffness ratio models (after Tutumluer and Thompson 1998).

3. Use the upper and lower band as well as Allen's linear fit indicated in Figure 62 to obtain the corresponding K_1/K_4 constant ratio;
4. From Figure 63, select values equal in magnitude but opposite in sign for the stress exponents K_2-K_5 and K_3-K_6 to be used in the horizontal stiffness ratio model (an approximate value of 2.5 has been used as suggested by Tutumluer and Thompson, 1998); and finally
5. From Figure 63, select values equal in magnitude but opposite in sign for the stress exponents K_8-K_5 and K_9-K_6 to be used in the shear stiffness ratio model (an approximate value of 0.2 has been used as suggested by Tutumluer and Thompson 1998). Note that because of the low to nonexistent horizontal compressive confining pressures under the wheel load, approximating these stress exponents does not have any significant effect in the overall anisotropic dilative behavior of granular bases.

Recent ICAR Procedure for Determining Anisotropic Model Parameters

Based on the ICAR test protocol established for determining stress-dependent anisotropic M_R properties of unbound aggregate materials (Adu-Osei et al. 2001; Tutumluer et al. 2001), Ashtiani and Little (2009) developed a methodology for designing aggregate mixtures for base courses. A comprehensive aggregate database was developed to identify the contribution level of different aggregate materials and base course features to the directional dependency of material properties. Accordingly, to characterize the level of anisotropy in unbound aggregate systems, the fitting parameters in material models (k-values) were used as characterizers of the level of anisotropy, which can vary considerably depending on aggregate base properties such as gradation, saturation level, and the geometry (that is, shape properties of the aggregate particles). Three aggregate sizes for each of the 10 aggregate sources were tested for angularity, form, and texture using Aggregate Image Measurement System (AIMS), and the distributions were fitted to two parameter cumulative Weibull distributions. Three gradations (fine, intermediate, and coarse) of the aggregate materials were used to determine dry density and moisture states of aggregate systems used in the aggregate database to account for the effects of optimum, dry of optimum, and wet of moisture conditions on directional dependency of material properties. From anisotropic modulus testing, the k model parameters were determined to capture the stress sensitivity, nonlinearity, and anisotropic behavior of the aggregate systems studied in the laboratory. Among the particle geometry features in the aggregate database, the vertical to horizontal modular ratio (E_h/E_v) was found to be most sensitive to the degree of elongation of the aggregate particles or how cubical the aggregate particles were. In their study, Ashtiani and Little (2009) also developed a new mechanistic performance protocol based on plasticity theory to ensure the stability of the pavement foundations under traffic loads.

Field Validations

As part of the ICAR 502 research project, field validation data were collected from two previous full-scale pavement test studies: the TTI and Georgia Tech studies (Tutumluer et al. 2001). The validation of the nonlinear anisotropic behavior of UABs was accomplished by analyzing these full-scale pavement test sections using TTI-PAVE and GT-PAVE FE analysis programs, predicting UAB responses and comparing them with the measured ones.

The TTI project dealt with two flexible pavement test sections, one with a thin and the other with a thick asphalt surface layer, built at the TTI Research Annex. The base course in each pavement was a crushed Texas limestone meeting the Texas DOT Grade 1, Item 248, aggregate base specifications. The test sections were instrumented with multidepth deflectometers (MDDs), and an FWD was positioned directly over the MDDs and at several different positions away from MDD and the pavement responses (deflections) collected. FWD data were used to backcalculate material properties of the two pavement sections. For validation of the anisotropic resilient behavior, the limestone was characterized in the laboratory according to the ICAR testing protocol. Based on the FWD surface deflections and MDD depth deflections, several computer runs were made using the TTI-PAVE FE program. The linear elastic analyses had much higher errors between the measured and the predicted when compared with those obtained from the nonlinear isotropic and cross-anisotropic analyses. The nonlinear cross-anisotropic material models used in the base layer predicted vertical deflections closest to field deflections (Tutumluer et al. 2001).

The Georgia Tech full-scale pavement test study (Barksdale and Todres 1983) had provided the original field data for the anisotropic base modeling study conducted by Tutumluer (1995). The pavements studied consisted of three conventional sections and two inverted sections, which had an UAB sandwiched between an upper asphalt concrete surfacing and a lower cement-stabilized subbase. A total of eight response parameters, stresses, and strains at different locations in the test sections and surface deflections were measured in each test using strain gages, pressure cells, and LVDTs. After characterizing the crushed granitic gneiss used in the test sections for cross-anisotropic properties through advanced laboratory tests, Tutumluer et al. (2001, 2003) further analyzed the Georgia Tech test sections using the GT-PAVE FE program at different locations in the test sections considering several methods of UAB characterization for comparison and field validation. These methods included (1) a linear elastic, isotropic analysis; (2) a linear elastic, cross-anisotropic analysis; (3) a nonlinear, stress-sensitive isotropic analysis; (4) characterization of the vertical resilient modulus as nonlinear stress sensitive according to a model similar to that of Uzan (1985) and then assuming that the horizontal modulus is some percentage of the vertical modulus (work done by Tutumluer 1995); (5) a nonlinear, stress-sensitive, cross-anisotropic analysis using modulus models developed following the laboratory SID approach

(Adu-Osei et al. 2001); and (6) a nonlinear, stress-sensitive, cross-anisotropic analysis with model parameters obtained from a simplified procedure that uses AASHTO T307-99 resilient modulus test results and was adopted earlier by Tutumluer and Thompson (1998). The accuracy of the overall modeling of resilient behavior of both the conventional and inverted sections was related to how well the measured response variables were predicted at the same time. Only when a nonlinear cross-anisotropic model was used in the UAB (either method 4 or method 6), were the resilient behaviors of five pavement test sections predicted reasonably accurately for as many as eight response variables (i.e., displacements, stresses, and strains) from the same analysis. The resilient moduli computed in the horizontal direction, typically in the range of 12% to 27% of the vertical, were shown to correctly predict the horizontal and vertical measured strains in the UAB (Tutumluer et al. 2003).

More recent field validations of anisotropic UAB behavior have been reported by Masad et al. (2006), Steven et al. (2007), and Kwon et al. (2008). Masad et al. (2006) successfully demonstrated the efficacy of using anisotropic aggregate properties to represent unbound layers by comparing AASHO road test pavement surface deflection measurements under wheel loads to FE predictions based on models that incorporated isotropic and anisotropic properties for the unbound base and subbase layers. The surface deflections in the flexible pavements of the AASHO road test were selected for this comparison because the AASHO road test is such a widely used database and because of the tight control of traffic, pavement cross sections, and material quality at the road test (Masad et al. 2006). The deflection predictions correlated best with the experimental measurements when the horizontal moduli were about 30% of the vertical moduli in the UAB layers.

Steven et al. (2007) performed elastic nonlinear FE analyses of a flexible pavement section, which was instrumented and tested in the New Zealand CAPTIF full-scale pavement test facility subjected to varying FWD loads. An inductive coil soil strain system was installed in the test section to measure vertical compressive strains within the granular and subgrade layers, and pressure cells were used to measure the vertical compressive stresses. The measured values of stress and strain at the top of the subgrade were used to give an indication of the stiffness. In an effort to match the measured FWD deflections and the vertical strain profile in the pavement section with the FE predictions, a nonlinear anisotropic modulus model with $n = M_R^h/M_R^v$ as low as 0.15 had to be assigned in the granular layer.

Kwon et al. (2008) reported on the resilient response predictions of instrumented full-scale pavement test sections, both geogrid base reinforced and control sections, studied under single and dual wheel loadings at the University of Illinois. A mechanistic FE model, which considers the nonlinear, stress-dependent pavement foundation as well as the isotropic and anisotropic layer stiffness behavior of the granular base/subbase materials, was used to predict the field measured responses needed for the FE model validation. The cross-anisotropic modulus model parameters for the resilient moduli in vertical and horizontal directions (M_R^v and M_R^h) and shear modulus (G_R) were characterized from laboratory testing in accordance with the approach by Tutumluer and Thompson (1997a, 1997b). Figure 63 shows for the unreinforced B1 test section (76-mm asphalt concrete underlain by 305-mm UAB) comparisons of the measured pavement responses and the initial response predictions as a result of the different magnitudes of dual wheel loading with 689 kPa tire pressure. The cross-anisotropic base characterization gave much better predictions for the vertical LVDT displacements on top of subgrade and the radial LVDT displacements at the bottom of base course (see Figure 64).

In the design of future full-scale pavement test studies, the performance prediction parameters, such as deflection basin shape and magnitude, degraded stiffnesses, rutting in the base course and subgrade, and other manifestations of distress, should be monitored during accelerated testing for developing transfer functions (or distress models) to adequately relate pavement response variables to pavement performance. Masad et al. (2006) nicely pointed out that the performance models originally developed using isotropic material properties would require refinement and calibration for use with anisotropic material properties. Such a refinement would lead to smaller

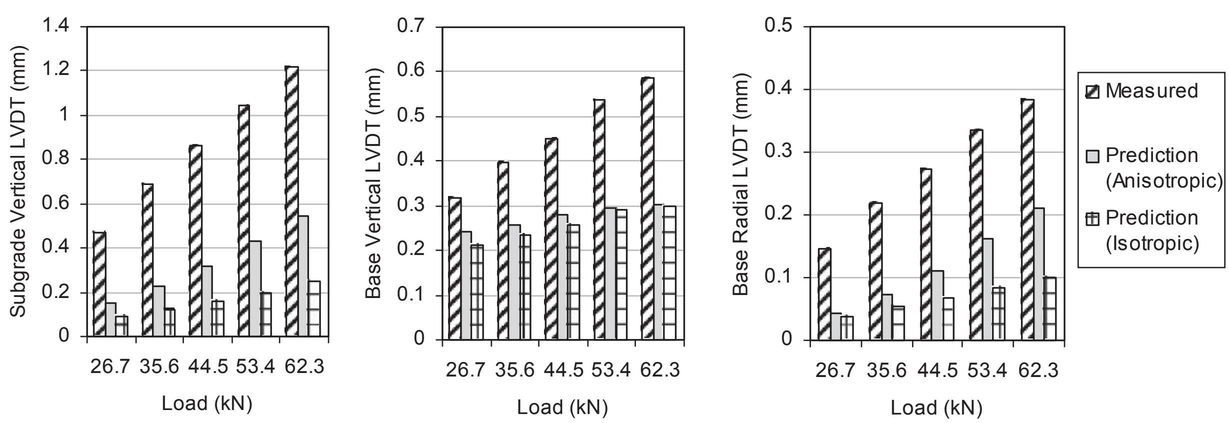


FIGURE 63 Comparisons of measured and initial pavement response predictions from B1 unreinforced section (tire pressure of 689 kPa) (after Kwon et al. 2008).

shift factors and calibration coefficients owing to the improved match between the actual anisotropic material behavior and the response mode. The periodic monitoring and testing of pavement test sections also should help incorporate anisotropy and material nonlinearity in backcalculation methods to better account for the behavior of flexible pavements with unbound granular layers and estimate remaining life and performance.

Anisotropy as Aggregate Quality Indicator

Tutumluer and Seyhan (2000) evaluated the anisotropic resilient properties of aggregate systems through advanced laboratory tests and reported that the aggregate matrix showed significant softening behavior as the percentage of P_{200} fines (materials smaller than 75 µm or passing the No. 200 sieve) exceeded 12%. Research by Kim et al. (2005) has shown that aggregate type, gradation, and particle shape, texture and angularity significantly affect the level of anisotropy: that is, the ratio of horizontal to vertical aggregate layer moduli $n = M_R^h/M_R^v$. The anisotropy levels of aggregate base (the horizontal and shear moduli model parameters) could be approximated from regression analyses based on the model parameters of the vertical resilient moduli (K_4 to K_6) and some fitting parameters developed for aggregate physical properties, such as grain size distribution, form, angularity, and surface texture. Typically, higher values of moduli and modulus ratios were obtained when aggregate particles were well-graded, less elongated, and more angular with rougher surface texture. Later, Kim et al. (2007) successfully used a similar anisotropy level assessment technique to estimate in situ resilient modulus properties of sandy subgrade soils from FWD test results based on gradation properties, granular-base-to-asphalt-concrete-pavement thickness ratios, and the applied surface loading.

An extension of the approach by Kim et al. (2005) was adopted recently by Ashtiani et al. (2007), who evaluated the impact of increasing fines content on the performance of unbound (unstabilized) and lightly cement-stabilized aggregate systems. It was found that with the proper design of fines content, cement content, and moisture, the performance of the stabilized systems with high fines content could perform in a manner equivalent to or even better than could the systems with standard fines content. Ashtiani et al. (2007) also reported that by enhancing the resilient properties (increase in stiffness and decrease in anisotropy), compressive strength and permanent deformation properties could be improved in lightly cemented aggregate systems.

Recently, Ashtiani and Little (2009) developed a comprehensive aggregate database to identify the contribution level of different aggregate material types and properties as well as base course features to the directional dependency of nonlinear, stress-dependent M_R properties. Figure 64 demonstrates the impact of particle texture and aggregate angularity on the level of anisotropy characterized by vertical-to-horizontal modular ratios (i.e., E_x/E_y). Aggregate systems containing particles with rougher texture and more crushed surfaces (more angular) result in much higher E_x/E_y (= E_h/E_v) ratios to more efficiently distribute load with greater aggregate interlock and friction in the unbound aggregate layer and thus to become less prone to plastic deformation under traffic (Ashtiani and Little 2009). Note that an isotropic system would correspond to a modulus ratio (E_x/E_y) of 1.0.

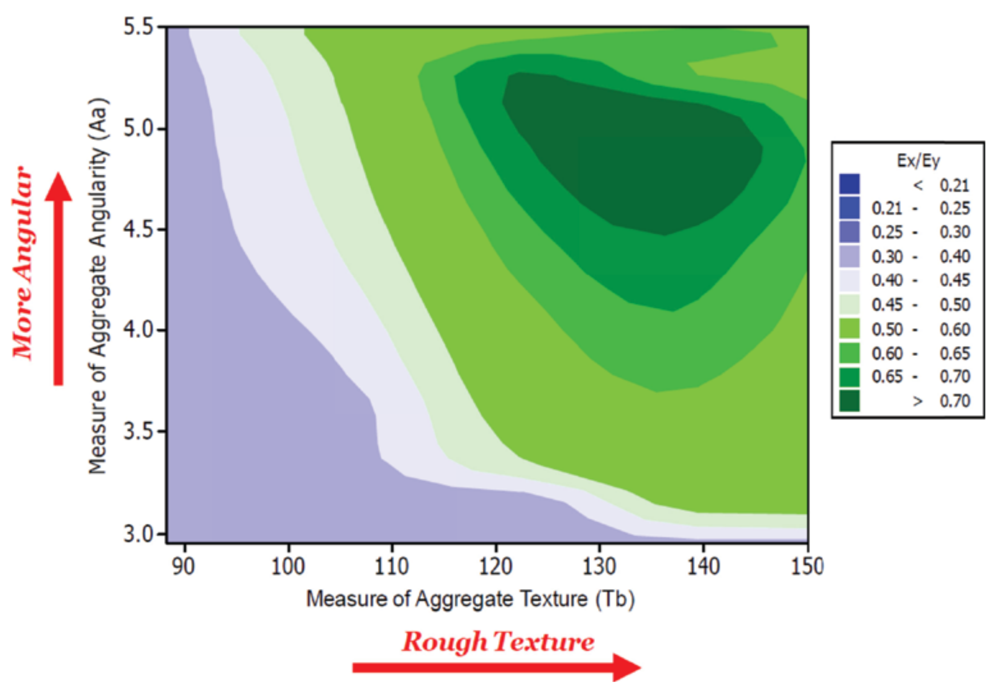


FIGURE 64 Impact of aggregate angularity and texture on anisotropy level assessed using the axial modulus ratio (E_x/E_y) (Ashtiani and Little 2009).

TABLE 6
ANISOTROPY AS AGGREGATE QUALITY INDICATOR AFFECTING PAVEMENT RESPONSE AND PERFORMANCE

	$n = M_R^h / M_R^v$		Bottom AC ε_{AC} ($\mu\varepsilon$)	Top Subgrade ε_{SG} ($\mu\varepsilon$)	Top Subgrade σ_d (kPa)
Anisotropy Decreases ↓	0.15	Pavement Responses Decrease ↓	295	652	25.5
	0.2		284	622	24.1
	0.3		268	565	22.0
Agg. Quality Increases ↓	0.4	Performance Increases ↓	253	518	20.7
	0.5		227	478	18.6

Improving aggregate properties, such as by using well-graded cubical shaped and crushed aggregates with rough surface texture and reducing the amount of fines decreases the level of anisotropy while keeping M_R^v constant. Table 6 illustrates the relationship between aggregate quality and the level of anisotropy affecting pavement response and performance from GT-PAVE FE analyses of a conventional flexible pavement. The results given in Table 6 agree quite well with the known effective practices of the transportation agencies that pay attention to aggregate properties for building long-lasting pavements with deep UABs/subbases (Beatty et al. 2002). Thus, properly accounting for stress sensitivity and modulus anisotropy of unbound aggregate structural layers will be essential to the optimized use of available aggregate resources, building pavements with deep aggregate base/subbase courses, and accurately predicting their expected field performances. Note that the M_R^h and M_R^v values shown in Table 6 are similar in definition to the E_x and E_y values indicated in Figure 65; as for pavement design purposes, resilient modulus values often are used as the layer moduli.

Key Lessons

- Several research studies and field validation projects have highlighted the benefits of considering aggregate anisotropy in pavement design; traditional pavement design methods based on the assumption that the unbound aggregate layer is isotropic may under-design flexible pavements and overestimate the pavement design life.
- For adequately considering the behavior of UAB/subbase pavement layers under loading, the cross-anisotropic properties of such layers are best incorporated into pavement design. A simplified approach (see Simplified Procedure for Determining Anisotropic Model Parameters) is available for agencies to incorporate such cross-anisotropy of unbound aggregate layers into pavement design without the need to conduct advanced triaxial tests.

CONSIDERATION OF DRAINAGE IN UNBOUND AGGREGATE LAYER DESIGN

As a factor well-known for significantly affecting strength and modulus characteristics of pavement base/subbase materials and subgrade soils, excessive field moisture content is among the primary causes of pavement deterioration and premature failure. Among the primary challenges associated with maintaining pavement structures in adequate serviceable conditions, one aspect is to adequately cope with increasing moisture contents and minimize the effects of seasonal spring thaw by removing all transient water from the pavement structure as rapidly as possible before excessive accumulation (Birgisson and Roberson 2000). Such requirements imply the following two objectives of pavement foundation design: (1) proper selection/utilization of base/subbase materials retaining low average moisture contents and/or moisture susceptibility, and (2) efficient subsurface drainage design. Many field forensic studies have demonstrated that good drainage and subsurface layer grading are essential for long-term satisfactory performance (Khazanovich et al. 1998; Chen and Lin 2010). In this section, previous findings on the mechanisms of moisture-related deterioration and corresponding mitigation measures are reviewed.

NCHRP 4-23 project listed physical and mechanical properties of base/subbase materials in relation to major concrete pavement distresses for performance evaluation (Saeed et al. 2001). In concrete pavements, subbase and subgrade are required to be stable under the application of traffic and environmental loadings; however, mechanical properties of unbound granular materials, such as resilient modulus (M_R), shear strength, and rutting resistance, have long been found to be affected significantly by moisture content of the materials. Saarenketo and Scullion (1996) reported the dielectric value and electrical conductivity related to both the strength and deformation properties and frost-susceptibility of base course aggregates. The substantial effect of saturation hysteresis on base strength was also among important findings. Toros and Hiltunen (2008) characterized time-dependent changes in strength and stiffness of Florida base materials. These are changes that can be explained with unsaturated soil mechanics framework, such as changes in moisture or moisture distribution as a result of changes in internal pore pressure to influence the effective confining pressure constraining the material. Such changes in moisture content anticipated in the base/subbase during the service life of a pavement structure should be properly accounted for in a pavement design process.

Moisture-Related Deterioration

Base/Subbase Erosion

Erosion mechanism base/subbase layers below concrete pavements should be properly designed to provide a stable construction platform, uniform slab support, and erosion resistance. Different base/subbase materials below concrete pavements have been applied over the years, including dense- and open-graded unbound materials, cement- and asphalt-treated bases, lean concrete bases, and the combinations of such layers (sometimes with asphalt concrete interlayer for debonding). The degree of erosion resistance, called erodibility, depends on the type and nature of the base/subbase materials. In general, base/subbase erodibility increases with higher fines content and lower admixture stabilizer content (cement or asphalt).

Figure 65 illustrates the subbase erosion process with which the pavement deteriorates (Jung et al. 2009). Any water infiltrated into pavement structure through joints/cracks becomes trapped if it cannot flow out on a timely basis. Water then is injected as pressurized water into the pores between granular particles under dynamic moving wheel loads, causing the migration and accumulation of fine materials at the top surface of base/subbase layers. As this process evolves, distresses such as pumping and joint faulting could be initiated and accelerated in combination with other mutual effects responsible for pavement deterioration [for example, concrete slab curling and warping resulting from temperature and moisture variations and decreasing load transfer efficiency (LTE) along joints]. Both field and laboratory observations have indicated that the combination of traffic loading, trapped subsurface water, and erosion susceptible base/subbase materials is the primary cause of base/subbase erosion and subsequent joint faulting.

According to Jung and Zollinger (2011), shear stress along the slab-subbase interface resulting from a moving load is the main contributor for the subbase erosion process. As shown in Figure 65, fines generated by such shear stress along the interface can be dislocated by pumping and then deposited into the void beneath concrete slab created by pumping, which eventually results in further loss of joint stiffness and joint faulting. As joint stiffness and the interface bonding keep decreasing, the magnitude of the interface shear stress increases and the erosion process becomes exacerbated. Such hydraulic erosion was modeled by Jung and Zollinger (2011) for predicting erosion depth and joint faulting as a function of the number of wheel load applications.

Evaluation of Base/Subbase Erodibility

The need for a non-erodible subbase to maintain uniform support under concrete pavements and thus ensure satisfactory service performance has long been recognized. The erosion resistance of materials beneath a concrete slab is an important performance-related property. As summarized in Table 7 by Jung et al. (2009), despite the existence of several empirical and subjective test methods, no well-accepted laboratory test methods are available for characterizing the erosion resistance of base/subbase materials using a mechanistic approach.

Tables 8 and 9 summarize existing erosion models and design procedures that take erosion into account (Jung

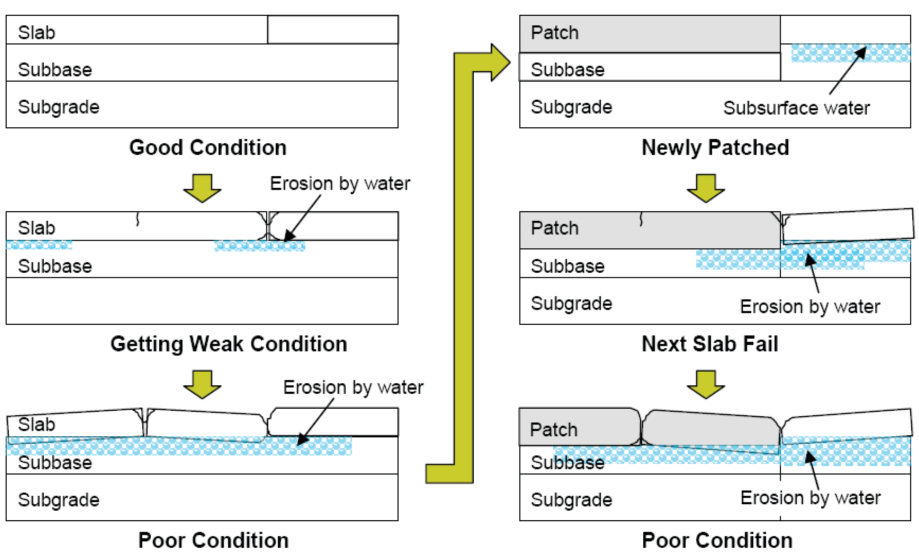


FIGURE 65 Subbase erosion and pavement deterioration processes (Jung et al. 2009).

TABLE 7
SUMMARY OF EXISTING EROSION TEST METHODS

Test Method	Features	Strengths	Weaknesses
Rotational shear device	Stabilized test samples are eroded by application of hydraulic shear stress. The critical shear stress is recommended as an index of erosion resistance.	Easy and precise to control shear stress	No consideration of crushing or compressive failure. Overestimation of weight loss by coarse aggregates loss.
Jetting device	Pressurized water at an angle to the upper surface of unstabilized samples generating weight loss over time.	Easy to test	Shear stress is not uniform and inaccurate. Overestimation of weight loss by coarse aggregates loss.
Brush test device	Rotational brush abrasions generate fines. An erosion index, IE, is defined as the ratio of the weight loss to that of a reference material.	Easy to test; consider durability of wet and dry cycle; relative erodibility of each material is defined using an erosion index, IE	Long test time and overestimation of weight loss by coarse aggregate loss.
Rolling wheel erosion test device	Wheel movements over a friction pad on sample induce erosion. Measure average erosion depth after 5,000 wheel load applications.	Simulate field conditions for flexible pavement; no coarse aggregate loss	Voiding of the subbase under concrete slab cannot be considered. Sample saw-cut can damage sample surface.

Source: Jung et al. (2009).

TABLE 8
SUMMARY OF EXISTING EROSION PREDICTION MODELS

Erosion Model	Features	Strengths	Weaknesses
Rauhut	Empirical model using COPES data	Include many erosion related factors	Rough categories for each material factor
Markow	Empirical model using AASHTO data: traffic, slab thickness, drainage	Consider more detail drainage condition	Subbase material properties are ignored
Larralde	Empirical model using AASHTO data: traffic, slab thickness	Normalized pumping index to eliminate the effect of slab length and reinforcement	No consideration about many erosion-related factors
Van Wijk	Fusion model of Rauhut and Larralde models with more field data	Consider various erosion-related factors and four types of climates	Rough categories for each material factor
Portland Cement Association	Mechanistic-empirical model using AASHO data	Significant advancement in the mechanistic analysis	Application of the model is limited to subbase types used in AASHO test
Jeong and Zollinger	Mechanistic model using theoretical hydraulic shear stress model	Predict erosion depth based on feasible mechanistic equations	Calibration required through lab tests and field performances

Source: Jung et al. (2009).

TABLE 9
SUMMARY OF CURRENT SUBBASE DESIGN PRACTICES AND GUIDELINES

Design Guide	Features	Strengths	Weaknesses
Portland Cement Association	Provide erosion factor as a function of the slab thickness, composite k value, dowel, and shoulder type	Consider erosion analysis in design procedures as the most critical distress in rigid pavement performance	Proposed composite design *k*-values for treated bases are overestimated and need discrimination for different stabilization levels
1993 AASHTO	Composite modulus of subgrade reaction considers the loss of support (LS) caused by the foundation erosion	Accounting structural degradation of support caused by erosion using LS factor	*k*-value obtained from the chart is overestimated and LS is insensitive to various stabilized materials
NCHRP 1-37A MEPDG	Classified erodibility of subbase materials is utilized in jointed concrete pavement faulting prediction models as well as erosion width estimation of continuously reinforced concrete pavement	Employed the erodibility class based on the type and level of stabilization along with compressive strength	Erodibility class is determined based on dry brush test results and strength even though erosion occurs mostly under saturated condition
Texas Department of Transportation	Select one from two types of stabilized subbase and require minimum 7-day compressive strength	Historical performances and erosion resistance are demonstrated as good	Costly excessive design regardless of subgrade and environmental condition

Source: Jung et al. (2009).

et al. 2009). The first concrete pavement design procedure addressing erosion may be the Portland Cement Association procedure, which relates subbase erosion with pavement deflection (at the slab corner) owing to axle loading. The equations for estimating percent erosion damage, together with the erosion criterion, were developed primarily from the AASHO Road Test results. Note that only one highly erodible subbase type was used during the AASHO Road Test, so extending the application of these equations to other different subbase types for mechanistic analysis of pavement support conditions may be problematic. The AASHTO 1993 Guide relates foundation erosion to the potential loss of support (*LS*), which is numerically categorized into four different contact conditions (i.e., *LS* = 0, 1, 2, and 3). Each contact condition is associated with an effective reduction of the modulus of subgrade reaction in the thickness design procedure. An *LS* value of 0 represents the best contact condition when the concrete slab and the foundation are in full contact, whereas the value of 3 represents the worst case when the concrete slab is completely separated from the foundation. The major limitation of this method, as pointed out by Jung et al. (2010), is that it is too subjective to be sensitive to material factors causing erosion, which may lead to inconsistency and limiting applicability. The MEPDG recommends five different erodibility classes (from 1 to 5) for assessing the erosion potential of treated and untreated base materials on the basis of material type and stabilizer percent. The erodibility factor of base/subbase materials is incorporated as an input parameter for modeling maximum and minimum transverse joint faulting. Note that none of those widely used analysis and design procedures explicitly include base/subbase erosion in a mechanistic approach.

After reviewing previous erosion test methods and models, Jung et al. (2010) proposed a new test configuration that uses a rapid triaxial test and a Hamburg wheel-tracking device for evaluating the erodibility of various base/subbase materials under dry and wet conditions, respectively. Figure 66 shows the schematic diagrams of those test setups. During the rapid triaxial test, shear stresses of varying magnitudes result from different combinations of deviator, and confining pressures are applied on the interface between concrete and subbase samples. The erodibility of subbase materials is measured from the percent weight loss caused by shear-induced interfacial abrasion. By integrating this new erosion test scheme with the theoretical hydraulic shear stress model (Jung and Zollinger 2011), they also developed a new laboratory-based M-E model for faulting in jointed concrete pavement and calibrated it using lab test results and LTPP field performance data.

Key Lessons

- Several pavement distresses may result from the presence of excessive moisture in unbound aggregate layers.
- Base/subbase erosion leads to pavement distress in the original PCC slab and in (repair) patches installed on the original slab. If initial pavement distresses indicate the presence of pumping, base/subbase repairs are required to eliminate chances of the (repair) patch failing by the same mechanism as the original slab.

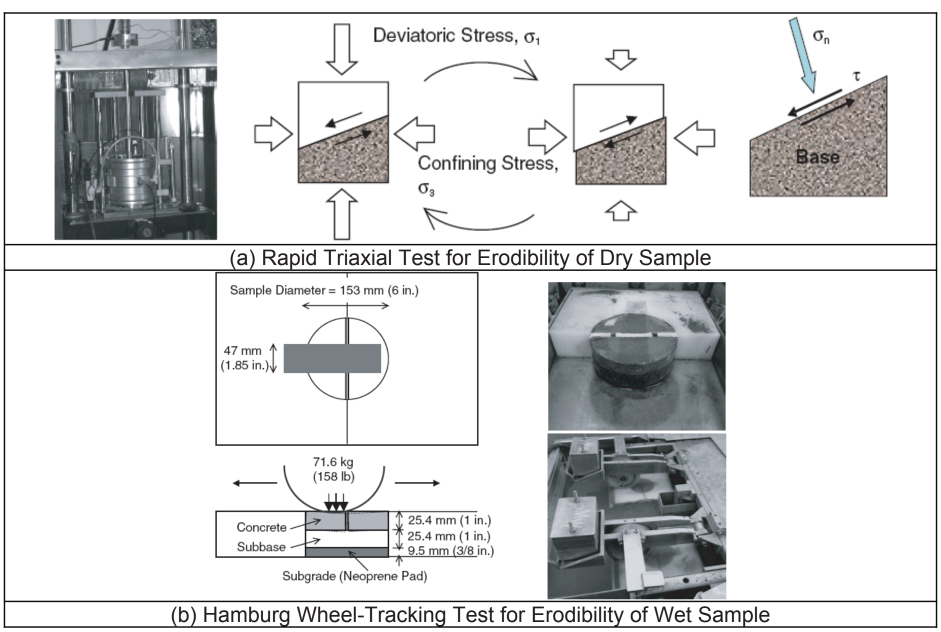


FIGURE 66 Schematic diagrams of the new erodibility tests by Jung and Zollinger (2011).

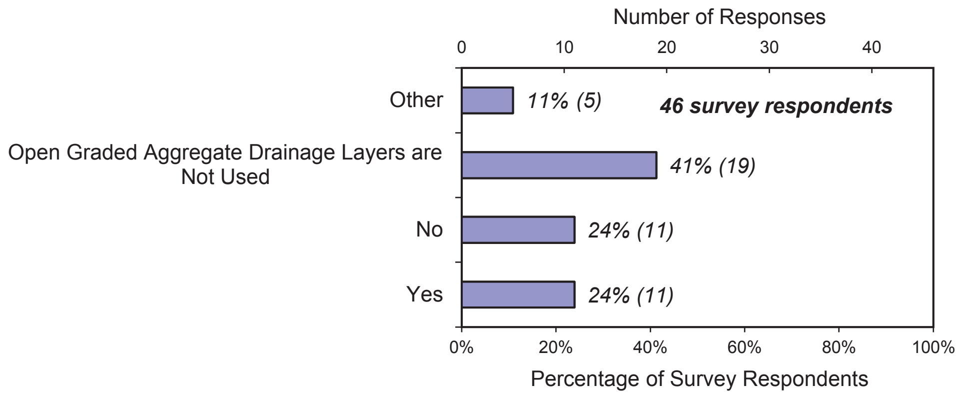


FIGURE 67 Consideration of structural contribution of open-graded aggregate drainage layers in pavement thickness design.

- Rapid removal of excessive moisture from unbound aggregate layers can be achieved through selection of aggregate materials with low water-retaining tendencies and design of suitable subsurface drainage systems.
- Aggregate materials are best tested for erosion potential or "erodibility" before being used in unbound base/subbase layers.

STATE OF THE PRACTICE REGARDING THE CONSIDERATION OF DRAINAGE AND CLIMATIC EFFECTS ON UNBOUND AGGREGATE BASE/SUBBASE LAYERS

The survey of U.S. state and Canadian provincial transportation agencies included questions regarding the state of the practice as far as the effects of climatic conditions and drainage on UAB and subbase layer design is concerned.

Only 11 of 46 respondents indicated that the structural contribution of open-graded aggregate drainage layers is considered during the pavement design procedure (see Figure 67). Eleven agencies reported not considering the structural contribution of drainage layers, whereas 19 agencies do not use open-graded aggregate drainage layers in pavement systems.

Twenty-seven of 46 respondents do not consider the effects of climate changes on the performance of unbound aggregate pavement layers (see Figure 68). Only nine agencies consider the effects of climatic conditions on unbound aggregate layer performance, whereas 10 respondents were not certain about prevalent agency practices. Of the nine agencies that do consider the effects of climate changes, seven do so by adjusting the aggregate layer resilient modulus value. Four agencies reported changing the layer structural coefficients, whereas only one agency (Virginia Department of Transportation) reported adjusting the aggregate layer shear strength, as shown in Figure 69. Note that Virginia DOT also reported adjustments to layer resilient modulus values and structural coefficients as common practice while considering the effects of climate change.

Only 19 agencies currently specify different gradations for drainable and low-permeability applications of unbound aggregate layers, and 24 agencies reported no such practice

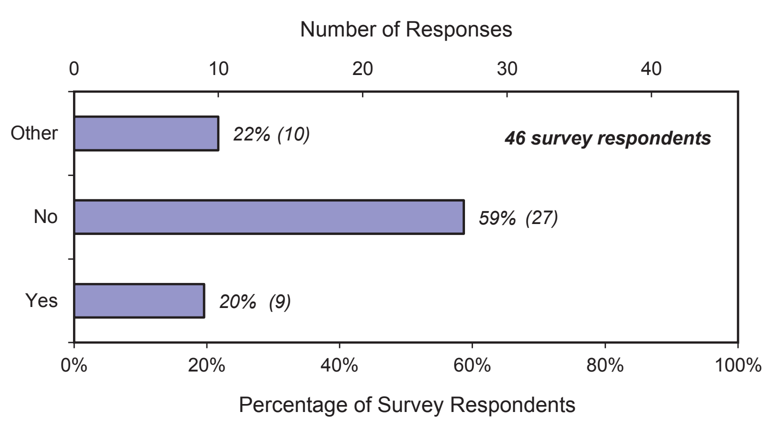


FIGURE 68 Agency response to whether the effects of climatic changes on unbound aggregate layer performance are considered.

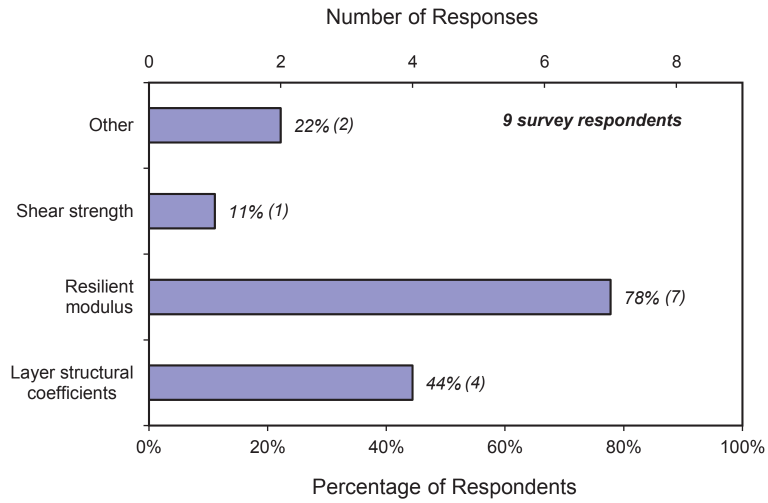


FIGURE 69 Agency response to unbound aggregate layer properties that are adjusted to account for climatic effects.

(see Figure 70). As shown in Figure 71, 23 agencies do not consider drainage to be one of the primary functions of flexible pavement UAB/subbase layers. Of the ones that do consider drainage to be one of the primary functions, 17 facilitate the drainability of such layers by limiting the maximum allowable percent fines (material passing No. 200 sieve). Six agencies reported adjusting the material gradation to construct a more open-graded layer.

As far as measuring the effectiveness of open-graded aggregate drainage layers is concerned, eight agencies use laboratory tests to measure the permeability of aggregate samples. Nine agencies use empirical correlations to estimate layer permeability from aggregate physical properties. Only two agencies (Maryland and Utah) reported using in situ permeability measurements, as shown in Figure 72.

Twelve agencies use a geosynthetic filter layer to prevent OGDLs from clogging. Five agencies construct the filter layers using open-graded aggregates, whereas nine agencies do not construct any extra layer for filtration purposes (see Figure 73). Finally, as shown in Figure 74, construction subsurface drainage systems, such as edge drains, are frequently constructed by 12 agencies. Twenty-one others construct such subsurface drainage systems only for specific projects when required by the design.

Key Lessons

- Only nine of 46 responding agencies consider the effects of climatic conditions on unbound aggregate pavement layer performance.
- Agencies that do consider the effects of climatic conditions on unbound aggregate layer performance do so by adjusting the resilient modulus or layer structural coefficient values.

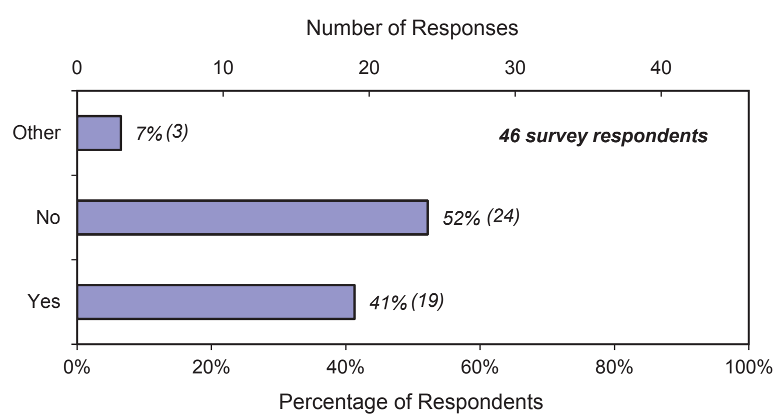


FIGURE 70 Agency response to whether different gradations are specified for unbound aggregate applications targeting drainable versus Low-permeability layers.

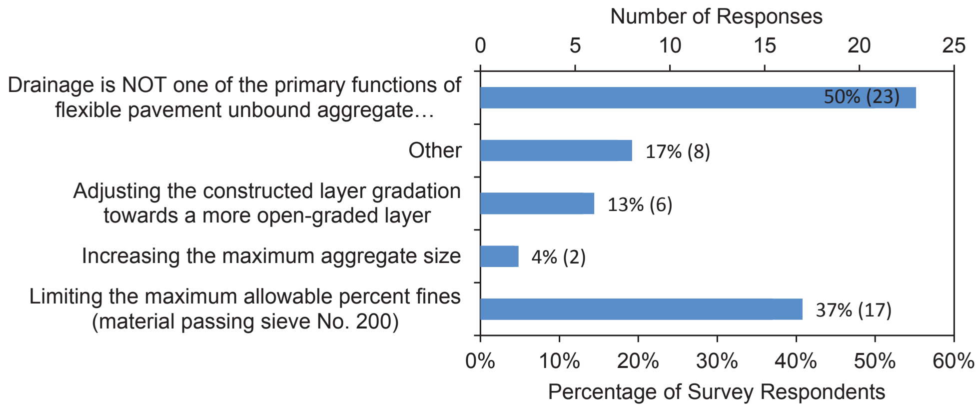


FIGURE 71 Different approaches adopted by agencies to facilitate the drainage of dense-graded base courses.

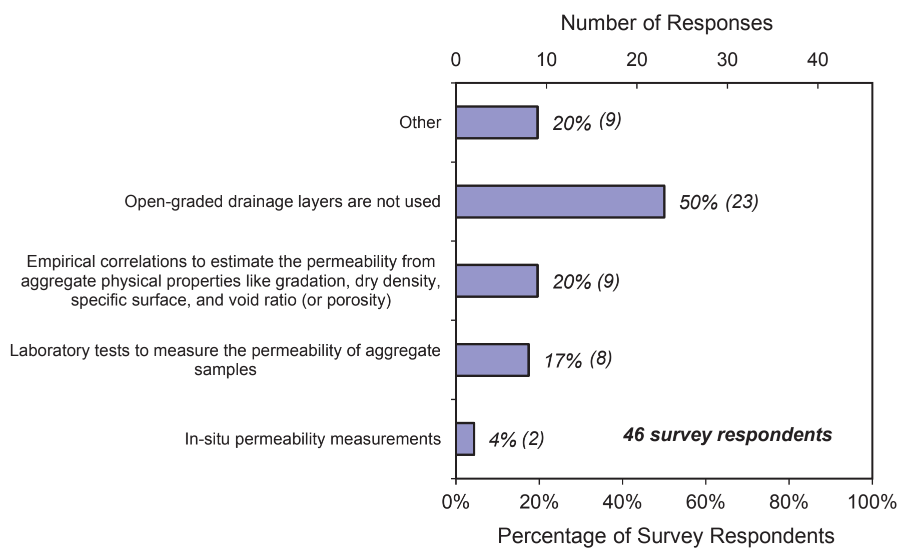


FIGURE 72 Different methods used by agencies to measure the effectiveness of open-graded aggregate drainage layers.

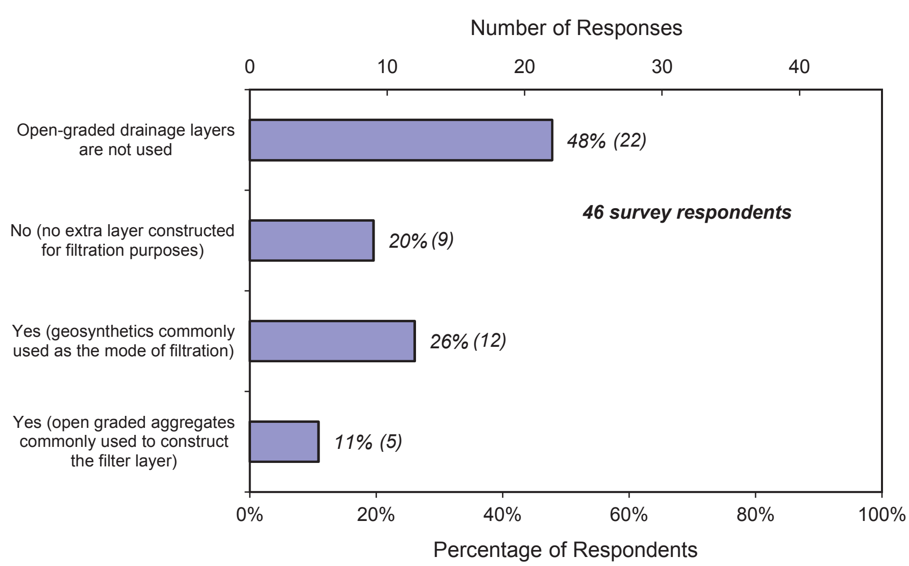


FIGURE 73 Agency response to whether filter layers are used to prevent the clogging of open-graded drainage layers.

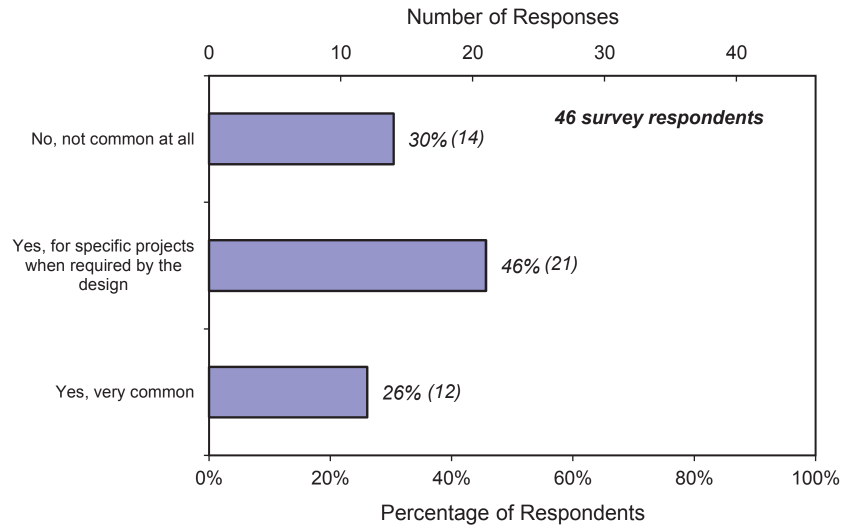


FIGURE 74 Agency response to whether the construction of subsurface drainage systems, such as edge drains, is a common practice.

- Nineteen of 46 responding agencies specify different gradations for permeable versus low-permeability unbound aggregate pavement layers.
- The permeability of drainage layers usually is estimated from laboratory test results or empirical correlations. Only two agencies conduct in situ permeability tests on unbound aggregate drainage layers.

EFFECTS OF AGGREGATE MATERIAL PROPERTIES ON LAYER PERMEABILITY

Permeable Base Designs

The use of permeable bases, open-graded drainage layers (OGDLs) and installation of edge drains are the primary options for restoring drainage efficiency. The hydraulic conductivity *k* (or the coefficient of permeability) is an essential parameter needed for analysis and design of the subsurface drainage systems, along with other engineering properties of materials, such as the grain size distribution (gradation), packing, degree of saturation, and frost susceptibility. Laboratory determination of the coefficient of permeability is very important for permeable base/subbase designs, fill materials, and other drainage layers, and test methods for this purpose are quite mature. Obtaining in situ measurements of permeability for base/subbase layers is desired from the standpoint of adequate engineering design. With the advent of portable gas permeameter test (GPT) devices, such as that introduced by White et al. (2010), this becomes possible and widely used in QC/QA. In cases in which field measurement and laboratory testing of permeability are not possible, empirical approaches can be used with caution to estimate permeability from readily available material properties, such as gradation and moisture and density conditions.

Two main permeable base materials are widely used: stabilized and unbound. For example, in Minnesota, gradation specifications for an unstabilized base and a stabilized base are represented by the following percent passing values (Arika et al. 2009):

Unstabilized Base

- 1 inch: 100%;
- ¾ inch: 65% to 100%;
- ⅜ inch: 35% to 70%;
- 4.75 mm (#4): 20% to 45%;
- 2 mm (#10): 8% to 25%;
- 0.425 mm (#40): 2% to 10%;
- 0.075 mm (#200): 0% to 3%

Stabilized Base

- 1½ inch: 100%;
- 1 inch: 95% to 100;
- ½ inch: 25% to 60%;
- 4.75 mm (#4): 0% to 10%;
- 2.38 mm (#8): 0% to 5%.

With the addition of a stabilizer (asphalt or cement), the gradation of a stabilized granular material becomes much coarser; however, unstabilized materials require considerable amounts of finer-size aggregates to achieve better packing (low voids content) and stability through aggregate interlock. Typical permeability values are 6,800 ft/day for stabilized bases and 1,000 to approximately 3,000 ft/day for unbound granular ones. Note that permeability values as low as 1,000 ft/day are considered acceptable as far as pavement layer drainage requirements are concerned. Table 10 shows the MnDOT guidelines for selecting permeable aggregate base. In addition to maintaining adequate permeability, these layers are required to remain stable during construction and future rehabilitation activities over the design service life.

Hagen and Cochran (1995) described and evaluated in a study (MD-RD-95-28) the drainage characteristics and pavement performance of four drainage systems under jointed concrete pavement: MnDOT standard dense-graded base, two dense-graded base sections incorporating transverse drains placed under the transverse joints, and permeable asphalt-stabilized base reflecting the MnDOT drainable base concept at that time. Moisture sensors were placed in the pavement to assist in evaluating the relative performance of the traditional and new drainage systems and their effects on pavement performance. Longitudinal edge drains were installed in all sections. Drainage flows, percent of rainfall drained, time to drain, base and subgrade moisture content, and pavement and joint durability were the variables studied. The following important observations were reported: (1) although all systems appeared capable of removing drainable water from the pavement base, the permeable asphalt-stabilized base commonly drained the most water within 2 hours after rainfall ended while providing the driest pavement foundation with the least early pavement distress; (2) approximately 40% of the rain infiltrated into the concrete pavement; (3) the open-graded and geocomposite systems removed water most rapidly; (4) spring thaw flows were roughly equivalent to a major rain event; and (5) all rain inflow was reduced temporarily by sealing the longitudinal and transverse joints but resumed after approximately 2 weeks, despite the joint sealants appearing to be intact.

In Illinois, Dhamrait and Schwartz (1979) evaluated four types of subbases (4-in. thick cement-aggregate mixture, 4-in. thick bituminous-aggregate mixture, 8-in. thick lime-stabilized soil mixture, and 4-in. thick granular materials) and three types of subsurface drainage systems, such as shoulder drainage. Pavement behaviors in terms of transverse cracking and deflections were analyzed and correlated with the type of subbase and type of subsurface drainage system. The lime-stabilized soil mixture as subbase was reported to offer the potential for reducing construction costs. The subsurface drainage system with longitudinal underdrains placed at the edge of the stabilized subbase was the most efficient in removing free water from beneath the pavement structure; the system was adopted by the Illinois DOT as the standard treatment for interstate highways. Winkelman (2004) investigated OGDL performance in Illinois. The OGDL consisted of a uniform size aggregate that may be bound together as a lean concrete mixture or low asphalt cement content bituminous mixture. Pavement performance was monitored in terms of FWD measurements, International Roughness Index values, visual distress surveys, and condition rating survey values. Despite the OGDL being more costly than a stabilized base, its use under a continuously reinforced concrete pavement was not recommended according to the findings. This study suggested that a geotextile fabric or dense-graded aggregate filter be used under the OGDL to prevent the intrusion of subgrade fines.

As compared with UABs, asphalt- or cement-treated bases become more expensive solutions and thus less desirable for some roadways, especially for low- to medium-volume ones. In these situations, it is worth exploring if the use of a properly graded unbound aggregate can maintain adequate drainability and structural stability during construction and the expected service lifetime after the roadway is open to traffic.

In Louisiana, Tao and Abu-Farsakh (2008) studied typical permeable base materials for their drainage benefits, including asphalt- and cement-treated aggregates, open-graded aggregates, and dense-graded unbound aggregates. The permeability of unbound aggregate was quantified by its saturated hydraulic conductivity, whereas its structural stability was characterized by the results of various laboratory tests for strength, stiffness, and permanent deformation of the material. A trade-off between structural stability and permeability of unbound aggregates was observed; the increase of

TABLE 10
MINNESOTA DEPARTMENT OF TRANSPORTATION CONCRETE PAVEMENT PERMEABLE AGGREGATE BASE APPLICATION GUIDELINES

Subgrade Soil	Plastic / Non-Granular				Granular (D)			
Traffic Level	VH	H	M	L	VH		M	L
Interstate	R	R	NA	NA	R	R/AR	NA	NA
Non-Interstate	R	R	R	AR	R	R/AR	NR	NR

Legend:	Traffic Level:	35-yr Design Lane CESALs
AR = As Recommended (A)	VH = (Very High)	> 30 million
NA = Not Applicable (B)	H = (High)	9 – 30 million
NR = Not Recommended	M = (Medium)	3 - million
R = Recommended	L = (Low)	< 3 million
R/AR = (C)		

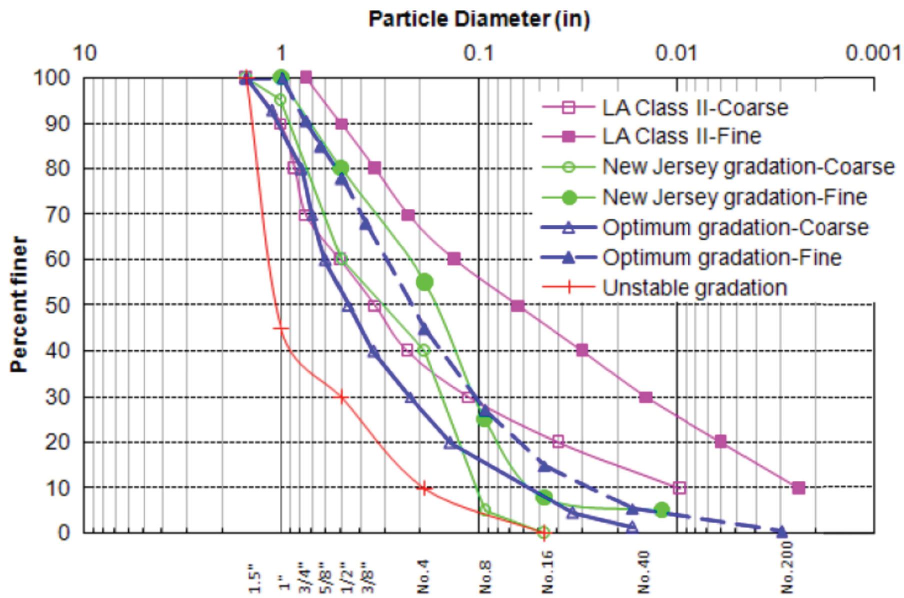


FIGURE 75 Particle size distributions of aggregate types tested by Tao and Abu-Farsakh (2008).

permeability was often at the cost of structural stability or vice versa. Therefore, the criteria for selecting such an optimum gradation were: (1) an adequate permeability to drain the infiltrated-water from the pavement as rapidly as possible; and (2) a sufficient structural stability to support the traffic loading. Laboratory tests were conducted on a Mexican limestone (commonly used in Louisiana highways) with different gradations, including constant-head permeability, CBR, Dynamic Cone Penetrometer (DCP), tube suction test (TST), monotonic load triaxial tests, and repeated load triaxial tests. The gradations under investigation included coarse and fine branches of Louisiana class II gradation, New Jersey gradation medium, and an optimum gradation (fine and coarse branches). As a result, Tao and Abu-Farsakh (2008) determined a proper/optimum gradation for permeable base materials. For a detailed discussion of the proposed proper/optimum gradation, the reader is directed to the original publication (Tao and Abu-Farsakh, 2008). Figure 75 shows the gradation curves of different aggregate materials tested by Tao and Abu-Farsakh (2008).

Key Lessons

- Permeability values as low as 1,000 ft/day are considered acceptable as far as pavement layer drainage requirements are concerned.
- Stable open-graded or gap-graded (a gradation with some intermediate-size particles missing) aggregates with low fines contents (P_{200}) are best selected for use in unbound aggregate drainage layers.

CONSIDERATION OF CLIMATIC CONDITIONS IN UNBOUND AGGREGATE BASE DESIGN

The life cycles of most pavements are significantly shorter than the time span over which climatic effects will have a statistically significant effect on pavement performance (Dawson 2010). However, the effect of climate change on UAB/subbase performance can be manifested primarily through changes in moisture content, effect of freeze-thaw cycles, and depth of frost penetration. As discussed in chapter two, moisture often has been identified by researchers and practitioners as one of the most important factors affecting unbound aggregate layer performance. Permanent deformation is more likely to occur in an unbound aggregate layer during wet spring months when the modulus and strength properties are greatly reduced, especially in the northern climates with wet freeze and thaw conditions. Unbound pavement layers are most likely to reach equilibrium moisture contents, often on the wet side of compacted optimum moisture conditions, and this can drastically affect the long-term modulus and permanent deformation behavior.

The seasonal variation in unbound pavement material moduli is widely recognized as contributing to decreased load-carrying capacity and pavement failure. Factors influencing layer moduli are stress state, moisture, suction, density, and material characteristics. Climate factors such as precipitation and temperature contribute to seasonal variation in layer moduli. These seasonal variations are mainly the result of variations in moisture/suction. Depending on the magnitude of the load or applied stress state in relation to the strength, modulus and permanent deformation properties vary considerably with moisture/suction and temperature, which in turn depend on the weather conditions.

Through investigation of several pavements in Washington State, Newcomb et al. (1989) observed that seasonal variations in the moduli of the subgrade materials were much less significant than those observed in granular base materials. Uhlmeyer et al. (1996) reported that the effect of seasonal variations on base layer performance were greater than those for the subgrade. Moreover, they observed that the seasonal effect on unbound aggregate layer performance was reduced significantly when the stress sensitivity of unbound aggregate materials was considered, rather than treating the layers as linearly elastic.

Kolisoja et al. (2002) conducted cyclic-loading triaxial tests on base course aggregates to simulate seasonal conditions of dryness, moisture, and the period after a freeze-thaw cycle. From the test results they reported that even though the permanent deformation behavior of aggregates were significantly affected by freeze-thaw cycles, no significant changes in the resilient modulus values were observed even during the spring thaw phase.

Werkmeister et al. (2003) observed that even a 1% increase in moisture had a significant effect on the permanent deformation behavior of unbound aggregates. Although the increase in moisture content did not result in significant changes in the resilient modulus values (and thus the stress levels), it was reflected through drastic changes in the permanent deformation behavior.

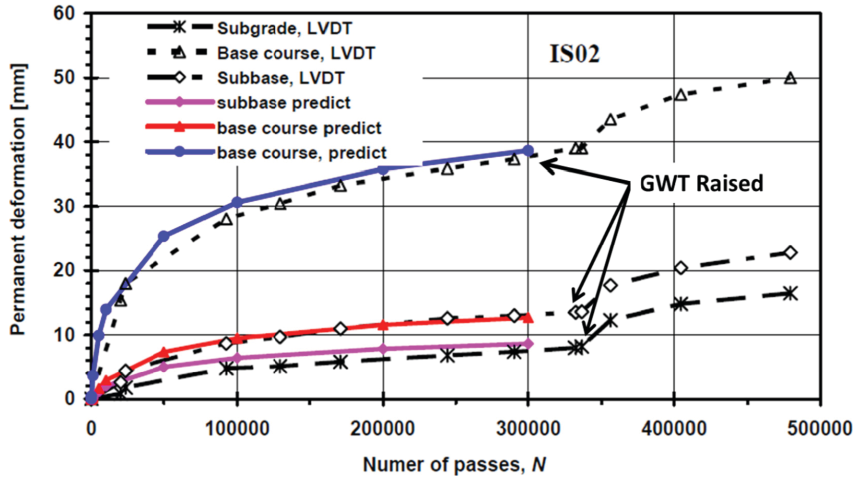
Carrera et al. (2009) listed the following factors as significantly affecting the moisture sensitivity of unbound aggregates: (1) compaction properties, (2) amount of degradation, (3) grain size composition, and (4) quality of P_{200} fines (plasticity and swelling index). Mishra (2012) studied the effects of material quality on aggregate behavior using aggregate specimens containing different amounts of nonplastic and plastic P_{200} fines (material finer than 0.075 mm) compacted to different moisture-density conditions. Conducting laboratory tests to characterize the shear strength, resilient modulus, and permanent deformation behavior, Mishra concluded that the effect of moisture content on aggregate behavior was particularly severe for specimens containing high amounts of plastic fines. Moreover, the quality of fines (plastic or nonplastic) was significant only for specimens with intermediate to high fines contents (8% or higher for crushed aggregates, 6% or higher for uncrushed gravel).

A shallow depth to GWT decreases suction and critically affects the long-term modulus and permanent deformation behavior of aggregate base/subbase, especially in regions with a moisture surplus, as measured by the Thorntwaithe Moisture Index when equilibrium moisture contents are on the wet side of compacted optimum conditions (Zapata and Houston 2008). Figure 76a shows increased permanent deformations in all of the subgrade, subbase, and base layers during accelerated pavement testing at 330,000 cycles when the water table was raised to 30 cm below the top of sand subgrade (Erlingsson and Ingason 2004). Such an effect of wetting from the water table up is depicted in Figure 76b, where the initial moisture contents are indicated at the compacted optimum moisture condition. This increase in moisture content in excess of initial compaction value, primarily as a result of capillary rise from the water table, was indicated to be more critical in the long term than was the seasonal variation in layer moduli (AASHTO 2004, Appendix DD).

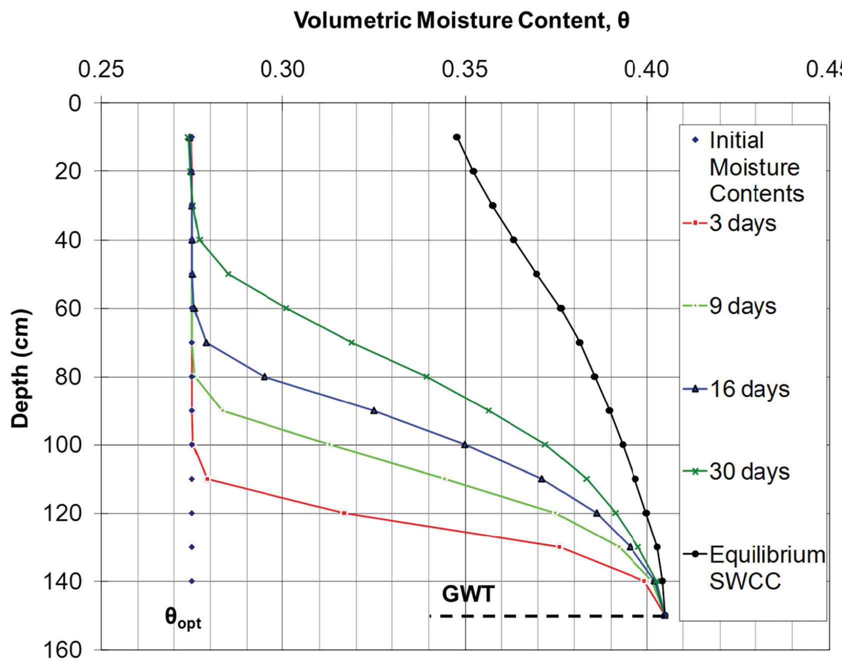
Approaches for predicting the seasonal variation in pavement layer moduli have evolved from models that rely on regional adjustment factors that do not directly address seasonal variations in pavement structure to climate models that relate the changes in modulus to key factors affecting those changes, such as suction (Larson and Dempsey 1997). Explicit consideration of seasonal variation came with the introduction of mechanistically based design methods (Richter 2006).

The MEPDG approach produces a design section that includes required thicknesses and elastic moduli for UAB and subbase for flexible and rigid pavements. The resilient modulus (M_R) of the unbound layer materials used in MEPDG may be specified by means of stress-dependent k_i parameters determined from lab testing (Level 1 MEPDG) or as a single average value determined per lab testing/field nondestructive FWD testing (Level 1 MEPDG), through correlation (Level 2 MEPDG) or estimated with typical values (Level 3 MEPDG). The parameters required to estimate M_R for Level 1 MEPDG by means of laboratory testing are derived from samples compacted at optimum moisture and maximum dry unit weight (standard or modified Proctor). For Level 2 MEPDG, M_R is estimated by means of correlation from laboratory measured parameters such as CBR, Hveem stabilometer R-value, and so forth. For rehabilitation Level 1 MEPDG, M_R is estimated through FWD backcalculation. Through the Enhanced Integrated Climatic Model (EICM), MEPDG seasonally adjusts the subgrade and unbound layer moduli when performing fatigue and permanent deformation analyses. The EICM provides M_R seasonal adjustment through suction model parameters and soil water characteristic curves (SWCCs).

The EICM used in the MEPDG takes into account unsaturated soil mechanics concepts through the climatic-materials-structure and two-dimensional-drainage-infiltration models to calculate coupled heat-moisture flows in pavement structures and predict pavement temperature (AASHTO 2004). The model evaluates the expected changes in moisture condition from the initial or reference condition (gener-



(a)



(b)

FIGURE 76 Effect of shallow groundwater table on permanent deformation accumulation (Erlingsson and Ingason 2004).

ally optimum moisture condition and maximum dry density) as the unbound materials reach equilibrium moisture condition. Seasonal variation in modulus is determined by (1) computing the environmental effects such as layer moisture condition, (2) translating the computed layer moisture into suction through the Fredlund and Xing (1994) SWCC, and (3) predicting a seasonal modulus value from a modulus-suction relationship. The model also evaluates the seasonal changes in moisture condition and consequently the changes in resilient modulus, M_R. This often is done on a biweekly basis for flexible pavements. The model also computes moisture and temperature in the middle of sublayers (established as finite difference node points in the EICM), calculates the effects of freezing, thawing, and recovery on M_R, and uses the new M_R values, corrected for environmental conditions, for calculation of critical pavement response parameters and damage at various points within the pavement system. The effects of varying moisture, freezing, thawing, and recovery on M_R are reflected in the calculation of critical pavement responses and in the damage accumulation within the pavement system (AASHTO 2004, Appendix GG). However, a major concern exists in the way permanent deformation damage is computed using the unbound base/subbase rutting model adopted; the individual rutting amounts in the UAB/subbase layers are computed by incorporating only the changes in moisture content and M_R but no applied stress state in relation to the strength properties. Furthermore, the EICM does not permit the use of models other than that of Fredlund and Xing (1994) to predict SWCC.

The state of the practice in the United States regarding M-E design, field measurement of modulus, and its connection to design modulus was summarized in *NCHRP Synthesis 382* (Puppala 2008). Of the 41 states that responded to *NCHRP Synthesis 382,* 24 used the 1993 AASHTO Design Guide, seven states used the 1972 AASHTO Design Guide, five states (including California and Minnesota) used internally developed mechanistic procedures, four states used internally developed empirical procedures, and one state used the 2002 AASHTO MEPDG. Regarding the use of subgrade and aggregate base design moduli, 22 of the 41 states used M_R in pavement design. Fourteen states determined M_R through correlation to CBR, *R*-value, and so forth, and nine directly measured M_R in the laboratory. Twenty states used FWD-based backcalculation of subgrade and sometimes base modulus for design of rehabilitation projects. In addition, 12 states used FWD results to determine layer coefficients for the 1993 Design Guide. Twenty-two states took the seasonal variation of modulus into account during pavement design in a variety of ways. The majority of states, including California, did not take seasonal variation into consideration. Arkansas chose the lowest modulus value from saturated lab testing, and Minnesota used internally developed charts.

In the current synthesis study, 30 of 46 responding agencies indicated that climatic effects were a major concern as far as pavement subgrade performance was concerned (see Figure 77). Upon further investigation, a change in subgrade soil properties resulting from seasonal fluctuations was identified as the primary concern. In addition, nearly 79% of the respondents indicated that the presence of fine-grained soils in areas susceptible to upward movement of the GWT was responsible for adverse climatic effects on pavement performance.

Thirty-nine of 46 responding agencies do not conduct any testing to evaluate the aggregate materials selected for use in granular base/subbase applications for effects of adverse climatic conditions. Twenty-seven of 46 respondents indicated that effects of climatic changes on unbound aggregate layer performance were not considered in the pavement design procedure. Ten other agencies indicated that the approach adopted by the pavement design procedure to incorporate the effects of climatic changes on unbound aggregate layer performance was not clearly defined. Of the nine agencies accounting for the effect of climatic conditions on unbound aggregate layer performance, four adjusted the layer structural coefficients, seven modified the resilient modulus of unbound aggregate layers under different climatic conditions, and one adjusted the shear strength of the unbound aggregate layer. Moreover, one agency indicated that the drainage coefficients of unbound aggregate layers were changed under different climatic conditions. One more agency indicated that the minimum thickness requirements for unbound aggregate layers were modified according to climatic conditions. Note that the survey results reported in this synthesis reflect state practices as of May 2012.

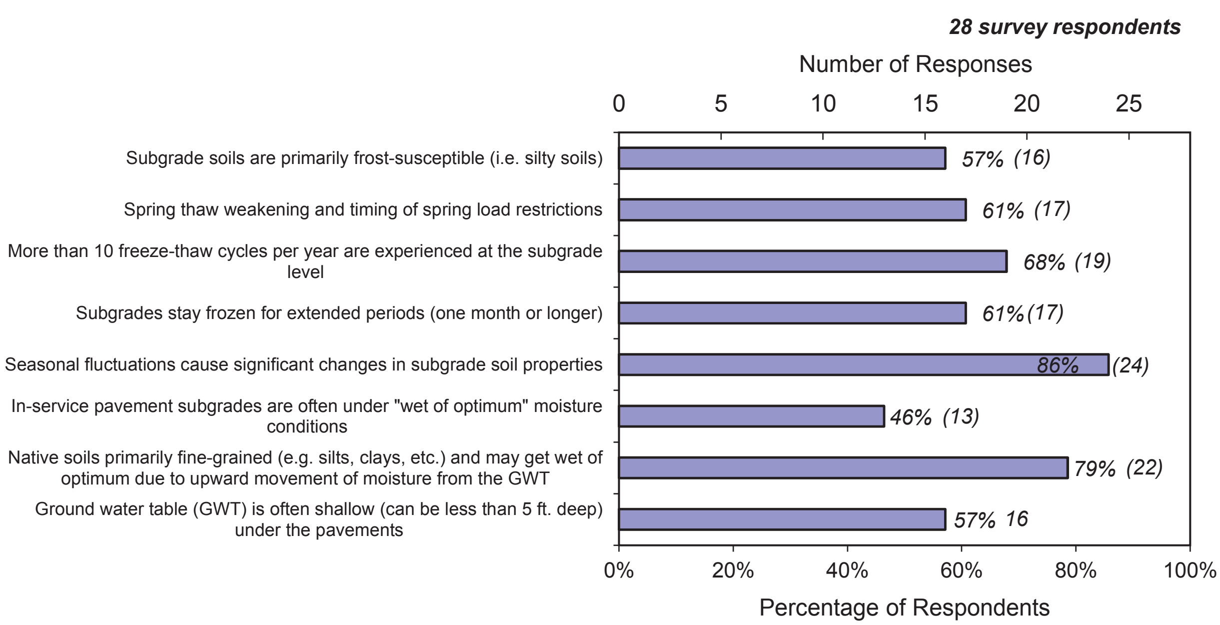


FIGURE 77 Different factors identified by state and Canadian provincial transportation agencies as responsible for affecting pavement performance under adverse climatic conditions.

Freeze-Thaw and Frost Penetration

Frost susceptibility refers to the degree to which an unbound aggregate layer is affected by the action of freeze-thaw in the presence of water. In many northern states and Canadian provinces, the pavement, base, subbase, and subgrade materials experience one or more freeze-thaw cycles during each year, leading to frost-associated pavement distresses. Pavement distresses associated mainly with frost heaving and thaw weakening can be commonly encountered given the presence of three factors: freezing temperatures, availability of moisture, and presence of frost-susceptible soils. The pavement failure mechanism associated with freeze-thaw involves nonuniform heave, which is destructive in terms of causing uneven support, whereas thaw weakening causes deformation in the base or subgrade and eventually damages pavement surface. For example, in Minnesota, frost depth typically ranges between 40 and 70 in., greatly exceeding the thicknesses of the nonfrost susceptible bases and subbases (anecdotal evidence suggests that frost depths to 96 in. have been measured in northern Minnesota). If base/subbase layers have low fines (passing No. 200 sieve or smaller than 0.075 mm) content, treating frost-susceptible subgrade soils is the emphasis of mitigating freeze-thaw damages; otherwise, both base/subbase and subgrade may need to be properly addressed.

The existing methods of mitigating frost damage in flexible pavements can be costly and sometimes cumbersome (Khan 2008). Current MnDOT concrete pavement design practices require that a certain thickness of nonfrost-susceptible or frost-free materials be incorporated into pavement designs. Frost-free materials may include aggregate base (MnDOT's Specification 3138, Classes 3, 4, 5, 6, and 7) and select granular borrow (MnDOT's Specification 3149.2B2) containing less than 12% passing the No. 200 sieve (0.075 mm). The minimum thickness of the frost-free materials is a function of the 20-year design lane ESALs and varies between 30 and 36 in. for most bituminous designs. To examine the adequacy of existing design standards for frost protection, better understanding of thermophysical properties of aggregate base/subbase materials and accurate modeling of pavement temperature-depth profile are required.

Other existing methods of preventing or minimizing frost damage are briefly reviewed as follows: (1) simply increasing the pavement thickness to account for the damage and loss of support caused by frost action as the AASHTO 1993 Guide implies; (2) reducing the depth of frost-impacted subgrade under the pavement (between the bottom of the pavement structure and frost depth) by extending the pavement sections well into the frost depth; (3) replacing the frost susceptible subgrade with nonfrost susceptible material; (4) using an insulation layer between the pavement and subgrade; (5) preventing free water from infiltrating into pavement structures; (6) providing a capillary break in the subgrade water flow path; (7) using alternative insulation materials (sawdust, sand/tire chips mix, extruded Styrofoam) for preventing frost action; (8) using a peat layer above the subgrade soils; and (9) engineering a pavement structure with reduced heat conductivity using lightweight aggregate, as proposed by Khan (2008).

According to Saeed (2008), the frost susceptibility of aggregates can be determined in terms of the USACE "F" categories and from the results of the TST (Saarenketo and Scullion 1996). The USACE method categorizes soils into several categories based on their degree of susceptibility, from F1 (least susceptible) to F4 (most susceptible). The F categories are based on general soil type and the amount of material finer than 0.02 mm. The TST measures the amount of free water that exists within an aggregate sample. The asymptotic dielectric constant value (DCV) at the end of the test can be used to characterize an aggregate as a poor (>16), marginal (10 to 16), or good (<10) performer in terms of its moisture susceptibility and frost resistance.

Key Lessons

- With proper consideration of the effects of climatic conditions, such as frost penetration and freeze-thaw cycles, on UAB/subbase layer performance, premature pavement failures can be prevented.
- TSTs are conducted to evaluate the frost susceptibility of aggregates before their application in unbound base/subbase layers in areas experiencing significant frost penetration.

REFERENCES

American Association of State Highway Officials (AASHO), AASHO Interim Guide for the Design of Rigid and Flexible Pavements, AASHO, Washington, D.C., 1961.

American Association of State Highway and Transportation Officials (AASHTO), *Pavement Design Guide,* Section 2.0, Fundamental Design Parameters, AASHTO, Washington, D.C., 1998.

American Association of State Highway and Transportation Officials (AASHTO), Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils—SHRP Protocol P46, Standard Specifications for Transportation Materials and Methods of Sampling and Testing, AASHTO T294-94, 17th ed., AASHTO, Washington, D.C., 1995.

American Association of State Highway and Transportation Officials (AASHTO), Standard Test Method for Determining the Resilient Modulus of Soils and Aggregate Materials, AASHTO T307-99, AASHTO, Washington, D.C., 2003.

American Association of State Highway and Transportation Officials (AASHTO), *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures,* Appendix GG-1: Calibration of Permanent Deformation Models for Flexible Pavements, prepared for NCHRP by ARA, Inc., AASHTO, Washington, D.C., Feb. 2004.

Adu-Osei, A., D.N. Little, and R.L. Lytton, *Structural Characteristics Of Unbound Aggregate Bases To Meet AASHTO 2002 Design Requirements,* International Center for Aggregates Research (ICAR) Report 502-1, Texas Transportation Institute, The Texas A&M University System, College Station, 2001.

Allen, J.J., *The Effects of Non-Constant Lateral Pressures on The Resilient Response of Granular Materials,* Ph.D. Dissertation, Department of Civil Engineering, University of Illinois, Urbana, 1973.

Allen, J.J. and M.R. Thompson, "Resilient Response of Granular Materials Subjected to Time-Dependent Lateral Stresses," *Transportation Research Record 510,* Transportation Research Board, National Research Council, Washington, D.C., 1974, pp. 1–13.

Arika, C.N., D.J. Canelon, and J.L. Nieber, *Subsurface Drainage Manual for Pavements in Minnesota,* Final Report MN/RC 2009-17, Minnesota Department of Transportation, St. Paul, 2009.

Ashtiani, R.S. and D. Little, *Acceptability Criteria for High Fines Content Aggregate Pavement Layers,* Report No ICAR 401131, International Center for Aggregates Research (ICAR), College Station, Tex., 2007, 100 pp.

Ashtiani, R.S. and D. Little, *Methodology For Designing Aggregate Mixtures For Base Courses,* Report No 508-1, International Center for Aggregates Research (ICAR), College Station, Tex., 2009, 336 pp.

Barksdale, R.D. and H.A. Todres, *A Study of Factors Affecting Crushed Stone Base Performance,* School of Civil Engineering, Georgia Institute of Technology, Atlanta, 1983.

Barksdale, R.D., S.F. Brown, and F. Chan, *NCHRP Report 315: Potential Benefits of Geosynthetics in Flexible Pavements,* Transportation Research Board, National Research Council, Washington, D.C., 1989.

Barksdale, R.D. and J.L. Alba, *Laboratory Determination of Resilient Modulus for Flexible Pavement Design,* Interim Report No. 2, prepared for NCHRP, Transportation Research Board, National Research Council, Washington, D.C., 1993.

Barksdale, R.D., J. Alba, P.N. Khosla, R. Kim, P.C. Lambe, and M.S. Rahman, *NCHRP Web Document 14: Laboratory Determination of Resilient Modulus for Flexible Pavement Design,* Transportation Research Board, National Research Council, Washington, D.C., 1997, 486 pp.

Beatty, T.L., et al., *Pavement Preservation Technology in France, South Africa, and Australia,* Office of International Programs, Federal Highway Administration, U.S. Department of Transportation, and the American Association of State Highway and Transportation Officials, Alexandria, Va., 2002.

Birgisson, B. and R. Roberson, "Drainage of Pavement Base Material: Design and Construction Issues," *Transportation Research Record: Journal of the Transportation Research Board, No. 1709,* Transportation Research Board of the National Academies, Washington, D.C., 2000, pp. 11–18.

Boussinesq, J., "Application des Potentiels a l'etude de l'equilibre et du Mouvement des Solids Elastiques," Gauthier-Villars, Paris, 1885.

British Standard BS EN 13286-7, Unbound and Hydraulically Bound Mixtures. Cyclic Load Triaxial Test for Unbound Mixtures, British-Adopted European Standard, Feb. 2004, 40 pp.

Broms, B., "Lateral Earth Pressure Due to Compaction of Cohesionless Soils," *Proceedings, 4th Budapest Conference on Soil Mechanics and Foundation Engineering,* 1971, pp. 373–383.

Brown, S.F. and A.F.L. Hyde, "Significance of Cyclic Confining Stress in Repeated-Load Triaxial Testing of Granular Material," *Transportation Research Record 537,* Transportation Research Board, National Research Council, Washington, D.C., 1975, pp. 49–58.

Brown, S.F. and J.W. Pappin, "Analysis of Pavements with Granular Bases," *Transportation Research Record 810,* Transportation Research Board, National Research Council, Washington, D.C., 1981, pp. 17–23.

Brown, S.F. and B.V. Brodrick, "25 Years' Experience with the Pilot-Scale Nottingham Pavement Test Facility," *Proceedings of the International Conference on Accelerated Pavement Testing,* Reno, Nev., Oct. 18–20, 1999.

Carrera, A., A. Dawson, and J. Steger, *State of the Art Materials' Sensitivity to Moisture Content Change: Report Nr. 2,* P2R2C2, Pavement Performance and Remediation Requirements due to Climate Change, Road ERA Net, 2009.

CEN EN 13286-7, Unbound and Hydraulically Bound Mixtures—Test Methods—Part 7: Cyclic Load Triaxial Test for Unbound Mixtures, CEN, European Committee for Standardization, 2004.

COST Action 337, *Unbound Granular Materials for Road Pavements,* Final Report, Chapter 8, European Commission, Directorate General for Research and Technological Development, Brussels, Belgium, 2002.

Chen, D.H. and H.H. Lin, "Effect of Base Support and Load Transfer Efficiency (LTE) on Portland Concrete Pavement Performance," *Journal of Testing and Evaluation,* No. 38, No. 1, 2010, pp. 1–10.

Christopher, B.R., C. Schwartz, and R. Boudreau, *Geotechnical Aspects of Pavements,* Report FHWA NHI-10-092, National Highway Institute, Federal Highway Administration, Washington, D.C., 2010.

Crockford, W.W., L.J. Bendana, W.S. Yang, S.K. Rhee, and S.P. Senadheera, Modeling Stress and Strain States In Pavement Structures Incorporating Thick Granular Layers, Final Report, Contract F08635-87-C-0039, The Texas Transportation Institute, Texas A&M University, College Station, 1990.

Cundall, P.A. and O.D.L. Strack, "A Discrete Numerical Model for Granular Assemblies," *Geotechnique,* Vol. 29, 1979, 47 pp.

Dantu, P., "Contribution a l'Etude Mechanique et Geometrique des Milieux Pulverulents," *Proceedings, 4th International Conference on Soil Mechanics,* Vol. 1, London, U.K., 1957.

Dawson, A., "Pavement Performance and Remediation Requirements Following Climate Change," 2010 [Online]. Available: http://www.nottingham.ac.uk/~evzard/P2R2C2/Brochure.pdf.

Dhamrait, J.S. and D.R. Schwartz, *Effect of Subbase Type and Subsurface Drainage on Behavior of CRC Pavements,* Interim Report IHR-36, Illinois Department of Transportation, Springfield, 1979, 70 pp..

Dobry, R., T.T. Ng, and E. Petrakis, "Deformation Characteristics of Granular Soil in the Light Of Particulate Mechanics," Proceedings, *14th Conference on Geotechnics,* Italian Geotechnical Association, Torino, Nov. 28–30, 1989.

Edwards, J.P., N.H. Thom, P.R. Fleming, and J. Williams, "Accelerated Laboratory Based Mechanistic Testing of Unbound Materials within the Newly Developed NAT Springbox," *Transportation Research Record: Journal of the Transportation Research Board, No. 1913,* Transportation Research Board of the National Academies, Washington, D.C., 2005.

Elhannani, M., *Modelisation et Simulation Numerique des Chaussees Souples,* Ph.D. Thesis, University of Nantes, France, 1991.

Erlingsson, S. and T. Ingason, "Performance of Two Thin Pavement Structures during Accelerated Pavement Testing Using a Heavy Vehicle Simulator," *Proceedings of the Second International Conference on Accelerated Pavement Testing,* University of Minnesota, Minneapolis, 2004.

Federal Highway Administration (FHWA), "Transportation Applications of Recycled Concrete Aggregate," *FHWA State of the Practice National Review,* FHWA, Washington, D.C., Sep. 2004.

Fledlund, D.G. and A. Xing, "Equations for the Soil-Water Characteristic Curve," *Canadian Geotechnical Journal,* Vol. 31, No. 4, 1994, pp. 533–546.

Garg, N., E. Tutumluer, and M.R. Thompson, "Structural Modeling Concepts for the Design of Airport Pavements for Heavy Aircraft," *Proceedings, 5th International Conference on the Bearing Capacity of Roads and Airfields,* Trondheim, Norway, July 1998, pp. 115–124.

Hagen, M.G. and G.R. Cochran, "Comparison of Pavement Drainage Systems." *Transportation Research Record 1519,* Transportation Research Board, National Research Council, Washington, D.C., 1995, pp. 1–10.

Hicks, R.G., *Factors Influencing the Resilient Properties of Granular Materials,* PhD Dissertation, Institute of Transportation and Traffic Engineering, University of California, Berkeley, 1970.

Holl, D.L., "Plane Strain Distribution of Stress in Elastic Media," *Iowa Engineering Experimentation Station Bulletin,* 1941, pp. 148–163.

Hornych, P., A. Kazai, and A. Quibel, "Modeling a Full-Scale Experiment of Two Flexible Pavement Structures with Unbound Granular Bases," *Unbound Aggregates in Road Construction,* A.R. Dawson, Ed., A.A. Balkema Publishers, *Proceedings of the Unbound Aggregates in Roads* (*UNBAR5*) *Symposium,* University of Nottingham, England, June 21–23, 2000, pp. 359–367.

Ingold, T.S., "The Effect of Compaction on Retaining Walls," *Geotechnique,* Vol. 29, No. 3, 1979, pp. 265–283.

Jung, Y.S., D.G. Zollinger, M. Won, and A.J. Wimsatt, *Subbase and Subgrade Performance Investigation for Concrete Pavement,* Final Report FHWA/TX-09/0-6037-1, Texas Department of Transportation, Austin, 2009.

Jung, Y.S., D.G. Zollinger, and A.J. Wimsatt, "Test Method and Model Development of Subbase Erosion for Concrete Pavement Design," *Transportation Research Record; Journal of the Transportation Research Board, No. 2154,* Transportation Research Board of the National Academies, Washington, D.C., 2010, pp. 22–31.

Jung, Y.S. and D.G. Zollinger, "New Laboratory-Based Mechanistic-Empirical Model for Faulting in Jointed Concrete Pavement," *Transportation Research Record; Journal of the Transportation Research Board, No. 2226,* Transportation Research Board of the National Academies, Washington, D.C., 2011, pp. 60–70.

Karasahin, M., A.R. Dawson, and J.T. Holden, "Applicability of Resilient Constitutive Models of Granular Material for Unbound Base Layers," *Transportation Research Record 1406,* Transportation Research Board, National Research Council, Washington, D.C., 1993, pp. 98–107.

Khan, A.A., *Feasibility of Using Lightweight Aggregate to Mitigating Frost Damage in Flexible Pavements,* PhD Thesis, Department of Civil Engineering, University of New Brunswick, Canada, 2008.

Khazanovich, L., M. Darter, R. Bartlett, and T. McPeak, *Common Characteristics of Good and Poorly Performing PCC Pavements,* Final Report FHWA-RD-97-131, Federal Highway Administration, McLean, Va., 1998.

Kim, I.T. and E. Tutumluer, "Unbound Aggregate Rutting Models for Stress Rotation and Effects of Moving Wheel Loads," *Transportation Research Record: Journal of the Transportation Research Board, No. 1913,* Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 41–52.

Kim, I.T., *Permanent Deformation Behavior of Airport Flexible Pavement Base and Subbase Courses,* PhD Dissertation, University of Illinois at Urbana–Champaign, Urbana, 2005.

Kim, S.-H., D.N. Little, and E. Masad, "Simple Methods to Estimate Inherent and Stress-Induced Anisotropy of Aggregate Base," *Transportation Research Record: Journal of the Transportation Research Board, No. 1913,* Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 24–31.

Kim, S.-H., E. Tutumluer, D.N. Little, and N. Kim, "Effect of Gradation on Nonlinear Stress-Dependent Behavior of a Sandy Flexible Pavement Subgrade," *Journal of Transportation Engineering,* ASCE, Vol. 133, No. 10, Oct. 2007, pp. 590–598.

Kolisoja, P., T. Saarenketo, H. Peltoniemi, and N. Vuorimies, "Laboratory Testing of Suction and Deformation Properties of Base Course Aggregates," *Transportation Research Record: Journal of the Transportation Research Board, No. 1787,* Transportation Research Board of the National Academies, Washington, D.C., 2002, pp. 83–89.

Kwon, J., E. Tutumluer, I.L. Al-Qadi, and S. Dessouky, "Mechanistic Model Response Predictions of Geogrid Base Reinforced Flexible Pavements," *Proceedings, GeoAmericas 2008,* The First Pan American Geosynthetics Conference, Cancun, Mexico, Mar. 2–5, 2008.

Larson, G. and B. Dempsey, *Enhanced Integrated Climatic Model,* Version 2.0, Report DTFA MN/DOT 72114, Department of Civil Engineering, University of Illinois at Urbana–Champaign, Urbana, 1997.

Lekarp, F., U. Isacsson, and A. Dawson, "State of the Art. I: Resilient Response of Unbound Aggregates," *Journal of Transportation Engineering,* Vol. 126, No. 1, 2000a, pp. 66–75.

Lekarp, F., U. Isacsson, and A. Dawson, "State of the Art. II: Permanent Strain Response of Unbound Aggregates," *Journal of Transportation Engineering,* Vol. 126, No. 1, 2000b, pp. 76–83.

Love, A.E.H., *A Treatise on the Mathematical Theory of Elasticity,* 4th ed., Dover Publications, New York, N.Y., 1944, 643 pp.

Masad, S., D.N. Little, and E. Masad, "Analysis of Flexible Pavement Response and Performance Using Isotropic and Anisotropic Material Properties," *Journal of Transportation Engineering,* ASCE, Vol. 132, No. 4, 2006, pp. 342–349.

Mishra, D., *Aggregate Characteristics Affecting Response and Performance of Unsurfaced Pavements on Weak Subgrades,* PhD Thesis, University of Illinois at Urbana–Champaign, Urbana, 2012.

Mishra, D. and E. Tutumluer, "Aggregate Physical Properties Affecting Modulus and Deformation Characteristics of Unsurfaced Pavements," Paper 11-3486, 90th Annual Meeting of the Transportation Research Board, Jan. 2011, Washington, D.C., 16 pp.

Nataatmadja, A., *Variability of Pavement Material Parameters under Repeated Loading,* PhD Dissertation, Monash University, Australia, 1989.

NCHRP 1-37A, *Mechanistic-Empirical Pavement Design Guide,* Draft Report, Part 2, Design Inputs, Transportation Research Board of the National Academies, Washington, D.C., 2004.

Newcomb, D.E., S.W. Lee, J.P. Mahoney, and N.C. Jackson, "The Use of Falling Weight Deflectometer Data in Monitoring Flexible Pavement Systems," *Nondestructive Testing of Pavements and Backcalculation of Moduli,* ASTM STP 1026, A.J. Bush III and G.Y. Baladi, Eds., American Society for Testing and Materials, Philadelphia, Pa, 1989, pp. 470–486.

Oda, M., "A Mechanical and Statistical Model of Granular Materials," *Soils and Foundations,* Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 14, 1974.

Puppala, A.J., *Synthesis of Highway Practice 382: Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design,* Transportation Research Board of the National Academies, Washington, D.C., 2008.

Richter, C.A., *Seasonal Variations in the Moduli of Unbound Pavement Layers,* Report FHWA-HRT-04-079, Federal Highway Administration, McLean, Va., 2006.

Saarenketo, T. and T. Scullion, *Using Electrical Properties to Classify the Strength Properties of Base Course Aggregates,* Final Report FHWA/TX-97/1341-2, Texas Department of Transportation, Austin, 1996.

Saeed, A., J.W. Hall, Jr. and W. Barker, *NCHRP Report 453: Performance-Related Tests of Aggregates for Use in Unbound Pavement Layers,* Transportation Research Board, National Research Council, Washington, D.C., 2001, 55 pp.

Saeed, A., *NCHRP Report 598: Performance-Related Tests of Recycled Aggregates for Use in Unbound Pavement Layers,* Transportation Research Board of the National Academies, Washington, D.C., 2008, 53 p.

Selig, E.T., "Tensile Zone Effects on Performance of Layered Systems," *Geotechnique,* Vol. 37, No. 3, 1987, pp. 247–254.

Semmelink, C.J. and M. de Beer, "Rapid Determination of Elastic and Shear Properties of Road-Building Materials with the K-Mould," *Proceedings of the UNBAR4 Symposium,* A.R. Dawson and R.H. Jones, Eds., July 17–19, 1995, Nottingham, U.K., pp. 151–162.

Seyhan, U. and E. Tutumluer, "Unbound Granular Material Characterization from Stress Path Loading Tests," *Geotechnical Special Publication* (*GSP*) *No. 89, Recent Advances in the Characterization of Transportation Geomaterials,* E. Tutumluer and A.T. Papagiannakis, Eds., 1999, pp. 49–60.

Seyhan, U., E. Tutumluer, and H. Yesilyurt, "Anisotropic Aggregate Base Inputs for Mechanistic Pavement Analysis Considering Effects of Moving Wheel Loads," *ASCE Journal of Materials in Civil Engineering,* Vol. 17, No. 5, Oct. 2005, pp. 505–512.

Shackel, B., "Repeated Loading of Soils–A Review," *Australian Road Research,* Vol. 5, No. 3, 1973, pp. 22–49.

Shackel, B., *Some Effects of Repeated Triaxial Stresses on Road Pavement Materials,* PhD Thesis, The University of New South Wales, 1974.

Steven, B., D. Alabaster, and J. de Pont, "Elastic Nonlinear Finite Element Analysis of a Flexible Pavement Subjected to Varying Falling Weight Deflectometer Loads," *Transportation Research Record: Journal of the Transportation Research Board, No. 2016,* Transportation Research Board of the National Academies, Washington, D.C., 2007, pp. 31–38.

Stewart, E.H., E.T. Selig, and G.H. Norman-Gregory, "Failure Criteria and Lateral Stresses in Track Foundations," *Transportation Research Record 1022,* Transportation Research Board, National Research Council, Washington, D.C., 1985, pp. 59–64.

Tao, M. and M.Y. Abu-Farsakh, *Effect of Drainage in Unbound Aggregate Bases on Flexible Pavement Performance,* Final Report FHWA/LA.07/429, Louisiana Transportation Research Center, Baton Rouge, 2008, 48 pp.

Tao, M., M.N. Louay, N.D. Munir, Z. Zhang, and Z. Wu, "Application of Shakedown Theory in Characterizing Traditional and Recycled Pavement Base Materials," *Journal of Transportation Engineering,* Vol. 136, No. 3, 2010, pp. 214–222.

Thompson, M.R., "State-of-the-Art: Unbound Base Performance," *Proceedings of the 6th Annual Symposium of International Center for Aggregate Research* (*ICAR*), Austin, Tex., 1998.

Toros, U. and D.R. Hiltunen, "Effects of Moisture and Time on Stiffness of Unbound Aggregate Base Course Materials," *Transportation Research Record: Journal of the Transportation Research Board, No. 2016,* Transportation Research Board of the National Academies, Washington, D.C., 2008, pp. 41–51.

Tutumluer, E., *Predicting Behavior of Flexible Pavements with Granular Bases,* PhD Dissertation, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, Ga., 1995.

Tutumluer, E., "Anisotropic Behavior of Unbound Aggregate Bases—State of the Art Summary." Invited Paper, *Proceedings, 6th Annual Symposium of the International Center for Aggregate Research* (*ICAR*), St. Louis, Mo., Apr. 19–21, 1998, pp. 11–33.

Tutumluer, E. and R.D. Barksdale, "Behavior of Pavements with Granular Bases—Prediction and Performance." *Proceedings of the International Symposium on Unbound Aggregates in Roads (UNBAR4),* Nottingham, U.K., July 1995.

Tutumluer, E. and M.R. Thompson, "Granular Base Moduli for Mechanistic Pavement Design," *Proceedings,* ASCE Airfield Pavement Conference, Seattle, Wash., Aug. 17–20, 1997a, pp. 33–47.

Tutumluer, E. and M.R. Thompson, "Anisotropic Modeling of Granular Bases in Flexible Pavements," *Transportation Research Record 1577,* Transportation Research Board, National Research Council, Washington, D.C., 1997b, pp. 18–26.

Tutumluer, E. and M.R. Thompson, *Anisotropic Modeling of Granular Bases,* Technical Research Report Submitted to the Federal Aviation Administration, Department of Civil Engineering, University of Illinois at Urbana–Champaign, Urbana, 1998.

Tutumluer, E. and U. Seyhan, "Laboratory Determination of Anisotropic Aggregate Resilient Moduli Using An Innovative Test Device," *Transportation Research Record: Journal of the Transportation Research Board 1687,* Transportation Research Board, National Research Council, Washington, D.C., 1999, pp. 13–21.

Tutumluer, E. and I.T. Kim, "Permanent Deformation Behavior of Airport Pavement Granular Layers," *Proceedings of the 3rd International Conference on Deformation Characteristics of Geomaterials* (*IS LYON 2003*)," H. DiBenedetto, T. Doanh, H. Geffroy, and C. Sauzeat, Eds., Lyon, France, Sep. 22–24, 2003, A.A. Balkema Publishers, the Netherlands, pp. 345–352.

Tutumluer, E. and U. Seyhan, "Effects of Fines Content on the Anisotropic Response and Characterization of Unbound Aggregate Bases," *Proceedings, 5th International Conference on Unbound Aggregates in Roads* (*UNBAR5*), Nottingham, England, 2000, pp. 153–161.

Tutumluer, E., A. Adu-Osei, D.N. Little, and R.L. Lytton, *Field Validation of the Cross-Anisotropic Behavior of Unbound Aggregate Bases,* International Center for Aggregates Research (ICAR) Report 502-2, Texas Transportation Institute, The Texas A&M University System, College Station, 2001.

Tutumluer, E., D.N. Little, and S.H. Kim, "Validated Model for Predicting Field Performance of Aggregate Base Courses," *Transportation Research Record: Journal of the Transportation research Board, No. 1837,* Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 41–49.

Tutumluer, E., "State of the Art: Anisotropic Characterization of Unbound Aggregate Layers in Flexible Pavements," *ASCE Geotechnical Special Publication No. 184, Pavements and Materials—Modeling, Testing and Performance,* Z. You, A.R. Abbas, and L. Wang, Eds., 2009, pp. 1–16.

Uhlmeyer, J.S., J.P. Mahoney, G. Hanek, G. Wang, R.L. Copstead, and D.J. Janssen, *Estimation of Seasonal Effects for Pavement Design and Performance—Volume 1,* Report FHWA-FLP-95-006, Federal Highway Administration, Washington, D.C., 1996.

U.S. Army Corps of Engineers, "The Unified Soil Classification System," *Technical Memorandum 3-357,* Waterways Experiment Station, Vicksburg, Miss., 1953.

Uzan, J., "Characterization of Granular Material," *Transportation Research Record 1022,* Transportation Research Board, National Research Council, Washington, D.C., 1985, pp. 52–59.

Werkmeister, S., *Permanent Deformation Behaviour of Unbound Granular Materials in Pavement Constructions,* PhD Thesis, Dresden University of Technology, Dresden, Germany, 2003.

Werkmeister, S., R. Numrich, A.R. Dawson, and F. Wellner, "Design of Granular Pavement Layers Considering Climatic Conditions," *Transportation Research Record: Journal of the Transportation Research Board, No. 1837,* Transportation Research Board of the National Academies, Washington, D.C., 2003, pp. 61–70.

Werkmeister, S., A.R. Dawson, and F. Wellner, "Pavement Design Model for Unbound Granular Materials," *Journal* *of Transportation Engineering,* ASCE, Vol. 130, No. 5, 2004, pp. 665–674.

White, D.J., P.K.R. Vennapusa, D. Eichner, H. Gieselman, L. Zhao, and C. Jahren, *Rapid, Self-Contained In situ Permeameter for Field QA/QC of Pavement Base/Subbase Materials,* Final Report for Highway IDEA Project 130, Transportation Research Board of the National Academies, Washington, D.C., 2010, 50 pp.

Winkelman, T.J., *Open Graded Drainage Layer Performance in Illinois,* Final Report FHWA/IL/PRR 147, Illinois Department of Transportation, Springfield, 2004.

Xiao, Y., E. Tutumluer, Y. Qian, and J. Siekmeier, "Gradation Effects Influencing Mechanical Properties of Aggregate Base and Granular Subbase Materials in Minnesota," *Transportation Research Record: Journal of the Transportation Research Board, No. 2267,* Transportation Research Board of the National Academies, Washington, D.C., 2012, pp. 14–26.

Zapata, C.E. and W.N. Houston, *NCHRP Report 602: Calibration and Validation of the Enhanced Integrated Climate Model for Pavement Design,* Transportation Research Board of the National Academies, Washington, D.C., 2008.

Zeilmaker, J, and R.J. Henny, "The Measurement of Residual Stresses Due to Compaction in Granular Materials," In *Proceedings, International Symposium on Unbound Aggregates on Roads* (*UNBAR3*), Unbound Aggregates in Roads, R.H. Jones and A.R. Dawson, Eds., Reed International Publishing, United Kingdom, Apr. 1989, pp. 159–168.

CHAPTER FIVE

COMPACTION, QUALITY CONTROL, AND FIELD PERFORMANCE

INTRODUCTION

This chapter presents detailed findings on different approaches used by transportation agencies for compaction testing on laboratory samples, field compaction, QC/QA, and field performance evaluations of constructed UAB/subbase layers. Different aspects of compaction and QC of UAB and subbase construction are discussed by first introducing the theory of compaction along with the objectives behind compacting unbound aggregate pavement layers. This is followed by a review of different types of compactors commonly used for compacting UAB and subbase layers in the field. The concept of QC is introduced, emphasizing that constructed layer density measurement is the most commonly used field evaluation tool for verifying the adequacy of UAB/subbase construction. However, laboratory testing is needed to establish the target densities and acceptance criteria in field compaction of aggregate layers. Different field techniques used to measure densities of constructed pavement layers are discussed, with particular emphasis on the widespread nuclear gauge-based direct density measurement methods.

The concept of modulus-based compaction control is introduced by highlighting its potential advantages, such as continuous compaction control for uniformity over the “spot checking” compared with density-based compaction control approaches. Different IC approaches are also discussed through a review of equipment manufacturers. Furthermore, experiences of different states in the United States for implementing IC approaches are presented, along with their preliminary findings.

Finally, this chapter discusses other portable devices used for measuring the in situ moduli of constructed pavement layers. Salient features of each device are discussed, and the advantages and disadvantages of individual devices are highlighted. Research studies and trial projects conducted by different agencies through QC using these portable devices are listed and a summary of their important findings provided.

COMPACTION AND QUALITY CONTROL

Theory and Objectives of Compaction

Compaction is defined as the densification of soils and construction materials through the application of mechanical energy. Primary objectives of compaction are to (1) reduce/prevent detrimental settlements (compaction leads to better packing of individual particles, thus reducing the potential for excessive settlement); (2) increase the shear strength and thus improve slope stability; (3) improve the bearing capacity of pavement subgrades and granular subbase/base layers; and (4) control undesirable volume changes caused by frost action, swelling, and shrinkage (Holtz 1990). Although most initial research efforts focused on compaction were concerned with the compaction of soils (Proctor 1933; Seed 1959), the compaction of aggregates as geomaterials is equally important in the construction of pavement layers. As discussed, the primary mechanism of load transfer within an aggregate layer is through particle-to-particle interlock. The process of compaction reorients the particles within a loose aggregate layer and creates a densely packed matrix. This densely packed aggregate matrix demonstrates significantly higher shear strength and resilient modulus, as well as significantly lower susceptibility to permanent deformation compared with a loose uncompacted layer of aggregates.

Although the process of compaction invariably results in higher densities achieved in the compacted layers, the achievement of higher densities is not one of the primary objectives of compaction. Rather, density is an indicator of achieved compaction levels and often can be linked to other mechanical properties of soils and aggregates, such as shear strength and susceptibility to permanent deformation accumulation. Inadequate compaction of pavement subgrade, subbase, or base layers may result in excessive rutting, leading to pavement shear failure.

Marek and Jones (1974) highlight the difference between “compaction” and “density,” emphasizing that two aggregate base materials compacted to the same density may be at completely different stages of compaction. They emphasize that the state of compaction of an aggregate material is dependent on its gradation, so depending on the amount of fines in an aggregate matrix, higher density numbers may not always correspond to “better” states of compaction. According to Proctor (1933), the compactability of a soil or aggregate layer depends on the following factors: (1) compactive energy, (2) moisture content, and (3) soil/aggregate type.

Establishing the Target Density for Field Compaction Control

The primary method for measuring the compaction level in a pavement layer is by comparing the achieved field densities with reference target values determined for the same material in the laboratory. The in-place densities of constructed layers are subsequently expressed as percentages of these reference

TABLE 11
COMPARISON OF ASTM AND AASHTO TEST METHODS GOVERNING THE COMPACTION OF SOILS AND AGGREGATES USING DROP HAMMER METHOD

Equipment/Test Parameter	ASTM Standard (D 698)	ASTM Modified (D 1557)	AASHTO Standard T 99	AASHTO Modified T 180
Mold diameter	Method A: 101.6 mm Method B: 101.6 mm Method C: 152.4 mm		Method A: 101.6 mm Method B: 152.4 mm Method C: 101.6 mm Method D: 152.4 mm	
Mold volume (cm^3)	943.0 for 101.6-mm diameter mold 2,124 for 152.4-mm diameter mold			
Number of layers	3	5	3	5
Number of blows/layer	25 for 101.6-mm diameter mold 56 for 152.4-mm diameter mold			
Material specifications [material finer than sieve opening size (%)]	Method A: 4.75 mm Method B: 9.50 mm Method C: 19.0 mm		Method A: 4.75 mm Method B: 4.75 mm Method C: 19.0 mm Method D: 19.0 mm	

densities established in the laboratory. Construction specifications for pavement layers often require the achieved field densities to be higher than a certain specified percentage of this target density value. The applicability of relative compaction values thus determined is dependent on the validity of the following two assumptions: (1) the material tested in the laboratory is identical to the field material in gradation and specific gravity, and (2) similar compactive energies are imparted to the material in the field, as well as in the laboratory. Upon the violation of one or both of these assumptions, the calculated "percent compaction" becomes meaningless (Marek and Jones 1974). Some of the commonly used methods for establishing the "target density" values of unbound aggregate materials in the field are discussed here.

Compaction Using Drop Hammer Methods

Drop hammer methods are the test methods most commonly used for establishing the compaction characteristics of soils and aggregates in the laboratory. Originally proposed by Proctor (1933), these methods involve the compaction of a representative portion of the material into a standard size mold using a rammer dropped from a fixed height. Depending on the weight of the rammer and the drop height, the procedure is termed either a standard or modified compaction procedure. It is important to note that the rammer blows in Proctor's method were specified as "firm strokes," whereas the test methods currently used involve free fall of the drop hammer over a fixed height. Equipment specifications, test methodology, and material to be tested using these methods are described in standard specifications by ASTM and AASHTO. The standard method involves compaction of a representative portion of the aggregate material into a standard size mold (101.6-mm or 152.4-mm diameter) with a 24.5-N (5.5-lbf) rammer dropped from a height of 305 mm (12.0 in.). The modified compaction method involves a 44.48-N (10.0-lbf) hammer dropped from a height of 457.2 mm (18.0 in.). Specifications for the standard compaction procedure have been provided as ASTM D 698 or AASHTO T 99, and those for the modified compaction procedure have been provided as ASTM D 1557, or AASHTO T 180. Note that the ASTM and AASHTO methods differ somewhat in the maximum size of aggregate particles that can be tested. Moreover, owing to the use of a heavier hammer and higher drop height, the modified compaction procedure imparts much higher compaction energy to the aggregate specimen (4.5 times) than does the standard compaction procedure. Table 11 lists the similarities and differences between the two compaction methods as specified by the ASTM and AASHTO standards. It is important to note that several state and Canadian provincial agencies use modified versions of the original ASTM and AASHTO specifications as part of their agency guidelines. Although these agency-specific guidelines are somewhat different from the ASTM and AASHTO standards, the basic procedures and principles remain the same.

Figure 78 shows the typical compaction curves for a commonly used dense-graded crushed limestone material with 10% P_{200} fines. As shown in the figure, a higher compac-

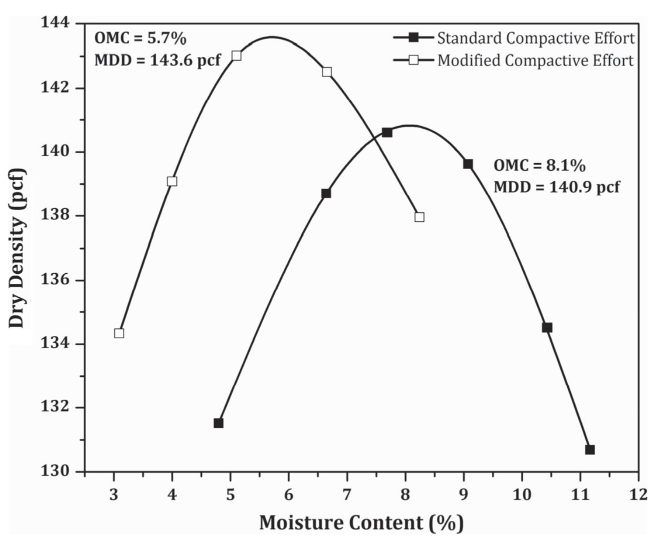


FIGURE 78 Typical compaction curves for a dense-graded crushed limestone material with 10% P_{200} fines (1 pcf = 16.02 kg/m^3).

tive energy leads to an increase in the maximum dry density (MDD) value and a decrease in the OMC.

Note that drop-hammer–based compaction methods are commonly used by transportation agencies to establish reference target densities, such as 95% to 100% of laboratory MDD values, before the construction of UAB and subbase layers. The survey of state and Canadian provincial transportation agencies conducted under the scope of the current synthesis study indicated that 42 of 46 responding agencies use drop-hammer–based methods to establish the compaction characteristics of unbound aggregate materials in the laboratory. Only two agencies (the Kansas and Alabama DOTs) reported the use of vibratory compaction methods. One agency (Alberta Transportation, Canada) does not require aggregate compaction characteristics to be established in the laboratory.

It is important to note that drop-hammer–based compaction methods specified in AASHTO T 99 and T 180 were derived from the original methods proposed by Proctor (1933), which in turn were developed for fine-grained soils. Accordingly, the use of impact compaction may not be adequate for establishing the compaction characteristics of certain aggregate types, such as open-graded materials with insufficient P_{200} fines. Absence of sufficient P_{200} fines results in "shifting" of individual aggregate particles under impact compaction, thus preventing the formation of a densely packed matrix. Thus, vibratory compaction can be used to establish the compaction characteristics of such materials. Although ASTM method D 7382 (Standard Test Methods for Determination of Maximum Dry Unit Weight and Water Content Range for Effective Compaction of Granular Soils Using a Vibrating Hammer) provides such an alternative, no AASHTO method directs the compaction of unbound aggregates using vibratory methods. Because the compaction of UAB/subbase layers commonly involves vibratory and shearing action, establishing aggregate compaction characteristics in the laboratory using vibratory or gyratory compaction methods may lead to better representation of field conditions in the laboratory.

Control Strip or "Test Strip" Method

The control strip or "test strip" technique involves the construction of a control strip using the same material as that used to construct the UAB/subbase layer. This strip is compacted through repeated rolling and vibration, and density tests are performed after each rolling until no additional increase in density is noticed. The average final density of the control strip is used as the "maximum" density for the particular aggregate material. Construction specifications require the aggregate base/subbase layer to be compacted to a certain percentage of this "maximum" density (Marek and Jones 1974). For successful implementation of the control strip method, the compaction of the strip is correlated to previously established compaction results. In the absence of adequate moisture and compaction equipment, the maximum density achieved in the strip may not represent the densest feasible state of compaction. A new "test strip" often is required when (1) a change in the source of material is made; (2) a change in the material from the same source is observed; and/or (3) when 10 test sections are approved without a new control strip (Anday and Hughes 1967).

Solid Volume Density Method

Certain construction specifications can also be based on the solid volume density of the aggregate as a reference. The solid volume density is obtained by multiplying the specific gravity of the aggregate material with the unit weight of water (9.81 kN/m^3 or 62.4 pounds per cubic foot). The solid volume density represents the density of a particular aggregate material in a "void-less" matrix. Constructed layer densities are expressed as a percentage of the solid volume density, and the fraction is termed relative solid density. One of the most common examples of construction specifications using the solid volume density method can be seen in the construction of G1 base in South Africa. Note that for successful implementation of the solid volume density method, the correlation between achieved densities in the field and the void-less density should be known. For example, a relative solid density value of 86% typically corresponds to approximately 100% to 105% of the maximum dry density value obtained using the modified compaction method as per AASHTO T 180 (Buchanan 2010). Note that this correlation is just an example, and the exact correlation will vary depending on the aggregate mineralogy, gradation, and particle shape and surface texture.

Key Lessons

- Compaction characteristics of aggregates established in the laboratory are strongly governed by compaction methods. For example, the maximum dry density values established using AASHTO T 99 are consistently lower than those established using AASHTO T 180 because of the lower compaction energy imparted to the aggregate specimen in the former.
- Drop-hammer–based compaction methods (e.g., AASHTO T 99 and T 180) may not be adequate for coarse-grained aggregates, particularly with low fines (P_{200}) contents.
- Test procedures similar to ASTM D 7382 that establish the moisture-density curves for unbound aggregates using a vibratory (or a gyratory) compactor may lead to better representation of field conditions in the laboratory.

Compaction Variables and Equipment Types

The DOC achieved in a constructed unbound aggregate layer is dependent on the interaction between several variables, which can be broadly classified into the following two categories: (1) aggregate material and layer characteristics and (2) compaction equipment and operating characteristics. Different variables falling under the two categories are described here.

Aggregate Material and Layer Characteristics

The following variables can be grouped under this category and affect the DOC of unbound aggregate layers by governing the arrangement of individual particles in the aggregate matrix:

1. Type of parent rock (in terms of the hardness and durability of individual particles);
2. Particle shape and surface texture;
3. Gradation or particle size distribution;
4. Construction lift thickness;
5. Moisture content; and
6. Layer support conditions.

Compaction Equipment and Operating Characteristics

The following variables related to the compaction equipment and operating characteristics affect the DOC achieved in unbound aggregate layers by governing the amount of energy imparted to the layer surface:

1. Roller type;
2. Roller weight/energy;
3. Roller speed or dwell time;
4. Number of passes or coverages;
5. Rolling zone; and
6. Rolling pattern.

The roller types commonly used in the compaction of constructed pavement layers are discussed here.

Smooth Drum Rollers Smooth drum rollers are probably the most commonly used compaction devices during the construction of UAB and subbase layers. These rollers can consist of a single drum or dual drums that apply pressure across the drum width. These rollers can also be "static" or "vibratory" in nature. Static smooth drum rollers compact the pavement layers through static application of the equipment dead weight, but vibratory smooth drum rollers are equipped with oscillatory vibrators to increase the energy transmitted to the layer surface. Vibratory smooth drum rollers are best suited for unbound aggregates and noncohesive soils. In addition, these rollers sometimes are used to finish subgrades before the construction of base/subbase layers. Figure 79 is a photo of a smooth drum vibratory roller (single drum) used to compact a crushed limestone base course.

FIGURE 79 Compaction of a crushed limestone base course using a smooth drum vibratory roller.

Sheepsfoot Rollers Also known as "studded rollers," these typically are used in the compaction of cohesive soils. These rollers have a drum with several rounded or rectangular protrusions or feet and apply very high contact pressures to the soil layer being compacted. The vertical contact stress is dependent on the spacing of the protrusions on the drum and creates a kneading action that compacts the layer "bottom up." Once compaction is complete, the roller "walks out" of the lift, leaving the surface fairly rough. This kneading or shearing action maximizes a cohesive soil's strength at high density levels. Some sheepsfoot rollers are equipped with oscillatory vibrators to increase the effectiveness across a broader range of soil (Christopher et al. 2010). One variation of the sheepsfoot roller, known as the tamping foot roller, has feet with sloping sides. Because of the sloping nature of the feet, tamping foot rollers leave the compacted layer surface fairly smooth. Figure 80 is a photo of a sheepsfoot roller used to compact a low plasticity clayey silt.

Pneumatic or Rubber-Tire Rollers Pneumatic-tired rollers generally have two tandem axles with three to six wheels each. The wheels are arranged so that the rear ones will run in the spaces between the front ones, theoretically leaving no ruts. The weight of ballast carried by the equipment chassis can

FIGURE 80 Compaction of a low plasticity clayey silt (CL-ML) using a sheepsfoot roller.

be varied to achieve the required compactive energy. Sometimes the wheels are mounted slightly out of line with the axle, giving them a weaving action and the name "wobble wheel." This condition improves the kneading action on the layer being compacted. These rollers often are used as an alternative for compacting a variety of soil types and are particularly effective for noncohesive silty soils. Construction vehicles such as loaded dump trucks also can be used to serve as pneumatic rollers, especially during the placement of embankments. Pneumatic rollers compact the soil layers top-down, and the zone of influence is relatively shallow, particularly for small-tire units (Ingersoll-Rand 1984).

Impact Rollers Impact rollers comprise triangular ellipsoids or hexagonal drums to apply impact energy on to the layers being compacted as the roller moves along. Owing to the high impact energies being applied to the layer surface, these rollers achieve compaction at a faster rate and have a greater zone of influence compared with conventional smooth drum or sheepsfoot rollers. Although the use of impact rollers is common in Europe and South Africa, their availability in the United States is limited. Figure 81 is a photo of an impact roller.

Grid Rollers Grid rollers have a cylindrical heavy steel surface consisting of a network of steel bars forming a grid with square holes and may be ballasted with concrete blocks. Grid rollers provide high-contact pressure but little kneading action and are suitable for compacting most coarse grained soils (RDSO, 2005). Table 12 is borrowed from Christopher et al. (2010) and lists the compactor types for different soil types (Original source: Rollings and Rollings 1996).

Key Lesson

The use of roller types that are most suitable for the particular material types is critical to ensuring adequate compaction of unbound aggregate pavement layers.

FIGURE 81 Impact roller (http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/08.cfm).

Measuring In-Place Density of Constructed Unbound Aggregate Layers

Different methods exist for determining the moisture content and achieved density of constructed UAB and subbase layers. Some of these methods are listed in Table 13 along with the test methods governing their respective procedures.

Moisture Measurement

Soil moisture measurements are routinely conducted during pavement construction for QA purposes. State construction guidelines typically specify methods, such as oven or hot plate drying or nuclear density gauge testing. Under some circumstances, these methods may not be reliable, may require special licensing, and may be time consuming. Developing a performance-based specification that relates soil moisture to modulus requires that a rapid and reliable measure of unbound pavement material moisture content be developed. The key issues to consider are (1) type of moisture content measured (gravimetric or volumetric), (2) accuracy, (3) durability, (4) response time, and (5) ease of use.

TABLE 12
RECOMMENDED FIELD COMPACTION EQUIPMENT FOR DIFFERENT SOILS

Soil Type	First Choice	Second Choice	Comment
Rock fill	Vibratory	Pneumatic	—
Plastic soils, CH-MH (A-7, A-5)	Sheepsfoot or pad foot	Pneumatic	Thin lifts usually needed
Low-plasticity soils, CL, ML (A-6, A-4)	Sheepsfoot or pad foot	Pneumatic, vibratory	Moisture control often critical for silty soils
Plastic sands and gravels, GC, SC (A-2-6, A-2-7)	Vibratory, pneumatic	Pad foot	—
Silty sands and gravels SM, GM (A-3, A-2-4, A-2-5)	Vibratory	Pneumatic, pad foot	Moisture control often critical
Clean sand, SW, SP (A-1-b)	Vibratory	Impact, pneumatic	—
Clean gravels, GW, GP (A-1-a)	Vibratory	Pneumatic, impact, grid	Grid useful for over-sized particles

Source: Rollings and Rollings (1996).

TABLE 13
DIFFERENT METHODS TO DETERMINE THE MOISTURE DENSITY OF COMPACTED AGGREGATE BASE AND SUBBASE LAYERS IN THE FIELD

Parameter to Be Determined	Name of Method		ASTM	AASHTO
Moisture content	Gravimetric		D 2216	T 265
	Microwave		D 4643	N/A
	Calcium carbide gas pressure test		D 4944	T 217
Density	Sand cone		D 1556	T 191
	Sand Replacement		D4914	N/A
	Balloon		D 2167	T 205[a]
	Oil or water			
	Drive cylinder		D 2937	T 204[a]
Moisture and density	Rapid		D 5080	N/A
	Nuclear	Moisture	D 3017	T 310
		Density	D 2922	
	Time domain reflectometry		D 6780	N/A[a]

N/A: Not available
[a] Withdrawn from latest standards.

Direct Methods for Measuring Moisture Content The oven dry method, the microwave oven method, the direct heating method, and the calcium carbide gas pressure tester method ("speedy moisture content") are examples of methods used to make gravimetric moisture measurements during pavement construction. Oven dry and direct heating methods operate on the principle that the water mass is the difference between the weights of the wet and oven dry samples. The soil water content is expressed by weight as the ratio of the mass of water present to the dry weight of the soil sample. The field moisture oven has been used to measure moisture content during pavement construction (Camargo et al. 2006; White et al. 2009). The advantages of these devices are their ease of use and relative inexpensive cost. Possible drawbacks are that their use can be time consuming, may require a large power source in the field, and requires an accompanying density test to convert to volumetric water content.

Indirect Methods for Measuring Moisture Content Indirect methods for measuring volumetric water content rely on an empirically derived calibration with a measured variable such as dielectric permittivity. Dielectric methods have been used extensively for measuring soil water content in agricultural and geotechnical engineering applications. Time domain reflectometers, frequency domain reflectometers, and capacitance probes use the principles of the matrix dielectric permittivity to indirectly measure the volumetric moisture content in the soil. Several studies document the use of dielectric methods for measuring and monitoring pavement layer water content (Janoo et al. 1994; Rainwater and Yoder 1999; Roberson 2007). Specific field devices that have been used to measure water content during pavement construction include the Percometer, the TRIME-EZ probe, and the Trident Moisture Meter (Veenstra et al. 2005; Camargo et al. 2006). Recently, the DM600 Roadbed Meter was developed specifically for measuring water content of pavement materials; however, the device has not been extensively tested in the field. The advantages of the dielectric methods are fast equilibrium measurement times, relatively accurate measurements, easy automation, and the elimination of the need for a density measurement. Drawbacks of the method are the possibility of inaccuracies resulting from high clay content and soil salinity, lack of durability, soil-specific calibration may be needed for some instruments, and instruments can be relatively expensive.

Nuclear gauge-based moisture-density measurements have been in common use for transportation agencies for the last three decades. Commonly referred to as "nuclear density gauges," these devices can measure the wet density and moisture content of compacted soil and aggregate layers. The wet density of a layer is measured by detecting the suppression of gamma waves from a source rod lowered into the ground (direct transmission mode). In a second mode of operation (backscatter mode), the source rod is at the same level as the detector (not lowered into the pavement layer), and gamma rays from the source are "scattered back" from the compacted layer to the gauge. Note that the use of the backscatter mode usually is not recommended for determining the density of granular base/subbase layers because granular layers usually are porous, and the presence of large voids can significantly reduce the amount of gamma rays that get reflected back to be captured by the detector.

A nuclear density gauge monitors the moisture content of constructed pavement layers using a strong neutron source that emits neutrons into the surface. These neutrons are reflected upon colliding with the hydrogen atoms (similar in size to the neutrons) present in water. The amount of reflected neutrons detected by the gauge can be used to estimate the moisture content of the pavement layers. Specifications for determining the moisture content of soil and aggregate layers using nuclear density gauges are provided in AASHTO T 310 and ASTM D 3017.

CURRENT STATE OF THE PRACTICE

Based on the survey conducted, Figures 82 to 85 review transportation agency practices related to field compaction and construction QC of unbound aggregate layers. More than 75% of

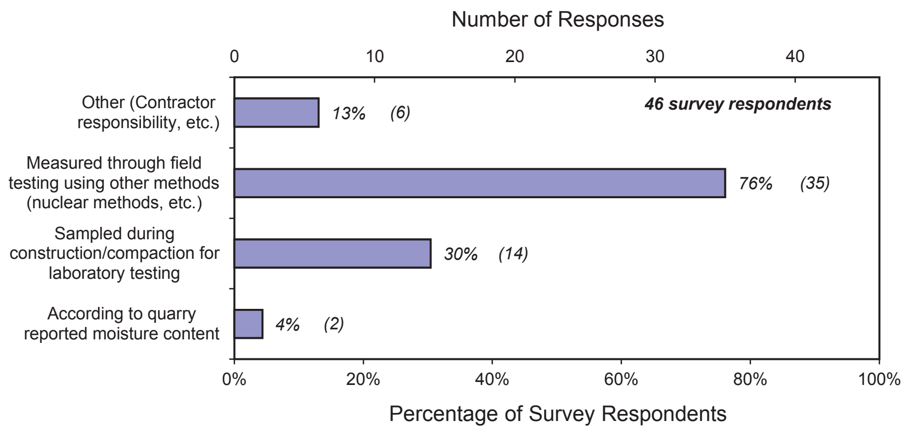


FIGURE 82 Different methods used by transportation agencies to control the moisture content of constructed/compacted UAB layers in the field.

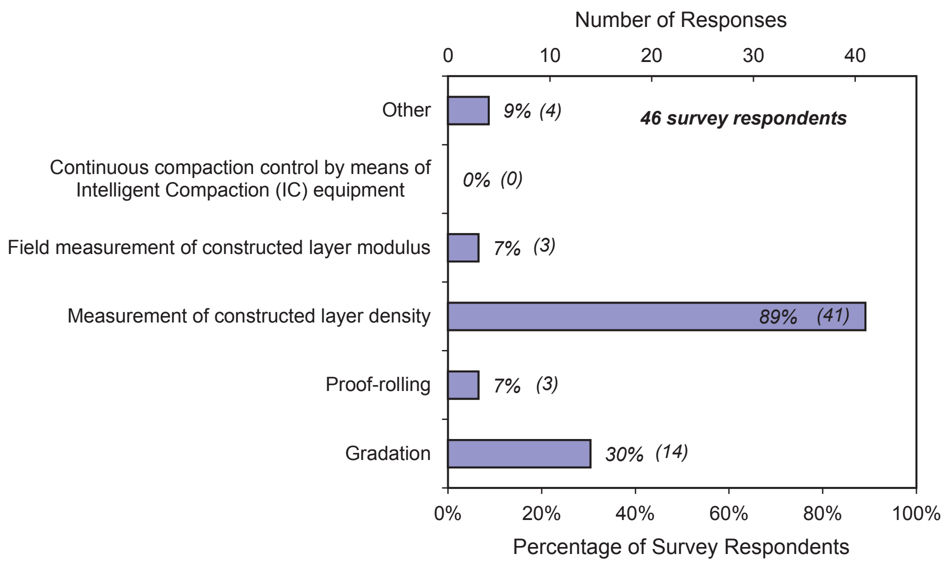


FIGURE 83 Primary approaches used by transportation agencies for evaluating degree of compaction and construction quality control of UAB/subbase layers.

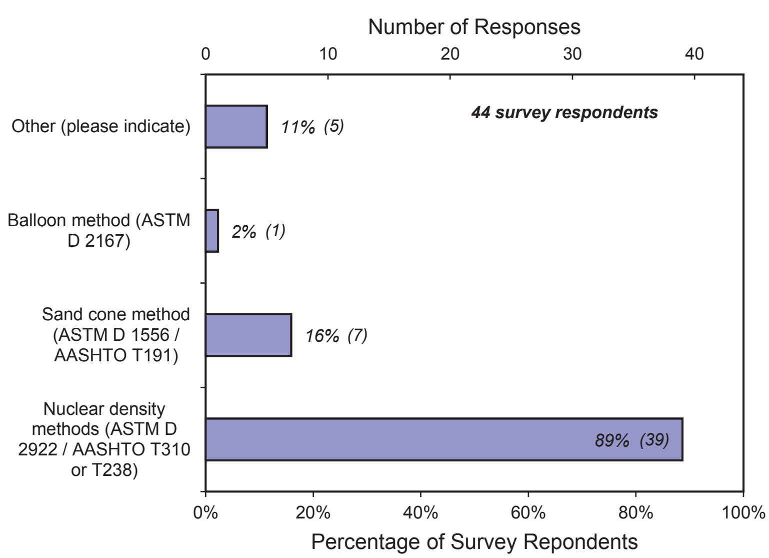


FIGURE 84 Methods commonly used by transportation agencies for measuring constructed aggregate base/subbase layer densities in the field.

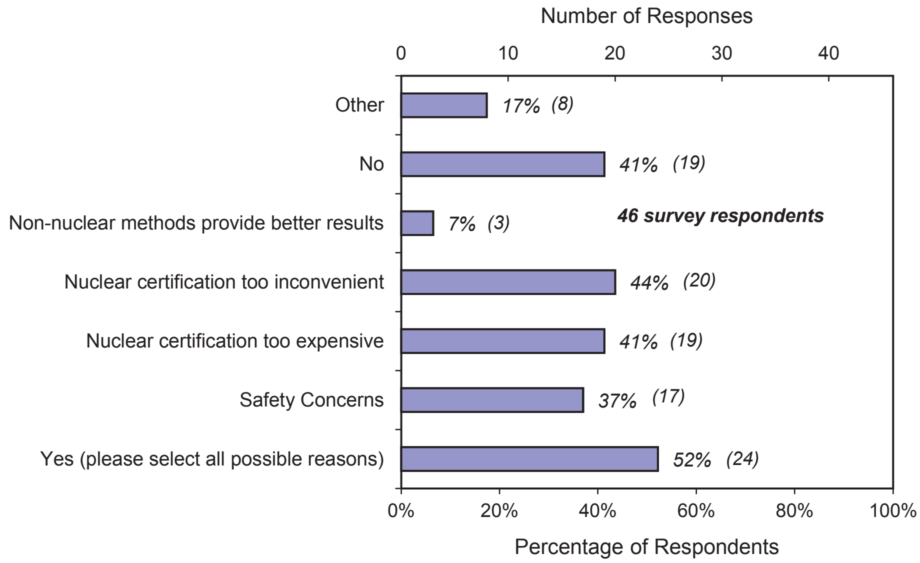


FIGURE 85 Responses to the question "Is there interest to implement nonnuclear density measurement methods for construction quality control of unbound aggregate base/subbase layers."

the responding agencies control moisture content of constructed/compacted UAB in the field (see Figure 82). Field moisture and density measurements using nuclear density gauges is a common practice in 89% of the responding agencies (see Figures 83 and 84). The in-place densities thus determined are compared with laboratory-established compaction characteristics to check the DOC achieved in constructed aggregate layers. Only 28% of the responding agencies construct test strips to establish roller patterns and check for compaction density growth of aggregate layers. More than 50% of the responding agencies expressed interest in implementing non-nuclear density measurement methods for construction control of UAB/subbase layers owing to one reason or another (see Figure 86). However, several of them indicated a lack of confidence in the performance of non-nuclear moisture-density measurement alternatives.

Key Lessons

- Measurement of compacted unbound aggregate layer density using a nuclear gauge is a common practice among transportation agencies.
- There is growing interest among agencies in gradually moving toward density measurement systems that are not nuclear based owing to certification and convenience issues associated with nuclear gauge testing.

IN-PLACE MODULUS MEASUREMENT OF CONSTRUCTED AGGREGATE LAYERS

Quality control and quality assurance (QC/QA) of constructed unbound aggregate pavement layers traditionally has been based on target density values, expressed with respect to the maximum achievable densities in the laboratory through commonly used compaction tests. Although research has successfully correlated higher densities to unbound aggregate layer stiffness or resilient modulus improvements (Rowshanzamir 1995; Tutumluer and Seyhan 1998), M-E pavement design methods do not consider aggregate layer density as an input into pavement thickness design. The resilient modulus, on the other hand, governs the nature of stress dissipation in an aggregate layer because of wheel load, and thus is an essential input for mechanistic analysis of the layered pavement structure. This alone has made the alternative of measuring in situ layer modulus attractive for pavement designers, although a challenging task now deals with how to develop related construction specifications for field modulus control.

Growing interest in modulus-based compaction control procedures has led to the development of several different alternatives for nondestructive field modulus measurements of pavement layers. Von Quintus et al. (2009) and Puppala (2008) present an extensive overview of different techniques and devices available for the measurement of in-place pavement layer moduli. The underlying techniques used for in-place modulus measurement of UAB and subbase layers are listed in Table 11 along with examples of devices based on the corresponding principles. Note that the devices listed in Table 11 all function based on different principles, so the reported values may have different dimensions. Some devices are based on the principle of measuring stiffness, whereas some measure modulus. It is important to note that "stiffness" is not an independent soil parameter and is dependent on the area over which the load is applied. However, "modulus" is truly an independent soil parameter and is independent of the compaction equipment. Thus, for true representation of compacted layer properties, a device should report the modulus value and not just the stiffness value (Briaud and Seo 2003).

Of the previously listed devices, the FWD is the most commonly used device by transportation agencies for indirectly measuring (or backcalculating from measured deflections) the in-service pavement layer moduli. The survey of state and Canadian transportation agencies conducted under the scope of the current synthesis study indicated that 27 of 46 responding agencies use FWD testing to assess the structural condition of UAB and subbase layers in existing pavement structures. However, although FWD testing on in-service pavement structures is a fairly common practice among transportation agencies, the use of an FWD device directly on top of UAB/subbase is a relatively new practice. For example, the UK performance-based specifications recommend the use of FWD to check the adequacy of constructed unbound aggregate layers (Interim Advice Note 2009). The most commonly adopted techniques for checking the quality of constructed unbound aggregate layers using in-place modulus measurements involve portable devices such as the LWD, GeoGauge, and surface seismic, or continuous measurement devices such as instrumented compactors. LWDs are used as primary field devices in several countries, including Germany, Austria, and Sweden, to measure earthwork stiffness/modulus. However, in the United States, only Indiana DOT uses LWD to measure the modulus of constructed unbound aggregate layers.

Several studies have been conducted focusing on the correlation between field measured stiffness/modulus to density, correlation between stiffness/modulus values reported by different devices, and repeatability of values reported by individual devices. (Puppala 2008; Von Quintus et al. 2009).

Chen et al. (1999) conducted field modulus/stiffness tests on different subgrade and base materials in more than six Texas districts and made the following observations:

- Field-measured density of constructed pavement layers was not sensitive to change in modulus.
- Both the soil stiffness gauge (Humboldt GeoGauge) and seismic techniques, such as Dirt-Seismic Pavement Analyzer and Olson-SASW, reported modulus values that were consistent with those reported by conventional FWD and showed promise for being used as QC devices,

Nazzal (2003) conducted extensive field testing to evaluate the potential of several NDT devices, such as the soil stiffness gauge (Humboldt GeoGauge), DCP, and LWD to measure the stiffness/strength parameters of highway materials and embankment soils during and after construction. A strong correlation was found between layer modulus values reported by LWD and GeoGauge-type devices and those measured from conventional FWD testing. Furthermore, higher coefficients of variation were reported to be associated with LWD-measured modulus values than were those measured by the GeoGauge, indicating the GeoGauge is a more "consistent" device (Nazzal 2003).

Von Quintus et al. (2009) reported that deflection-based methods such as LWD and FWD had limited potential for QC purposes. Testing several constructed pavement sections using different devices, they reported that deflection-based methods were not able to consistently identify areas with construction anomalies. Moreover, modulus values were influenced by the underlying layers, resulting in lower or higher and more variable modulus values.

More recently, Mishra et al. (2012) measured the field modulus values of full-scale unsurfaced pavement test sections using a Dynatest LWD (Model 3031) and a soil stiffness gauge (Humboldt GeoGauge). By measuring the layer moduli on top of the prepared subgrade as well as constructed unbound aggregate layers using both devices, they reported that both devices were capable of identifying anomalies in construction conditions. Higher modulus values were measured by the soil stiffness gauge compared with the LWD because of the relatively smaller magnitudes of strains imposed on the pavement layers by the soil stiffness gauge when compared with the LWD. Similar to the findings of Von Quintus et al. (2009), Mishra et al. (2012) reported that the LWD-measured modulus values were affected by layer thicknesses. Mooney and Miller (2009) also reported a depth of influence for LWD between 0.9 and 1.1 times the plate diameter, making it susceptible to the influences of underlying layers, especially for testing on thin aggregate layers.

The soil stiffness gauge was found to be more consistent in measuring field modulus values irrespective of constructed layer thicknesses. Although Von Quintus et al. (2009) reported a strong correlation between layer modulus values measured by a soil stiffness gauge and the achieved dry density values, Mishra et al. (2012) did not observe any such correlation from their testing.

Moreover, several research studies have focused on the "validity" of stiffness/modulus values reported by these devices with respect to the actual stress-strain states experienced by pavement layers under traffic loading. For example, Mooney and Miller (2009) measured the in situ stress and strain behavior during LWD testing and showed that the LWD test engages a nonlinear soil modulus.

Key Lessons

- Several research and implementation projects have reported different degrees of success with in-place modulus measurement devices.
- Although these devices have been used successfully to identify anomalies in construction conditions, extensive calibration for local materials is needed before they can be used as primary tools for QC.

MODULUS-BASED COMPACTION CONTROL

Need for Modulus-Based Compaction Control

Although the measurement of the dry unit weight and moisture content of constructed UAB/subbase layers is relatively straightforward and practical, it does not provide any direct indication about the layer modulus or shear strength. Moreover, it is important to note that the same density can be obtained for at least two different moisture contents on either side of the compaction (moisture-density) curve. Thus, it is not ideal to use the achieved dry density as the only criterion for compaction/construction QC. A modulus-based compaction control method combines the aspects of construction QC with in-place measurements of layer moduli.

Desired Characteristics of a Modulus-Based Compaction Control System

For developing a modulus-based construction specification, a few key issues must be properly considered, namely: (1) measurement depth, (2) induced stress state and stress path in relation to strength, and (3) use of proper algorithms for layer modulus estimation.

Ideally, a field technique would estimate the elastic modulus of the individual pavement layers separately to be consistent with how material is represented in the MEPDG. Note that field devices also may provide stiffness measurements of aggregate materials belonging to depths that are often inconsistent with layer thickness. This limitation for devices that measure deeper than the layer thickness can be overcome, as has been demonstrated in recent research by Senseney and Mooney (2010), who successfully extracted unbound layer moduli using the LWD with center position and radial offset sensors (similar to FWD). The approach is simple and robust for stiff-over-soft conditions (e.g., base over subbase, subbase over subgrade). If such techniques are not followed in the field, LWD moduli often will be dependent on depth of influence but not exactly on the layer thickness of the constructed aggregate base/subbase.

The way to control compaction is to ensure that the dry density is within tolerance from a target value, that the modulus is within tolerance from a target value, and that the water content is within tolerance of a target value. It is possible to achieve reasonable control of compaction by ensuring that two of these three properties are within tolerance of their target values. In that respect, it is possible to control compaction by ensuring that the soil modulus and the water content are within tolerance of their target values. Implementing the modulus-based compaction control is desirable, but it cannot be used readily in practice because of the lack of proper guidelines and because specifications have not been established. Future practice no doubt will bring a basic need for the engineer to check that his or her modulus design assumption is verified in the field.

CONTINUOUS COMPACTION CONTROL AND INTELLIGENT COMPACTION

Continuous compaction control (CCC) uses vibratory drum compactors, which combine the technologies of a global positioning system (GPS), compactor-integrated measurement system, and an onboard display of real-time compaction measurements (Chang et al. 2011). Integration of these components allows compaction data, also known as roller integrated compaction measurements (RICMs), to be tied to a specific project location, which is constantly updated as the compaction progresses. CCC typically involves the use of vertical drum acceleration processed in the time and/or frequency domains to assess the state of soil compaction. Early research in Sweden revealed that the vibration characteristics of the drum changed as the underlying soil was compacted (Thurner and Sandstrom 1980). For vibratory roller configurations, CCC involves measurement and analysis of output from an accelerometer mounted to the roller drum and can provide a spatial record of compaction quality when linked to position measurements and a documentation system (Chang et al. 2011). Roller measurement values calculated based on accelerometer measurements use one of two different approaches:

- Calculate a ratio of selected frequency harmonics for a set time interval, or
- Calculate ground stiffness or elastic modulus based on a drum-ground interaction model and some assumptions.

Continuous compaction control machines typically include the following (Peterson 2005):

- Sensors to measure vibration of the drum;
- Onboard electronics to record and process sensor output and record the stiffness;
- Linkages to the machine controls to adjust compaction effort according to the measured stiffness;
- Systems to record machine location; and
- Either local storage or wireless communications systems for data transfer.

IC differs from CCC by providing real-time, automatic adjustment of compactor settings based on RICM values to ensure maximum compactor efficiency as compaction progresses and soil properties change. The equipment adjustments based on RICM data generally involve modifying the eccentric mass moment with the drum(s) to affect excitation amplitude and frequency (Rinehart and Mooney 2008). Essentially, IC adds an additional feature over CCC by immediately interpreting RICMs and adjusting the compactor operating characteristics. A formal definition of IC has been given as:

> . . . the compaction of road materials, such as soils, aggregate bases, or asphalt pavement materials, using modern vibratory rollers equipped with an in situ measurement system and feedback control. (http://www.intelligentcompaction.com/)

NCHRP Project 21-09, Intelligent Soil Compaction Systems, listed the following as desirable features of an IC system: (1) continuous assessment of mechanistic soil properties (e.g., stiffness, modulus) through roller vibration monitoring; (2) automatic feedback control of vibration amplitude and frequency; and (3) an integrated global positioning system to provide a complete geographic information system-based record of the earthwork site.

Five different types of RICMs [also known as intelligent compaction measurement values (ICMVs)] are used by commonly available compaction equipment and typically vary from one equipment manufacturer to another. These five measurements can be broadly divided into three different theories. Compaction meter value (CMV) and compaction control value are derived from amplitudes of the operating frequency and various harmonics and subharmonics. Roller-integrated stiffness (k_s or k_b) and vibration modulus (E_{vib}) are based on measuring the soil displacement under a compactor-generated load. Finally, machine drive power, the relative newcomer to the group, is based on measuring the amount of power needed to propel the compactor over the soil. Chang et al. (2011) presents a summary of the ICMVs in common use in the United States; these ICMVs are based on vibration frequency analysis or mechanical modeling (see Table 14).

Need for Intelligent Compaction

Intelligent compaction using CCC provides continuous data indicating the level of compaction achieved with every pass. Real-time processing of the data enables the equipment operating characteristics to be changed frequently, thus imparting variable compactive energy levels to different spots as needed. This spontaneous adjustment of compactive effort has been found to be particularly important during the compaction of thick-lift aggregate layers. Evaluation of thick aggregate base materials in the United States has produced evidence to confirm the usefulness of this feature. This reduces the spatial variability associated with the DOC achieved in a given pavement layer. Continuous data collection and processing also eliminates the need for frequent "spot testing" for quality assurance process. Applying the optimum number of passes of the roller, an IC system significantly reduces the chances of over-compaction. Through comparison of data obtained from consecutive roller passes, an IC system can quickly identify "difficult to compact" areas, thus enabling field engineers to make decisions to remedy the problem. Moreover, continuous monitoring and compaction control can significantly reduce differential settlements that result from nonuniform compaction conditions in projects that rely solely on spot tests for QC.

Finally, the level of compaction information gathered from rollers during the IC process is a better indicator of achieved compaction levels. This is primarily because of the significantly larger influence zones under a compactor compared with those corresponding to spot-testing equipment such as FWD, LWD, soil stiffness gauge, nuclear density gauge, or DCP. Chang et al. (2011) compares the influence zone under a roller to those under commonly used spot-testing devices (see Figure 86).

In addition to the previously mentioned advantages, the following disadvantages of IC systems have been reported by researchers (Briaud and Seo 2003):

1. Requirement for sophisticated equipment in a rugged environment;
2. Requirement for operator training; and
3. More expensive than conventional compaction (may require an overall cost-benefit study).

TABLE 14
DIFFERENT METHODS AVAILABLE FOR IN-SITU MODULUS MEASUREMENT OF CONSTRUCTED PAVEMENT LAYERS

Test Category	Underlying Principle	Corresponding Devices
Surface deformation	Static load	• Benkelman beam • Briaud compaction device (based on measuring the bending strain on a loading plate in contact with the ground)
	Steady state vibratory	• Soil stiffness gauge (e.g., Humboldt GeoGauge)
	Impact load	• Falling weight deflectometer (FWD) • Portable falling weight deflectometer or light weight deflectometer (LWD)
	Sinusoidal load	• Dynaflect • Road rater
	Continuous load	• Rolling wheel deflectometer
Geophysical	Wave propagation	• Ultrasonic body waves • Ultrasonic surface waves • Spectral analysis of surface waves (SASW) • Multichannel analysis of surface waves • Free-free resonant column tests • Seismic pavement analyzer • Portable seismic pavement analyzer

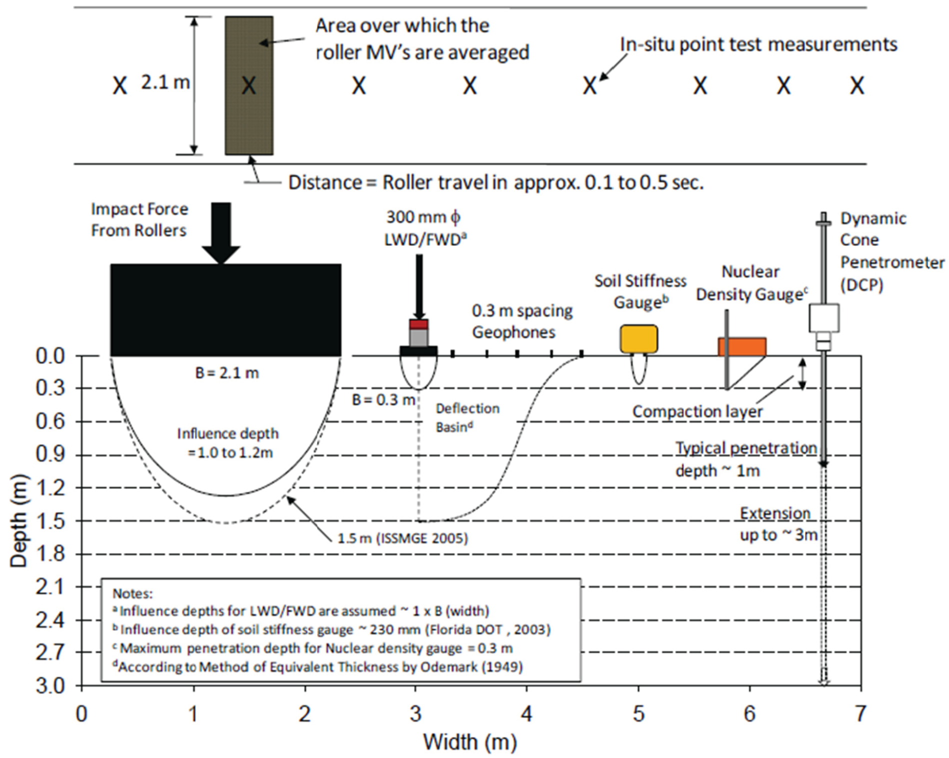


FIGURE 86 Illustration of differences in measurement influence depths for different measurements (Chang et al. 2011).

Synthesis of Past Research and Agency Experience with IC Systems

Several research and trial projects have been conducted in the United States evaluating the application of IC systems as QC tools for UAB/subbase layer construction. Most notably, a Transportation Pooled Fund project, TPF-5(128) was conducted from 2008 to 2011 involving 12 participating state transportation agencies (Georgia, Indiana, Kansas, Maryland, Minnesota, Mississippi, New York, North Dakota, Pennsylvania, Texas, Virginia, and Wisconsin). The primary objective of the study was to develop an IC expertise base, evaluate current IC equipment, and accelerate specification development. The following section presents a summary of some of the most notable findings from IC implementation studies in the United States.

Minnesota Experience

In 2005, Minnesota DOT used the MnROAD test track to demonstrate the Bomag system and other subgrade soil and aggregate base/subbase layer testing devices, including DCP, GeoGauge, and Light Weight Deflectometer (LWD), to determine the relationship between the IC roller response output and independently measured soil properties (Peterson 2005). In general, the demonstration project concluded that CCC was an effective QC mechanism for soil compaction.

Camargo et al. (2006) reported on a Minnesota case history involving IC equipment manufactured by Ammann, Bomag, and Caterpillar. Through compaction using IC equipment and spot-testing using QA devices such as DCP, LWD, and GeoGauge, it was observed that there was no significant difference between the modulus measurements obtained from QA devices such as LWD or GeoGauge and the Bomag IC. Camargo et al. also highlighted the challenges associated with handling the massive amounts of data generated by IC equipment before IC specifications can be implemented for use by transportation agencies.

Texas Experience

Under the pooled fund study TPF-5(128), a field IC demonstration was performed in Fort Worth, Texas, in 2008. The IC equipment used was Case/Ammann single-drum padfoot and smooth drum vibratory rollers and the Dynapac single-drum smooth drum vibratory roller. Using IC technology to compact cohesive subgrade and granular base layers, it was observed that in situ measurements using the calibrated moisture-density nuclear gauge, DCP, and LWD did not match well with those of the ICMVs. However, plate loading tests (PLTs) and FWD tests produced better correlation with the ICMVs (Chang et al. 2011) (see Table 15).

NCHRP Project 10-65

NCHRP Project 10-65 (Von Quintus et al. 2009) used several different IC rollers (Bomag, Caterpillar, Case/Ammann) and

TABLE 15
SUMMARY OF IC MEASUREMENTS

IC Measurement	Units	IC System	Model Definition
Compaction meter value (CMV)	None	Caterpillar Dynapac	$CMV = C\frac{A_{2\Omega}}{A_{\Omega}}$
Machine drive power	None	Caterpillar	$MDP = P_g - Wv\left(Sin\alpha + \frac{A'}{g}\right) - (mv+b)$
Compaction control values	None	Sakai	$CCV = \left[\frac{A_{0.5\Omega} + A_{1.5\Omega} + A_{2\Omega} + A_{2.5\Omega} + A_{3\Omega}}{A_{0.5\Omega} + A_{\Omega}}\right] \times 100$
Stiffness (K_b)	MN/m	Ammann/ Case	$k_b = \omega^2\left[m_d + \frac{m_0 e_0 \cos\phi}{z_d}\right]$
Vibration modulus (E_{vib})	MN/m^2	Bomag	$\frac{\Delta F}{\Delta z_1} = \frac{E_{vib}\cdot 2\cdot a\cdot\pi}{2\cdot(1-\nu^2)\cdot\left(2.14 + 0.5\cdot\ln\left(\frac{\pi\cdot(2\cdot a)^3\cdot E_{vib}}{(1-\nu^2)\cdot 16\cdot(m_b + m_e + m_\gamma)\cdot g\cdot\left(d/2\right)}\right)\right)}$

Source: Chang et al. (2011).

an instrumented vibratory roller for the QC/QA of HMA mixtures and unbound pavement layers. Through comparison of the IC response output parameters with modulus and density values measured using traditional as well as nondestructive testing devices, it was observed that the IC equipment was successful in detecting areas with significant density differences. Moreover, the IC equipment output was found to correlate with other nondestructive testing density and modulus values (Von Quintus et al. 2009).

In summary, nearly all previous studies have concluded that the use of IC rollers has many advantages for use as a contractor's QC tool to monitor the compaction of pavement materials and identify soft spots or weak areas along a project. Most studies have focused on the effect of increasing material compaction or density on the IC measured response and have reported good correlations between the IC output and density modulus for a specific material and project. Fewer studies have focused on the effect of temperature, moisture, material condition, and varying subsurface conditions on the responses and output from the IC measurement systems in terms of reducing the risk of making an incorrect decision during construction. Temperature of HMA, moisture content of unbound layers, and support conditions of the underlying layers are important factors related to the IC roller's output.

NCHRP Project 21-09

NCHRP Project 21-09, "Intelligent Soil Compaction Systems," evaluated the reliability of different IC measurement systems and developed construction specifications for the compaction of subgrades, embankments, and UAB/subbase layers. Upon investigation of four vibration-based roller measurement values (MVs); (Ammann and Case/Ammann k_s, Bomag E_{vib}, Dynapac CMV_D, and Sakai continuous compaction value), the study confirmed the dependence of roller MV on the amplitude and frequency of roller vibration, and thus recommended construction specifications that allow IC during compaction but do not permit its use during roller-based QA. The construction specifications developed through this project were grouped into the following three categories (Mooney et al. 2010):

1. **Option 1:** This option uses CCC to identify weak spots in a compacted area to be further tested using commonly used spot tests.
2. **Option 2:** This option is based on statistical change in the roller MV during compaction. It can be based on monitoring the difference between mean roller MV from one pass to the other or on the percentage change in spatial roller MV. Note that neither of these options requires the calibration of roller MV using test strips.
3. **Option 3:** This option was further subdivided into alternatives that required the calibration of roller MV with spot testing results and thus involves significant initial investments. Detailed discussion of these alternative specifications and the challenges associated with the implementation of each can be found elsewhere (Mooney et al. 2010).

Wisconsin Experience

Von Quintus et al. (2010) collected information and data on the use of IC technology to help Wisconsin DOT assess the validity and accuracy of IC in pavement construction. Through data collection from demonstration projects, they identified the following two usage areas as more mature and ready to have immediate positive benefits, especially for unbound materials: (1) use of IC rollers as a testing device to identify areas with weak supporting areas through continuous mapping of the stiffness, and (2) development of stiffness-growth relationships to determine the rolling pattern and number of passes to achieve a specific stiffness level. They also recommended additional pilot projects to increase contractor and agency personnel's confidence in using the IC technology.

Details on several other IC implementation projects were provided in Chang et al. (2011). Overall, all the IC implementation studies have shown promising results regarding the potential of this technology to be used for QC purposes.

Quality Assurance Specifications Based on Continuous Compaction Control

White et al. (2007) conducted three field studies to investigate the correlation of CMV (also known as Caterpillar compaction value) and machine drive power values from Caterpillar rollers and k_B stiffness from Ammann rollers with in situ test measurements such as dry unit weight, DCP index, Clegg Impact Value, and LWD modulus and made the following observations:

- The Ammann k_B value showed a strong correlation with in situ test results for strips with a relatively wide range of material stiffnesses and a relatively weak correlation for strips with more uniform conditions. White et al. were also able to correlate the Ammann k_B with rut depth measured after test rolling procedures.
- IC technology could be successfully applied by the MnDOT as the principal QC tool on a grading project near Akeley, Minnesota. The entire project passed the test rolling acceptance criteria.

Figure 87 shows the relationships between average in situ properties and RICM values as reported by White and Thompson (2008).

Rinehart et al. (2009) compared the in situ stress states and stress paths experienced by a soil beneath two IC rollers on instrumented vertically homogeneous embankment soil and on layered base over subgrade to the stress states applied during AASHTO T 307 resilient modulus testing. Measuring the stress states to a depth of 1 m below the roller wheel, they observed that stress fields varied significantly with depth for the homogeneous embankment and

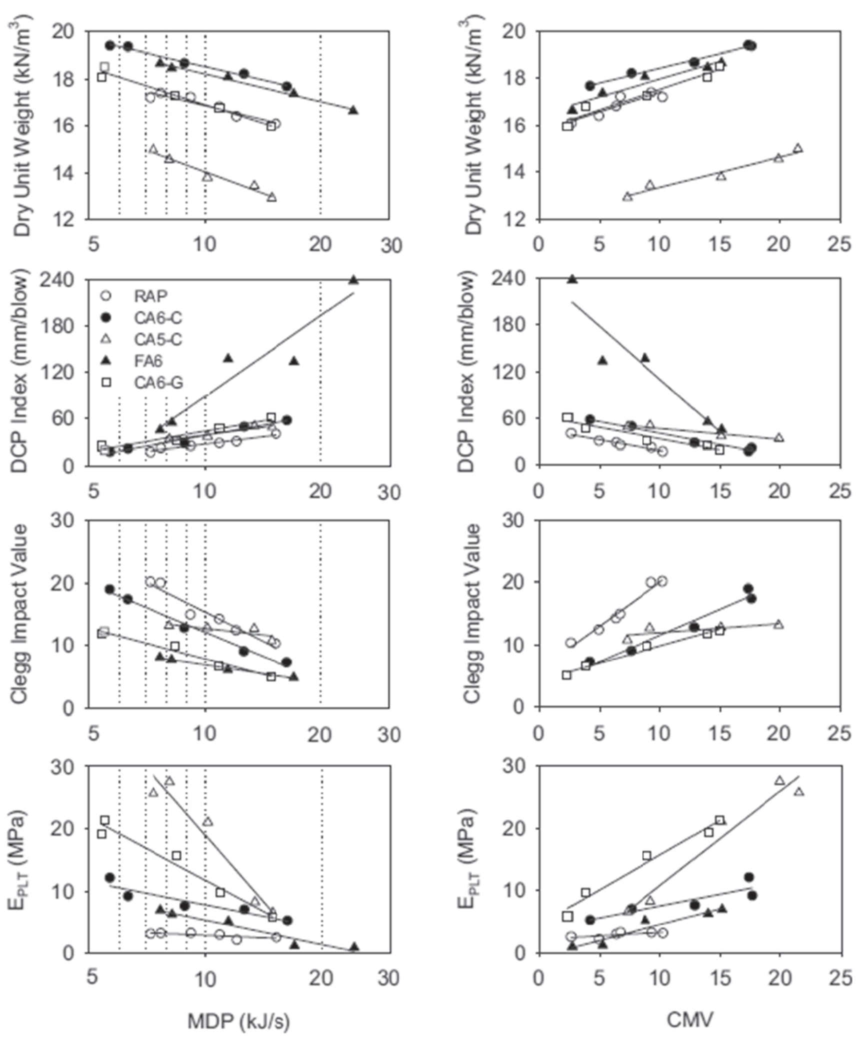


FIGURE 87 Relationships between average in situ and RICM values (White and Thompson 2008).

the layered base over subgrade. The measured deviator stress (q) and mean stress (p) were observed to decrease by factors of 4 and 6, respectively, within the 300-mm thick crushed base. Even for low excitation forces applied by the rollers, the estimated q values in the crushed base were as much as 2.5 times greater than the maximum q values applied during resilient modulus testing in the laboratory. Mean stress (p) values observed in the field were approximately 30% to 50% of the values applied during M_R testing in the laboratory.

Rinehart et al. (2009) also highlighted that the roller-based stiffness values determined during IC of layered constructions, such as pavement base over subgrade, are complex functions of material properties, layer thicknesses, and stress-dependent modulus parameters. Thus, any modulus-based compaction control protocol would be able to extract individual layer properties from these complex stiffness values. This task may become simpler in the case of homogeneous embankments with homogeneous modulus fields with depth.

Current Specifications Based on Roller Integrated Compaction Measurements

Specifications for IC were first introduced in Austria in 1990 (later modified in 1993 and 1999), Germany in 1994 (updated in 1997), Sweden in 1994 (revised in 2005), and Switzerland in 2006. The International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) adopted the Austrian specifications in 2005. In the United States, a pilot specification was developed by MnDOT in 2007 and updated in 2010. Similarly, a special specification was developed by Texas DOT in 2008, and a list of approved IC rollers was released in 2009. In July 2011, FHWA released generic specifications for compaction of soils and subbases using IC techniques. These generic specifications are to be modified by individual agencies to meet specific requirements (www.intelligent compaction.com).

The current IC specifications follow one of the following two approaches (www.intelligentcompaction.com):

- Use of RICM values to identify weak spots, which can then be assessed by spot-testing techniques, such as moisture-density tests, PLTs, and/or LWD tests. Acceptance of the constructed layer is dependent on these “weak” spots satisfying minimum thresholds with respect to PLT modulus, LWD modulus, or density. This is the only approach permitted in Sweden.
- Use of test beds to develop correlations between MVs to PLT modulus, LWD modulus, or density in a defined calibration area. If a suitable correlation is found to exist, a target roller MV is determined from the correlation and used for QA purposes.

The Austrian/ISSMGE and German specifications each permit either of the two previously mentioned alternatives for construction QA using continuous compaction control. Based on a survey of European practice, Mooney et al. (2010) reported that implementation of the calibration approach was challenging and required a high level of on-site knowledge.

Ongoing Effort: NCHRP Project 10-84

With an objective to develop modulus-based compaction control specification in the United States, a research study funded by NCHRP is being undertaken. The developed specification shall (http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2908):

1. Be based on field measures of the stiffness or modulus and moisture content of the compacted earthwork and unbound aggregate that can be correlated with design modulus values;
2. Provide a single, straightforward, and well-defined method for determining stiffness or modulus that is compatible with a variety of earthwork and unbound aggregate design methodologies;
3. Directly account for the seasonal variation of the modulus of the compacted earthwork or unbound aggregate as the means to determine specification criteria and limits for compaction;
4. Use available models, devices, and methods, as defined in the current literature, including *NCHRP Synthesis 382: Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design* (Puppala 2008); and
5. Be founded on a comprehensive review of the current literature on the long-term behavior of various soils and unbound aggregates in terms of the principles of unsaturated soil mechanics.

The research work under the scope of this project will be conducted in three phases that have been subdivided into a total of 11 tasks. The project is currently in Phase III, with interim reports for Phases I and II already submitted to NCHRP.

Current State of the Practice

From the survey of state and Canadian provincial agencies conducted under the scope of this synthesis, it was observed that only one agency (Texas) has actively implemented IC techniques to construct in-service pavements with UAB/subbase layers and has developed a specification for this purpose. Two agencies (Indiana and Georgia) implement modulus-based compaction control for the construction of UAB/subbase layers but only in demonstration projects. Indiana DOT reported the use of LWDs for field modulus measurement in demonstration projects.

Key Lessons

- Continuous compaction control using different roller measurement values can significantly reduce spatial variability in compaction levels and can reduce the potential for differential settlements in constructed pavement layers.
- Most research and implementation projects conducted in the United States involving the use of continuous compaction control and IC to construct UAB/subbase layers have reported considerable success. However, such practices are not common for transportation agencies. Encouraging more implementation projects across agencies can help to incorporate continuous compaction control and IC into agency practice.
- Target relative stiffness values established during continuous compaction control vary significantly from one compactor to another. For example, the compaction measurement value established using a double-drum IC roller is significantly different than that established using a single-drum IC roller.

CONSIDERATION OF SUCTION EFFECTS IN LAYER MODULUS ESTIMATION

Background

The modulus (and, correspondingly, the load response linked to performance) of earthwork and unbound aggregates is strongly influenced by the seasonal variation of their moisture content. This variation depends on material composition, DOC, and available free moisture, which is controlled primarily by the local climatic environment and the distance from the GWT. In developing a modulus-based construction specification for compaction of earthwork and unbound aggregate that will provide a direct link with design parameters, all three factors—material, compaction, and moisture—should be examined on the basis of the principles of unsaturated soil mechanics with respect to highway engineering and construction.

Unbound aggregate pavement layers are usually compacted at moisture contents corresponding to 80% to 90% saturation conditions (Gupta et al. 2007) and thus fall in the unsaturated regime. The unsaturated conditions and distribution of pore structure within the compacted aggregate layers lead to the development of negative pore water pressure (matric suction), which increases the effective stress states within the layers. This increase in the overall stress states may have a significant influence on the shear strength and stiffness (or modulus) of stress-dependent unbound aggregate materials. Moreover, suction conditions significantly affect soil volume change, the coefficient of permeability, and freeze-thaw susceptibility. Thus, a suitable procedure for evaluating constructed aggregate layer conditions includes the effects of matric suction.

Within an unsaturated soil mechanics framework, soil suction can be represented as an independent stress state variable, and resilient modulus (M_R) can be represented as shown in Equation 18 (Gupta et al. 2007):

$$M_R = \left(k_1 p_a \left(\frac{\sigma_b - 3k_6}{p_a} \right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + k_7 \right)^{k_3} \right) + \alpha_1 \psi_m^{\beta_1} \qquad (18)$$

where σ_b is the summation of all around or bulk stress; τ_{oct} is the octahedral shear stress, ψ_m is the matric suction, α_1 and β_1 are empirical fit parameters relating the resilient modulus (M_R) to ψ_m; p_a is the atmospheric (normalizing) pressure; and k_1, k_2, k_3, k_6, and k_7 are model parameters obtained from regression analyses.

A modulus-based QA specification includes field moisture target values to ensure permanent deformation in the field remains below the allowable limit. A modulus specification based on unsaturated soil mechanics uses the volumetric moisture content (θ) instead of the gravimetric value (w) (Gupta et al. 2007). A modulus-based compaction control specification incorporating soil suction effects also incorporates methods to measure the soil suction of constructed pavement layers. In addition, the detrimental effects of soil wetting and the resulting loss of suction with changes in aggregate layer capillary structure should be considered in a comprehensive modulus-based QA specification.

Methods for Measuring Soil Suction

Different methods available to measure soil suction in the field can be broadly divided into two categories: (1) direct methods and (2) indirect methods (Lu and Likos 2004; Munoz-Carpena 2009). Oven dry methods and the calcium carbide gas pressure tester method ("speedy moisture content") provide a direct measure of the gravimetric moisture content (w), whereas indirect methods for measuring θ, such as time domain reflectometry, rely on an empirically derived calibration with a measured variable, such as dielectric permittivity. Although many of the available devices have been used in pavement research, an evaluation of such devices for routine field use and specifically for use in the development of a performance-based construction specification is still needed.

Indirect methods for soil suction measurement include thermal conductivity methods and the filter paper method. Thermal conductivity methods have been used in pavement engineering research to characterize soil suction in the base and subgrade layers (Nichol et al. 2003; Roberson 2007). The measurement of soil suction is based on the theory that thermal conductivity properties of a soil are indicative of the soil water content. Soil suction is inferred by measuring the dissipation of heat within the sensor, which is related to the water content of the sensor that is in equilibrium with soil water (Roberson and Reece 1993; Reece 1996). The advantages of the thermal conductivity method are that it has a wide measurement range, is easily automated, and is not affected by salinity. The limitations include hysteresis, individual sensor calibration, and long equilibrium times.

Filter Paper Methods

Both the contact and noncontact filter paper methods are used to indirectly measure soil suction by measuring the amount of moisture transferred from a soil sample to a calibrated piece of filter paper. The filter paper is placed in direct contact with a soil specimen or is suspended (noncontact) over the soil specimen. Once equilibrium between the soil sample and the filter paper is reached, the water content of the filter paper is determined gravimetrically and related to the soil suction by means of a calibration curve particular to the type of filter paper (Lu and Likos 2004). The filter paper method is simple and relatively inexpensive. Drawbacks of the method are long equilibration times (6 to 10 days).

Key Lesson

Suction effects and resulting changes in aggregate layer modulus should be considered during the design of UAB/subbase layers.

REFERENCES

Anday, M.C. and C.S. Hughes, "Compaction Control of Granular Base Course Materials by use of Nuclear Devices and a Control Strip Technique," *Highway Research Record 177,* Transportation Research Board, National Research Council, Washington, D.C., 1967, pp. 136–143.

Briaud, J.L. and J. Seo, *Intelligent Compaction Overview and Research Needs,* Report to the Federal Highway Administration, Washington, D.C., Texas A&M University, College Station, Tex., 2003.

Briaud, J.L. and K.Y. Rhee, *The BCD: A New Instrumentation for Compaction Control,* Final Report for IDEA Project 118, Transportation Research Board of the National Academies, Washington, D.C., 2009.

Buchanan, S., Inverted Pavements-What, Why, and How? AFTRE Industry Education Webinar, Aggregates Foundation for Technology, Research, and Education, Alexandria, Va., June 1, 2010.

Camargo, F., B. Larsen, B. Chadbourn, R. Roberson, and J. Siekmeier, "Intelligent Compaction: A Minnesota Case History," 54th Annual University of Minnesota Geotechnical Conference, Feb. 17, 2006 [Online]. Available: http://www.intelligentcompaction.com/downloads/Papers Reports/MnDOT_Camargo_IC%20A%20Minnesota%20 Case%20History_2006.pdf.

Chang, G., Q. Xu, J. Rutledge, B. Horan, L. Michael, D. White, and P. Vennapusa, *Accelerated Implementation of Intelligent Compaction Technology for Embankment Subgrade Soils, Aggregate Base, and Asphalt Pavement Materials,* Federal Highway Administration, Washington, D.C., 2011.

Chen, D.H., W.W. Wu, R. He, J. Bilyeu, and M. Arrelano, "Evaluation of In situ Resilient Modulus Testing Techniques," *Geotechnical Special Publication, No. 89,* 1999, pp. 1–11.

Christopher, B.R., C. Schwartz, and R. Boudreau, *Geotechnical Aspects of Pavements,* Report FHWA NHI-10-092, National Highway Institute, Federal Highway Administration, Washington, D.C., 2010.

Gupta, S.C., A. Ranaivoson, T. Edil, C. Benson, and A. Sawangsuriya, *Pavement Design Using Unsaturated Soil Technology,* MN/RC-2007-1, Minnesota Department of Transportation, St. Paul, 2007.

Gupta, S.C., A. Singh, and A. Ranaivoson, *Moisture Retention Characteristics of Base and Subbase Materials,* Report submitted to the Minnesota Department of Transportation, St. Paul, Dec., 2004, p. 47.

Holtz, R.D., *Guide of Earthwork Construction,* State of the Art Report 8, Transportation Research Board, National Research Council, Washington, D.C., 1990.

Ingersoll-Rand, *Compaction Data Handbook,* Ingersoll-Rand Construction and Mining, 1984.

Interim Advice Note 73/06 Revision 1 (2009): Design Guidance for Road Pavement Foundations (Draft HD25), Department of Transport, London, U.K., 2009.

Lu, N. and W. Likos, *Unsaturated Soil Mechanics,* Wiley and Sons Inc., Hoboken, N.J., 2004, pp. 294–295.

Marek, C.R. and T.R. Jones, Jr., "Compaction-An Essential Ingredient for Good Base Performance," *Proceedings from Conference on Utilization of Graded Aggregate Base in Flexible Pavements,* Sponsored by National Crushed Stone Association, National Sand and Gravel Association, and National Slag Association, Mar. 25–26, 1974, Oak Brook, Ill.

Mishra, D., E. Tutumluer, M. Moaveni, and Y. Xiao, "Laboratory and Field Measured Moduli of Unsurfaced Pavements on Weak Subgrade," *Proceedings of the ASCE Geo-Institute GeoCongess,* State of the Art and Practice in Geotechnical Engineering, Geotechnical Special Publication No. 225, CD-ROM, R.D. Hryciw, A. Athanasopoulos-Zekkos, and N. Yesiller, Eds., Oakland, Calif., Mar. 25–29, 2012.

Mooney, M.A. and D. Adam, "Vibratory Roller Integrated Measurement of Earthwork Compaction: An Overview," *Proceedings,* FMGM2007—International Symposium on Field Measurements in Geomechanics, Sep. 24–27, 2007, Boston, Mass.

Mooney, M.A. and P.K. Miller, "Analysis of Lightweight Deflectometer Test Based on In situ Stress and Strain Response," *Journal of Geotechnical & Geoenvironmental Engineering,* ASCE, Vol. 135, No. 2, 2009, pp. 199–208.

Mooney, M.A., R.V. Rinehart, N.W. Facas, O.M. Musimbi, D.J. White, and P.K.R. Vennapusa, *NCHRP Report 676: Intelligent Soil Compaction Systems,* Transportation Research Board of the National Academies, Washington, D.C., 2010.

Munoz-Carpena, R., "Field Devices for Monitoring Soil Water Content," University of Florida IFAS Extension, 2009.

Nazzal, M., *Field Evaluation of In situ Test Technology for QC/QA Procedures during Construction of Pavement Layers and Embankments,* MS Thesis, Louisiana State University, Baton Rouge, 2003.

Nichol, C., L. Smith, and R. Beckie, "Long-term Measurement of Matric Suction Using Thermal Conductivity Sensors," *Canadian Geotechnical Journal,* Vol. 40, 2003, pp. 587–597.

Peterson, D.L., *Continuous Compaction Control: MnRoad Demonstration,* Report No. MN/RC-2005-07, Office of Materials and Road Research, Minnesota Department of Transportation, Maplewood, 2005.

Proctor, R.R., "Fundamental Principles of Soil Compaction," *Engineering News Record,* Vol. 7, Aug.–Sep. 1933.

Puppala, A.J., *NCHRP Synthesis 382: Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design,* Transportation Research Board of the National Academies, Washington, D.C., 2008, 137 pp.

Research Designs and Standards Organization (RDSO), *Study Report on Compaction Equipment and Construction Machinery,* Report No. GE-R-76, Geotechnical Engineering Directorate, RDSO, Manak Nagar, Lucknow, India, 2005.

Reece, C.F., "Evaluation of a Line Heat Dissipation Sensor for Measuring Soil Matric Potential," *Soil Science Society of America Journal,* Vol. 60, 1996, pp. 1022–1028.

Richter, C.A., Seasonal Variations in the Moduli of Unbound Pavement Layers, Publication No. FHWA-HRT-04-079, Federal Highway Administration, Washington, D.C., 2006.

Rinehart, R.V. and M.A. Mooney, "Instrumentation of a Roller Compactor to Monitor Vibration Behavior during Earthwork Compaction," *Automation in Construction,* No. 17, 2008, pp. 144–150.

Rinehart, R.V. and M.A. Mooney, "Measurement of Roller Compactor Induced Triaxial Soil Stresses and Strains," *Geotechnical Testing Journal,* ASTM, Vol. 32, No. 4, 2009.

Rinehart, R.V., M.A. Mooney, and J.R. Berger, "Comparison of Stress States and Paths: Vibratory Roller-Measured Soil Stiffness and Resilient Modulus Testing," *Transportation Research Record: Journal of the Transportation Research Board, No. 2116,* Transportation Research Board of the National Academies, Washington, D.C., 2009, pp. 8–15.

Rinehart, R.V. and M.A. Mooney, "Measurement Depth of Vibratory Roller-Measured Soil Stiffness," *Géotechnique,* Vol. 59, No. 7, 2009, pp. 609–619.

Roberson, R.L. and C.F. Reece, "Comparison of Heat Dissipation Sensor, Tensiometer, and Thermocouple Psychrometer Response to Changing Soil Moisture Conditions," Poster/Abstract 85th Annual Meeting Soil Science Society of America, 1993.

Roberson, R., *Impact of Pavement Drainage Design on Material Stiffness,* M.S. Thesis, University of Minnesota, 2007.

Rollings, M.P., and R.S. Rollings, *Geotechnical Materials in Construction,* McGraw–Hill, N.Y., 1996.

Rowshanzamir, M.A., *Resilient Cross Anisotropic Behavior of Granular Base Materials under Repetitive Loading,* Ph.D. Dissertation, University of New South Wales, Australia, 1995.

Seed, H.B., "A Modern Approach to Soil Compaction," *Proceedings of the Eleventh California Street and Highway Conference,* Institute of Transportation and Traffic Engineering, University of California, 1959, pp. 77–93.

Senseney, C.T. and M.A. Mooney, "Characterization of Two-Layer Soil System Using a Lightweight Deflectometer with Radial Sensors," *Transportation Research Record: Journal of the Transportation Research Board, No. 2186,* Transportation Research Board of the National Academies, Washington, D.C., 2010, pp. 21–28.

Thurner, H.F. and A. Sandstrom, "Continuous Compaction Control, CCC," *Proceedings of the International Conference on Compaction,* Paris, France, 1980, pp. 237–245.

Tutumluer, E. and U. Seyhan, "Neural Network Modeling of Anisotropic Aggregate Behavior from Repeated Load Triaxial Tests," *Transportation Research Record 1615,* Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 86–93.

Von Quintus, H.L., C. Rao, R.E. Minchin, S. Nazarian, K.R. Maser, and B. Prowell, *NCHRP Report 626: NDT Technology for Quality Assurance of HMA Pavement Construction,* Transportation Research Board of the National Academies, Washington, D.C., 2009.

Von Quintus, H.L., C. Rao, H. Titi, B. Bhattacharya, and R. English, *Evaluation of Intelligent Compaction Technology for Densification of Roadway Subgrades and Structural Layers,* Final Report: WHRP Study #00092-08-07, Wisconsin Department of Transportation, Madison, 2010.

White, D.J., M. Thompson, and P. Vennapusa, *Field Validation of Intelligent Compaction Monitoring Technology for Unbound Materials,* Report No. MN/RC-2007-10, Center for Transportation Research and Education, Iowa State University, Ames, 2007.

White, D.J., and M.J. Thompson, "Relationships between In Situ and Roller-Integrated Compaction Measurements for Granular Soils," *Journal of Geotechnical and Geoenvironmental Engineering,* Vol. 134, No. 12, 2008, pp. 1763–1770.

White, D.J., P. Vennapusa, J. Zhang, H. Gieselman, and M. Morris, *Implementation of Intelligent Compaction Performance Based Specifications in Minnesota,* Report No. MN/RC 2009-14, Minnesota Department of Transportation, St. Paul, 2009.

CHAPTER SIX

SUMMARY OF CURRENT PRACTICE AND EFFECTIVE PRACTICES

OBJECTIVES OF SYNTHESIS STUDY

The primary objective of NCHRP Synthesis 20-05 Topic 43-03, *Practices for Unbound Aggregate Pavement Layers,* was to gather information on the current state of the practice and the state-of-the-art research findings on the following topics:

1. Materials characterization and quality of natural aggregate and common recycled materials that relate to performance;
2. Properties of unbound aggregate layers that are used in the design of pavements and how they are determined (need to first determine the method of design);
3. Influence of gradation on permeability;
4. Current practices and innovations in construction, compaction, and quality assurance (QA) procedures (such as compaction in thicker layers, use of intelligent compaction (IC) systems, and the use of tests other than density in evaluating in-place modulus, stiffness, and quality);
5. Performance of different base types in research pavement sections;
6. Potential to save energy and hauling costs by better utilizing local aggregates and recycled materials;
7. How states manage storage, transport, and placement of materials to minimize degradation of material properties and performance, including lessons learned; and
8. How states address climatic, subgrade, and drainage considerations in design of aggregate base layers.

The previous aggregate base and subbase issues target both flexible and rigid pavement systems and exclude for the purposes of this synthesis gravel and/or unpaved roads. Other broader topics in the areas of chemical admixture (such as lime, cement, fly ash, or bitumen) and/or mechanical additive (geosynthetic, fiber, and so forth) stabilization of aggregates also were excluded from the scope of this synthesis. Relevant information was gathered through literature review, survey of the members of the AASHTO Highway Subcommittee on Materials (including Canadian Provinces), industry input, and selected interviews.

RESEARCH FRAMEWORK

Gaps in knowledge and current practice were noted along with research needs to address these gaps. Information gathered under the scope of this synthesis study findings were documented in this report under the following four chapters.

Aggregate Types and Material Selection

A brief overview of the different types of aggregate materials available as natural resources and mined from sand and gravel pit and quarry operations throughout the United States and Canada was provided. Important aggregate properties and quality aspects that enable a certain aggregate material to pass agency specifications for pavement granular base/subbase use were summarized to establish guidelines for aggregate source selection. The concept of best value granular material utilization was introduced for pavement projects with the potential to save energy and material hauling costs through examples of recent sustainable construction practices highlighting how local aggregates and recycled materials can be better used in granular base/subbase applications.

The use of recycled granular materials in base and subbase layers was discussed in detail. The following two categories of recycled materials were considered: (1) unbound aggregate materials recycled from old pavement base/subbase layers and (2) recycled surface course materials: that is, reclaimed asphalt pavement (RAP) and recycled concrete aggregate (RCA). The goal was to shed light onto what tests are used by agencies to characterize recycled materials for unbound granular base/subbase acceptance and design. Information was gathered on whether or not the same tests are used for characterizing virgin materials and recycled materials before they are included in unbound aggregate base (UAB)/subbase layer specifications and the potential environmental concerns when using RAP and RCA aggregate materials.

Granular Base/Subbase Construction Practices

Diverse agency specifications and construction practices for UAB layers were discussed from the survey results and literature to summarize aspects such as storage, transportation, and placement (e.g., lift thickness) of materials to minimize deviation from intended use and the degradation of material properties and maximize performance through improved structural load-taking ability. Applications of nonstandard or unconventional pavement types using unbound aggregate layers and related construction practices, such as the inverted pavement concept of a granular layer over a stiff layer at depth, were described in detail. Beneficial international practices (e.g., South African and Australian practices for constructing thinly surfaced pavements with the most effective utilization of unbound aggregate layers) were documented with proper construction techniques. In addition, new construction practices and performances of recently built test sections by several state highway agencies in the United States (e.g., Georgia, Louisiana, and Virginia) were summarized in this chapter to demonstrate the advantages of these new unconventional pavement types using unbound aggregate layers.

Unbound Aggregate Base Characterization for Design

Information was gathered from the survey results and research publications on specific unbound aggregate material property inputs obtained from different laboratory and field testing alternatives. All levels of material characterization and quality aspects of different aggregate types (crushed stone, sand and gravel, slag, and other types of recycled materials) were described through aggregate properties and properly evaluated for granular base/subbase strength, deformation, and modulus requirements. Consideration was given to the current agency specifications/design approaches in use and the new characterization tools and improved models [such as stress-dependent and anisotropic modulus, International Center for Aggregate Research (ICAR) model, and so forth] developed for aggregate base/subbase layers. The need for improved characterization of aggregate materials through nonlinear stress-dependent and anisotropic (directionally dependent) models was documented in this chapter based on the improved predictions of pavement responses through comparisons of the predicted and field-measured values in constructed unbound aggregate layer applications.

Finally, information on how highway agencies address climatic, subgrade, and drainage considerations in the design of unbound aggregate layers was discussed in this chapter. Influences of aggregate gradation, fines content, and other material properties, such as particle shape and angularity, on permeability were topics of specific interest when reviewing and summarizing agency survey responses and specifications related to the structural contributions of open-graded aggregate drainage layer applications (e.g., permeable bases and drainable subbases commonly used in rigid pavement foundations).

Compaction, Quality Control, and Field Performance

Detailed findings were presented on different approaches used by transportation agencies for compaction testing on laboratory samples, field compaction, quality control and quality assurance (QC/QA), and finally, field performance evaluations of constructed UAB/subbase layers. Different aspects of compaction and QC of UAB and subbase construction were discussed by introducing the theory of compaction along with the objectives behind compacting unbound aggregate pavement layers. This was followed by a review of different types of compactors commonly used for compacting UAB and subbase layers in the field. The concept of QC was introduced, emphasizing that constructed layer density measurement is the most commonly used field evaluation tool for verifying the adequacy of UAB/subbase construction. Different field techniques used to measure densities of constructed pavement layers were discussed with particular given to the widespread nuclear gauge-based direct density measurement methods.

The concept of modulus-based compaction control was introduced by highlighting its potential advantages, such as continuous compaction control for uniformity over the “spot checking” compared with density-based compaction control approaches. Different IC approaches currently available were discussed through a review of equipment manufacturers. In addition, experiences of different states in the United States for implementing IC approaches were presented along with their preliminary findings. Finally, other portable devices used for measuring the in situ moduli of constructed pavement layers were discussed.

SUMMARY OF STATE PRACTICES

Use of Unbound Aggregate Base and Subbase Layers

- All the responding transportation agencies indicated the use of UAB/subbase layers into the design and construction of pavement structures. Flexible pavement base courses appear to be the most common application of unbound aggregate layers.
- UAB courses are the most common type of aggregate layers used by transportation agencies (used by 96% of responding agencies), whereas only 24% of responding agencies indicated the use of open graded drainage layers (OGDLs). Another common instance of unbound aggregate application in pavements was in working platforms or subgrade replacement and subbase applications.

Material Selection and Construction Practices

- No common practice exists among state and Canadian provincial transportation agencies regarding the frequency of acceptance checking of materials obtained from normally used and approved aggregate sources. Only 39% of the responding agencies indicated that aggregate material quality checking was a requirement before every major construction project.
- Apart from gradation analysis, no other aggregate material quality test is consistently used by transportation agencies.
- Significant differences exist among agencies regarding the maximum allowable particle size for aggregates used in unbound aggregate layers.
- Ninety-eight percent of the responding agencies do not distinguish between nonplastic and plastic fines when specifying the maximum amount of fines allowed in base and subbase courses.

- Sixty-three percent of the responding agencies limit the maximum construction lift thickness to be less than 8 in. (205 mm). Although several research studies have reported on the successful construction of thicker unbound aggregate lifts, transportation agencies have not adopted such thick lift construction as a practice and into their specifications.

Unbound Aggregate Base Characterization for Design

- Shear strength index tests, such as California Bearing Ratio (CBR) and Hveem stabilometer *R*-value, are the most commonly used ones, not only for determining strength properties, but also for characterizing modulus and deformation behavior of UAB and subbase layers.
- The use of falling weight deflectometer (FWD) is the most common practice among transportation agencies for evaluating the characteristics of in-service unbound aggregate layers. Only 10% of responding agencies have started adopting portable field devices, such as the light weight deflectometers (LWD) and soil stiffness gauge (GeoGauge), for in-place modulus measurement of constructed aggregate base and subbase layers.
- Approximately 61% of the responding agencies use the AASHTO 1993 design guide for designing pavements with UAB and subbase layers; 22% of the responding agencies use empirical methods (AASHTO 1972, AASHTO 1986, or agency-specified empirical procedures), and 30% of the agencies have adopted the use of the Mechanistic-Empirical Pavement Design Guide (MEPDG).
- About 22% of the responding agencies do not use resilient modulus as an input for the design of UAB and subbase layers. Although the remaining agencies use aggregate resilient modulus as a key input for pavement design, the common practice is to assign a single modulus to the entire aggregate layer without considering any modulus distribution within the layer owing to the load- or stress-dependent nature of unbound aggregate layer modulus characteristics. Only one agency indicated the use of state-of-the-art concepts such as aggregate cross-anisotropy (directional dependency) in pavement design.
- More than 50% of the responding agencies do not run laboratory tests to determine the resilient modulus of aggregates and instead use empirical correlations with index properties such as CBR and aggregate gradation parameters.
- More than 80% of the responding agencies do not have specific guidelines for including locally available “marginal” aggregates into the thickness design procedure.

Compaction, Quality Control, and Field Performance

- Drop-hammer–based techniques, such as the standard and modified Proctor tests, are used by 91% of the responding agencies for establishing the compaction characteristics of aggregates in the laboratory.
- Field moisture and density measurements using nuclear density gauge are commonly used in 89% of the responding agencies. The in-place densities thus determined are compared with laboratory-established compaction characteristics to check the degree of compaction (DOC) achieved in constructed aggregate layers. Only 28% of the responding agencies construct test strips to establish roller patterns and check for compaction density growth of aggregate layers.
- There is no common practice among transportation agencies regarding the minimum compaction requirements in constructed aggregate base and subbase layers. Compaction requirements often are based on certain percentages of the laboratory-established dry density values. Such differences in compaction requirements by agencies may potentially lead to significantly different pavement aggregate layer responses (i.e., stiffnesses) resulting in much different rutting performances even when similar pavement configurations are constructed for standard design loads.
- More than 50% of the responding agencies expressed interest in implementing non-nuclear density measurement methods for construction control of UAB/subbase layers for one reason or another. However, several of them indicated a lack of confidence in the performance of non-nuclear moisture-density measurement alternatives.
- Although 37% of the agencies have participated in demonstration projects involving continuous compaction control of UAB/subbase layers using IC techniques, only one agency (Texas) has actively implemented IC techniques to construct in-service pavements with UAB/subbase layers; that agency also reported having such a specification currently adopted for use by practitioners.
- Ninety-six percent of the responding agencies do not implement modulus-based compaction control during the construction of unbound aggregate pavement layers, and use achieved layer density and density-based relative compaction as the primary indicator of construction quality.
- None of the agencies have incorporated nontraditional compaction techniques, such as the South African “slushing” method, into unbound aggregate layer construction practices. Any application of such technology has been confined to trial and demonstration projects involved with the application of inverted pavements.

Recycling Aggregates and Recycled Granular Materials

- Thirty-three percent of the responding agencies do not commonly recycle unbound aggregate materials from base and subbase layers of existing pavements, clearly showing a lack of more sustainable construction practices.

- Forty-eight percent of the agencies have not incorporated the use of recycled aggregates from existing base and subbase courses into their specifications.
- Eighty-three percent of the responding agencies indicated that contractors are not allowed to use locally available "marginal" or "out-of-specification" aggregates for UAB and subbase layer construction. Modifying the structural designs of pavements to accommodate such aggregate types on a project-basis may significantly reduce the transportation costs associated with material procurement.
- Sixty-eight percent of the responding agencies reported no environmental concerns associated with the use of recycled materials in the unbound aggregate layer applications. This indicates a possible gap in knowledge with respect to phenomena such as leaching from recycled aggregates.
- Sixty-four percent of the respondents do not require any strength, deformation, or modulus characterization of recycled materials such as RCA and RAP before their use in unbound aggregate pavement layers. For agencies that require such tests to be conducted on recycled materials, the quality requirements are the same as for virgin aggregates.

Climatic Effects and Drainage

- Sixty-one percent of the respondents indicated climatic effects on pavement subgrade performance as an issue of major concern. However, only 15% of the respondents currently test unbound aggregate materials for suction characteristics and other moisture effects, such as soil water characteristic curve or suction characteristics of fines and freeze-thaw durability.
- Fifty-nine percent of the responding agencies do not consider the effects of climatic changes on unbound aggregate layer performance during pavement design. For the agencies that take this aspect into consideration, changing the aggregate layer resilient modulus or structural layer coefficient (as defined by the AASHTO 1993 pavement design guide) appear to be the most common practices.
- Only 41% of the responding agencies specify different gradations for unbound aggregate applications targeting drainable versus low permeability aggregate layers.
- Fifty percent of the responding agencies indicated that drainage is not one of the primary functions of flexible pavement UAB/subbase layers. For the agencies that consider the drainability of dense-graded aggregate layers, limiting the maximum allowable percent fines (material passing sieve No. 200) appears to be the most common practice aimed at facilitating drainage.
- Only 4% of the respondents conduct in situ permeability tests to measure the effectiveness of open-graded aggregate drainage layers. Equal numbers of agencies rely on laboratory permeability measurements or empirical correlations to estimate the permeability of such drainage layers, when used.
- Thirty-seven percent of the respondents indicated the construction of filter layers (using aggregates or geosynthetics) as a common practice for protecting aggregate drainage layers from clogging.
- Thirty percent of the responding agencies do not construct subsurface drainage systems, such as "edge drains," whereas 26% of the respondents indicated such drainage systems are commonly used.

FUTURE RESEARCH AND IMPLEMENTATION

Based on information gathered from this comprehensive survey of state and Canadian provincial transportation agencies, the following topics have been identified where significant gaps in knowledge exist, and accordingly, future research and demonstration projects may be required to modify/improve or further develop agency specifications.

Use of Locally Available Marginal and Out-of-Specification Materials

Most agencies currently do not allow the use of marginal and/or out-of-specification materials in the construction of UAB/subbase layers. Future research needs to focus on how existing pavement designs can be modified for accommodating mechanistic-empirical (M-E) pavement design concepts so that properly mechanistic-based evaluations can be performed for the use of these materials, which would lead to significantly reducing material transportation costs. Laboratory and accelerated pavement testing efforts need to be carried out to evaluate and verify adequate pavement performance using marginal quality aggregates.

Use of Modulus-Based Construction Quality Control

Although several research and demonstration projects have advocated the benefits of implementing modulus-based construction quality control techniques, density-based compaction remains the most commonly used approach. The ongoing NCHRP 10-84, "Modulus-Based Construction Specification for Compaction of Earthwork and Unbound Aggregate," will shed more light on the desired characteristics when developing such specifications. Depending on the project findings, accelerated testing of full-scale pavement test sections and demonstration projects will need to be carried out to evaluate the effectiveness of such modulus-based construction specifications.

Use of Intelligent Compaction Techniques

Almost all research studies and demonstration projects focusing on continuous compaction control using IC techniques have advocated the promise shown by this method. Although pilot studies using IC techniques have been conducted in the United States since 2004, state transportation agencies are hesitant to use this technology more actively, which is primarily the result of the lack of having available standards and construction specifications. Transportation agencies would benefit from participating in IC demonstration projects, and subsequently developing state-approved standards and construction specifications. Ongoing demonstration research studies and demonstration projects funded by the FHWA can contribute significantly to this cause. Note that IC was selected as a FHWA Every Day Counts (EDC) initiative for 2013 (http://www.fhwa.dot.gov/hfl/innovator/issue32.cfm).

Alternative Base Course Applications Such as Inverted Pavements

A review of published literature established that there is widespread consensus among researchers regarding the benefits of alternative base course applications, such as the inverted pavement concept of constructing aggregate layers on a stiff subbase. The use of inverted pavements with thin asphalt surface courses is common in South Africa. Moreover, other countries, such as France and Australia, also use thick aggregate base courses as the primary structural layer in their pavement systems. However, current agency practices in the United States and Canada do not adopt these alternative construction practices. Optimal use of unbound aggregate layers as the primary structural component in pavement systems will greatly benefit with the construction of cost-effective and long-lasting pavement structures designed for improved performance. It would be beneficial to thoroughly evaluate performances of existing pavement sections constructed using such alternative base courses.

KEY LESSONS AND EFFECTIVE PRACTICES

Material Selection and Quality Testing

- The use of 100% uncrushed aggregates in UABs/subbase layers must be done with caution, realizing their substandard strength properties and high rutting potentials.
- To ensure ease of construction and adequate compaction, the maximum particle size allowed in UAB, subbase, and drainage layers are best restricted to 1.5 in., 2 in., and 4 in., respectively.
- Excessive fines (P_{200}) deteriorate aggregate layer performance, especially in the presence of moisture. The maximum amount of fines allowed in UAB/subbase layers are best restricted to 12% *unless prior performance of a material can be documented to show that the material performs satisfactorily at higher fine contents.*
- The presence of plastic fines in an unbound aggregate layer is best limited. For instances where the presence of plastic fines is unavoidable, different threshold limits can be set for the maximum allowable fines content for nonplastic and plastic fines.
- In addition to commonly used tests for evaluating the physical characteristics, the mechanical performance of recycled materials, by-products, and other marginal aggregates needs to be carefully studied.
- RAP materials are tested in the laboratory for resilient modulus and permanent deformation behavior before being used in UAB/subbase layers. Several studies have reported high resilient modulus values for RAP accompanied by significantly high permanent deformation accumulations.
- The expansive properties of RAP materials containing expansive components such as steel slag are best carefully evaluated before their application in UAB/subbase layers.
- Recycled crushed concrete often can be adequately used in UAB/subbase layers.
- Care needs to be taken while blending two different recycled aggregate types to ensure that the resulting blend possesses adequate physical, chemical, and mechanical properties.
- Recycled materials from unknown sources or those to be used in drainage applications are always tested for potential environmental impacts before being used in UAB/subbase layers.
- The use of recycled materials in pavements may be evaluated on a project basis, instead of following generic guidelines. For example, the use of RCA in UAB/subbase layers may or may not be allowed, depending on whether or not the pavement has an underdrain system.

Granular Base and Subbase Construction Practices

- Stockpiling of aggregates using the windrow concept has been proven to be the most efficient practice as far as minimizing segregation is concerned.
- From extensive review of the literature as well as current state practices, this synthesis study finds an optimum lift thickness of 12 in. for the construction of UAB/subbase layers. Note that this finding is based on the assumption that the UAB/subbase layer to be constructed is at least 12-in. thick. Moreover, the DOC achieved is contingent upon the use of adequate equipment by the contractor.

Unbound Aggregate Base Characterization for Design

- Compaction and stress-induced anisotropy may be considered during the design and analysis of pavement systems with UAB and subbase layers.
- Test procedures (AASHTO T 307 and NCHRP 1-28A) for conducting resilient modulus tests on aggregates have

been available for more than a decade. These methods can adequately capture the stress-dependent nature of unbound aggregates and are ready to be implemented in practice. Agencies may incorporate these specifications into practice.

- New research efforts are needed for developing harmonized test protocols for quantifying the permanent deformation behavior of aggregates.
- It would be useful for stress dependence of unbound aggregate materials to be incorporated into future releases of DARWin-ME, the current AASHTO mechanistic empirical pavement design procedure.
- A simplified approach is available for agencies to incorporate the cross-anisotropy of unbound aggregates into pavement design without the need to conduct state-of-the-art triaxial tests.
- Rapid removal of excessive moisture from unbound aggregate layers can be achieved through (1) selection of aggregate materials with low water-retaining tendencies and (2) design of suitable subsurface drainage systems.
- Aggregate materials may be tested for erosion potential or "erodibility" before being used in UAB/subbase layers, particularly under rigid pavements.
- Stable open-graded and gap-graded aggregates with low fine (P_{200}) contents are best used in unbound aggregate drainage layers.
- Tube suction tests can evaluate the frost susceptibility of aggregates before their application in unbound base/subbase layers in areas experiencing significant frost penetration.

Compaction, Quality Control, and Field Performance

- Drop-hammer–based compaction methods (e.g., AASHTO T 99 and T 180) may not be adequate for coarse-grained aggregates, particularly those with low fines (P_{200}) contents. Transportation agencies may need to adopt test procedures similar to ASTM D 7382 to establish the moisture-density curves for unbound aggregates using a vibratory or gyratory compactor.
- The use of roller types that are most suitable for the particular material types is critical to ensuring adequate compaction of unbound aggregate pavement layers.
- Several research and implementation projects have reported different degrees of success with in-place modulus measurement devices. Although these devices have been used successfully to identify anomalies in construction conditions, extensive calibration for local materials is needed before they can be used as primary tools for quality control.
- Most research and implementation projects conducted in the United States involving the use of continuous compaction control and IC to construct UAB/subbase layers have reported considerable success. However, such practices are not common for transportation agencies. Encouraging more implementation projects across agencies can help to incorporate continuous compaction control and IC into agency practice.
- Suction effects and resulting changes in aggregate layer modulus can be considered during the design of UAB/subbase layers.

ACRONYMS

ACPA	American Concrete Pavement Association
AIMS	Aggregate Image Measurement System
CBR	California Bearing Ratio
CCC	Continuous compaction control
CCP	Constant confining pressure
CH	Fat clay
CKD	Cement kiln dust
CL	Lean clay
CMV	Compaction meter value
DCP	Dynamic Cone Penetrometer
DOT	Department of Transportation
EICM	Enhanced Integrated Climatic Model
ESAL	Equivalent single-axle load
FWD	Falling weight deflectometer
GAB	Graded aggregate base
GC	Clayey gravel
GM	Silty gravel
GP	Poorly graded gravel
GW	Well-graded gravel
GWT	Groundwater table
HMA	Hot mix asphalt
IC	Intelligent compaction
ICAR	International Center for Aggregate Research
ICMV	Intelligent compaction measurement value
IPC	Industrial Process Control
LVDT	Linear variable differential transformer
LWD	Light weight deflectometer
MDD	Maximum dry density
MH	Elastic silt
M_R	Resilient modulus
MEPDG	*Mechanistic-Empirical Pavement Design Guide*
ML	Silt
MV	Measurement value
NSSGA	National Stone, Sand and Gravel Association
OH	Organic clay, organic silt
OL	Organic silt, organic clay
OMC	Optimum moisture content
P_{200}	Percentage of material (by weight) passing no. 200 sieve
PCC	Portland cement concrete
PI	Plasticity Index
QA	Quality assurance
QC	Quality control
RAP	Reclaimed asphalt pavement
RCA	Recycled concrete aggregate
RICM	Roller integrated compaction measurement
SASW	Spectral analysis of surface waves
SC	Clayey sand
SHRP	Strategic Highway Research Program
SM	Silty sand
SP	Poorly graded sand

SW	Well-graded sand
SWCC	Soil water characteristic curves
TCLP	Toxicity Characteristic Leaching Procedure
TPF	Transportation Pooled Fund
UAB	Unbound aggregate base
USGS	U.S. Geological Survey
VCP	Variable confining pressure

APPENDIX A

Questionnaire

SURVEY QUESTIONNAIRE
NCHRP PROJECT 20-5
TOPIC 43-03

PRACTICES FOR UNBOUND AGGREGATE PAVEMENT LAYERS

NCHRP TOPIC 43-03 SURVEY QUESTIONNAIRE
FEBRUARY 2011

The Transportation Research Board (TRB) is preparing a Synthesis on "Practices for Unbound Aggregate Pavement Layers." This is being done for NCHRP, under the sponsorship of the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration (FHWA).

Granular aggregate base and subbase layers are very important in pavement construction and performance. Properly designed and constructed bases have the potential to improve pavement performance and longevity while also addressing today's issues like the costs of other pavement materials, the need to save energy, and to reduce greenhouse gas emissions associated with the construction and reconstruction of pavements. A synthesis is being undertaken concerning the full range of unbound aggregate base and subbase issues for both flexible and rigid pavement systems.

States have diverse specifications and construction practices for unbound aggregate pavement layers; sharing this information among the states will most likely lead to better design and construction practices. Information is being gathered through a literature review, survey of the members of the AASHTO Highway Subcommittee on Materials (including Canadian provinces), and selected interviews. Gaps in knowledge and current practices will be noted, along with research needs to address these gaps. This synthesis will ultimately provide information for harmonization of specifications (particularly on a regional basis) to ultimately benefit both states and material producers without adverse impacts on pavement performance.

This questionnaire is being sent to *State Departments of Transportation.* Your cooperation in completing the questionnaire will ensure the success of this effort. **If you are not the appropriate person at your agency to complete this questionnaire, please forward it to the correct person.**

Please complete and submit this survey by April 6, 2012. For questions, please contact our principal investigator:

Erol Tutumluer, Ph.D.	E-mail: tutumlue@illinois.edu	Phone (217) 333-8637

Please identify your contact information. NCHRP will email you a link to the online report when it is completed.

Agency: ______________________________

Address: ______________________________

City: ____________ State: ____________ ZIP: ____________

Questionnaire Contact: ______________________________

Position/Title: ______________________________

In case of questions and for NCHRP to send you a link to the final report, please provide:

Tel: ____________________ E-mail: ____________________

General: Use of Unbound Aggregate Base and Subbase Layers

Please provide the contact information of the persons in your agency who directly deal with the material selection and characterization, as well as design and construction of unbound aggregate base and subbase layers for pavement applications. The consultants will contact the relevant personnel separately with specific questions.

Material Selection and Characterization

Name: ____________________

Position/Title: ____________________

Address: ____________________

City: ____________ State: ____________ ZIP: ____________

Telephone: ____________; E-Mail: ____________; Fax: ____________

Pavement Design

Name: ____________________

Position/Title: ____________________

Address: ____________________

City: ____________ State: ____________ ZIP: ____________

Telephone: ____________; E-Mail: ____________; Fax: ____________

Construction of Pavements with Unbound Aggregate Layers

Name: ____________________

Position/Title: ____________________

Address: ____________________

City: ____________ State: ____________ ZIP: ____________

Telephone: ____________; E-Mail: ____________; Fax: ____________

1. Is it common practice for your agency to incorporate unbound aggregate layers into the design and construction of pavement structures? (*Note that this survey focusses on unbound aggregate layers in asphalt, concrete, and composite pavements only, and does not include unbound aggregate layer applications in unsurfaced pavements and gravel roads.*)

   ☐ Yes

   ☐ No

   If your answer to the above question is "No", you do <u>not</u> need to complete this survey. Please include comments below regarding why your agency does not prefer to construct unbound aggregate layers as pavement base and subbase courses.

   ____

   ____

   ____

2. What types of unbound aggregate layers are commonly constructed by your agency? (Please check all that apply.)

   ☐ Base course

   ☐ Subbase course

☐ Open graded drainage layer

☐ Pavement working platforms for subgrade stability applications

3. What types of pavement structures designed and constructed by your agency commonly incorporate unbound aggregate layers?

☐ Flexible pavements (comprising hot mix or warm mix asphalt surface layer)

☐ Rigid pavements (comprising portland/hydraulic cement concrete slabs)

☐ Rehabilitated pavements (asphalt overlay over concrete, etc.)

☐ Others such as composite pavements, inverted pavements, etc.
(Please specify): ____________________

4. What primary functionalities of unbound aggregate layers are intended to serve in pavement systems designed and constructed by your agency? (Please check all that apply.)

☐ Dense graded base courses as primary structural layers

☐ Open graded layers under rigid pavements for uniform support and providing drainage

☐ Pavement construction platforms to protect weak subgrade layers from excessive rutting under heavy construction equipment loading

☐ Others (please specify): ____________________

Category 1: Material Selection and Construction Practices

5. How frequently does your agency check the acceptance of materials obtained from commonly used and/or approved aggregate sources?

☐ Prior to the use on every major construction project

☐ More than twice every year

☐ Twice every year

☐ Once every year

☐ Less than once every year

☐ Other (please explain): ____________________

6. What tests are used by your agency for evaluating quality aspects of virgin aggregate materials for pavement base and subbase applications? (Please check all that apply.)

☐ $Na_2SO_4/MgSO_4$ Soundness Test

☐ Los Angeles Abrasion and/or Micro Deval Test

☐ Sieve Analysis

☐ Percent Deleterious Materials

☐ Other (please indicate): ____________________
____
____

7. Does your agency permit the use of uncrushed aggregates in pavement base and subbase applications?

☐ Yes

☐ No

☐ Other (please explain): ____________________

8. What is the maximum aggregate particle size (D_{max}) in inches allowed by your agency in the following constructed unbound aggregate layers?

- ☐ Dense graded base: ____________________
- ☐ Dense graded subbase: ____________________
- ☐ Open graded base as drainage layers: ____________________
- ☐ Open graded subbase as drainage layers: ____________________
- ☐ Other (please list below):

____

____

9. What is the maximum amount of fines (material finer than 0.075 mm or passing sieve No. 200) allowed by your agency for aggregates to be used for unbound aggregate base and subbase course applications?

- ☐ Dense graded base (%):
- ☐ Dense graded subbase (%):
- ☐ Open graded base/subbase as drainage layers (%):

10. Does your agency specify different allowable percent fines (material finer than 0.075 mm or passing sieve No. 200) for aggregates having nonplastic and plastic fines?

- ☐ Yes (please give the specified values below):
  - ☐ Base, Nonplastic (%):
  - ☐ Base, Plastic (%)
  - ☐ Subbase, Nonplastic (%):
  - ☐ Subbase, Plastic (%):
- ☐ No

11. What is the maximum value of plasticity index (PI) allowed by your agency for the fines fraction of aggregate materials to be used in the following unbound pavement layers?

- ☐ Dense graded base: ____________________
- ☐ Dense graded subbase: ____________________
- ☐ Open graded base as drainage layers: ____________________
- ☐ Open graded subbase as drainage layers: ____________________

12. Does your agency have a list of approved aggregate types and sources for base and subbase course applications?

- ☐ Yes (please reference document/website) ____________________
- ☐ No
- ☐ Other (please explain): ____________________

13. If you answered "Yes" to Question 12, does your agency frequently allow new materials into the list of approved aggregate types and sources for base and subbase course applications?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

14. Does your agency have specific guidelines regarding the transportation and storage (stockpiling) of aggregate materials for base and subbase construction?

- ☐ Yes (please reference document/website) ____________________
- ☐ No
- ☐ Other (please explain): ____________________

15. What is the maximum construction lift thickness for unbound aggregate layers permitted by your agency?

- ☐ 6 in.
- ☐ 8 in.
- ☐ 10 in.
- ☐ 12 in.
- ☐ Other (please indicate): ____________________
- ☐ No such restrictions

16. Does your agency allow the construction of multiple unbound aggregate layers placed on top of each other (e.g., dense graded base over an open graded drainage layer)?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

17. If you answered "Yes" to the above question, do you separate the two unbound aggregate layers by any kind of constructed aggregate separation (i.e., filter) layer?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

18. Does your agency allow the construction of unbound aggregate layers over or under pavement layers stabilized/treated with lime, fly ash, cement, or bitumen?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

19. If you answered "Yes" to the above question, please give typical thicknesses of such constructed layers:

- Thickness (in.) of stabilized layer ____________________
- Thickness (in.) of unbound aggregate layer ____________________

Category 2: Unbound Aggregate Base Characterization for Design

20. Who is responsible for testing/characterizing aggregate materials and providing input properties for the design of pavements with unbound granular layers?

- ☐ Geotechnical/materials laboratory
- ☐ University laboratory (under research subcontract)
- ☐ Field engineer
- ☐ Other (please indicate): ____________________

21. What laboratory tests are conducted by your agency for strength, deformation, and modulus characterization of unbound aggregates used in base and subbase course applications? (Please check all that apply.)

☐ Strength index tests (e.g., CBR, Hveem stabilometer, etc.)
☐ Triaxial shear strength tests
☐ Repeated load triaxial tests for resilient modulus (standard tests such as AASHTO T 307, NCHRP 1-28, etc.)
☐ Repeated load triaxial tests for permanent deformation behavior
☐ Other (please indicate): ______________________________

22. What field tests are conducted by your agency for strength, deformation, and modulus characteristics of in-service unbound aggregate layers? (Please check all that apply.)

☐ Plate Load tests
☐ Dynamic Cone Penetrometer (DCP) tests
☐ Falling Weight Deflectometer (FWD) tests
☐ Light Weight Deflectometer (LWD) tests
☐ Soil Stiffness Gauge tests (e.g., GeoGauge™, etc.)
☐ Other (please indicate): ______________________________

23. How often are these laboratory/field tests conducted to characterize aggregate materials for use in granular base and subbase layers?

☐ Once on limited aggregate types/materials commonly used by the agency
☐ On a project-need basis
☐ Once on all agency-approved aggregate sources
☐ At regular intervals on agency-approved aggregate sources to establish a database
☐ Other (please indicate): ______________________________

24. Is the structural contribution of open graded aggregate drainage layers taken into account in pavement thickness design by your agency?

☐ Yes
☐ No
☐ Other (please explain): ______________________________

25. What method is used by your agency to design pavements with unbound aggregate base and subbase layers?

☐ 1972 AASHTO Design Guide
☐ 1986 AASHTO Design Guide
☐ 1993 AASHTO Design Guide
☐ Mechanistic-Empirical Pavement Design Guide (MEPDG)
☐ Agency-Specific Mechanistic Procedure: ______________________________
☐ Agency-Specific Empirical Procedure: ______________________________
☐ Other (please indicate): ______________________________

26. What aggregate properties and/or characteristics are used as inputs for the design of pavements with granular base/subbase layers by your agency? (Please check all that apply.)

☐ Percent passing sieve sizes (gradation) and/or maximum aggregate particle size
☐ Particle shape and angularity (crushed or uncrushed)

- ☐ Compaction characteristics; i.e., optimum moisture content and maximum dry density
- ☐ Shear strength properties (e.g., friction angle, CBR, etc.)
- ☐ Resilient modulus
- ☐ Other (please indicate): ____

27. What approach is adopted by your agency for assigning resilient modulus values to unbound aggregate base and subbase layers?

- ☐ Modulus is not used in pavement design
- ☐ Single modulus is assigned to the entire layer
- ☐ Stress-dependency of aggregate layer modulus is considered during pavement design
- ☐ Based on research by the International Center for Aggregate Research (ICAR), anisotropy (directional dependency) of aggregate layer modulus is considered
- ☐ Other (please indicate): ____

28. How does your agency determine the resilient modulus of unbound aggregate materials for use in granular base and subbase layers?

- ☐ Resilient modulus testing in the laboratory
- ☐ Empirical correlations with index properties like CBR, gradation parameters, etc.
- ☐ In-place modulus measurement of constructed layers by deflection-based methods such as FWD, LWD, etc.
- ☐ Other (please indicate): ____

29. Does your agency have special provisions for including new and/or locally available "marginal" aggregates in the thickness design of unbound base and subbase layers?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____

Category 3: Compaction, Quality Control, and Field Performance

30. How are aggregate compaction characteristics established in the lab by your agency?

- ☐ Drop-hammer based methods like the standard and modified Proctor tests
- ☐ Static compaction methods
- ☐ Vibratory compaction methods
- ☐ Gyratory compaction methods
- ☐ Kneading compaction methods
- ☐ Other (please indicate): ____

31. List typical equipment used for placement and construction of unbound aggregate base/subbase layers in your state. (Please check all that apply.)

- ☐ Dump truck
- ☐ Material Transfer Device
- ☐ Trucks and graders
- ☐ Aggregate spreaders

- ☐ Other (please indicate below):

____

____

32. Please describe briefly the preferred method of placement and construction of unbound aggregate base and subbase layers in your state.

____

____

____

33. How is moisture content of constructed/compacted unbound aggregate base and subbase controlled in the field?

- ☐ According to quarry reported moisture content
- ☐ Sampled during construction/compaction for laboratory testing
- ☐ Measured through field testing using (please indicate): ______________________________
- ☐ Other (please indicate): ______________________________

34. What is an acceptable variation from optimum moisture content for constructed/compacted unbound aggregate base and subbase allowed by your agency?

____

____

35. What is the primary approach used by your agency for evaluating degree of compaction and construction quality control of unbound aggregate base and subbase layers?

- ☐ Gradation
- ☐ Proof-rolling
- ☐ Measurement of constructed layer density
- ☐ Field measurement of constructed layer modulus
- ☐ Continuous compaction control by means of Intelligent Compaction (IC) equipment
- ☐ Other (please indicate): ______________________________

36. If your answer to the previous question was "Measurement of constructed layer density," what method is commonly used by your agency for measuring constructed layer densities in the field?

- ☐ Nuclear density methods (ASTM D 2922/AASHTO T310 or T238)
- ☐ Sand cone method (ASTM D 1556/AASHTO T191)
- ☐ Balloon method (ASTM D 2167)
- ☐ Oil or water method
- ☐ Drive cylinder method (ASTM D 2937)
- ☐ Rapid method (ASTM D 5080)
- ☐ Electrical density measurement method (ASTM D 7698)
- ☐ Other (please indicate): ______________________________

37. Is it common practice for your agency to construct "test strips" to establish roller patterns and check for compaction density growth of unbound aggregate base and subbase layers?

☐ Yes

☐ No

☐ Other (please explain): ____________________

38. If your answer to the above question was "Yes," do you primarily use nuclear density measurement method for this purpose?

☐ Yes

☐ No

☐ Other (please explain): ____________________

39. What is the primary mode of nuclear gauge operation used by your agency for density checks on constructed unbound aggregate base and subbase layers?

☐ Direct transmission

☐ Backscatter

☐ Not applicable (nuclear density measurement not used)

40. If your agency uses the "drop-hammer based" methods to establish compaction characteristics of aggregates in the laboratory, please circle and select all applicable options from the list below regarding the minimum compaction requirements in the field:

Pavement Type	**Aggregate Layer Type**	**Compaction Method**	**Field Relative Compaction**			
Flexible	Base	Standard Proctor	90%	95%	100%	Other ________
Flexible	Base	Modified Proctor	90%	95%	100%	Other ________
Flexible	Subbase	Standard Proctor	90%	95%	100%	Other ________
Flexible	Subbase	Modified Proctor	90%	95%	100%	Other ________
Rigid	Base/Subbase	Standard Proctor	90%	95%	100%	Other ________
Rigid	Base/Subbase	Modified Proctor	90%	95%	100%	Other ________

41. If you selected "Other" as one of the responses to the above question, please specify the answer below (submit your answer in the format: Flexible Base-Standard Proctor-xx%)

☐ ____________________

42. If your agency does NOT use "drop-hammer based" methods to establish compaction characteristics of aggregates in the laboratory, please specify the relative compaction requirements for the following unbound aggregate base/subbase applications:

Pavement Type	**Aggregate Layer Type**	**Field Relative Compaction**			
Flexible	Base	90%	95%	100%	Other ________
Flexible	Subbase	90%	95%	100%	Other ________
Rigid	Base/Subbase	90%	95%	100%	Other ________

43. If you selected "Other" as one of the responses in the above question, please specify the details below:

☐ ____________________

44. Is there common interest among engineers in your agency to implement non-nuclear density measurement methods for construction quality control of unbound aggregate base/subbase layers?

- ☐ Yes (please select all possible reasons):
  - ☐ Safety concerns
  - ☐ Nuclear certification too expensive
  - ☐ Nuclear certification too inconvenient
  - ☐ Non-nuclear methods provide better results
- ☐ No
- ☐ Other (please explain): ______________________________

45. Has your agency ever participated in demonstration projects involving continuous compaction control of unbound aggregate base/subbase layers using Intelligent Compaction (IC) techniques?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ______________________________

46. Has your agency actively implemented IC techniques to construct in-service pavements with unbound aggregate base/subbase layers?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ______________________________

47. If your answer to the above question was "Yes," does your agency have a specification for continuous compaction control using IC techniques and construction of unbound aggregate base/subbase layers?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ______________________________

48. Does your agency implement modulus-based compaction control for the construction of unbound aggregate base/subbase layers?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ______________________________

49. If you answered "Yes" to the previous question, what method is used for measuring the field modulus of constructed unbound aggregate layers?

- ☐ Falling Weight Deflectometer (FWD)
- ☐ Light Weight Deflectometers (LWD)
- ☐ Soil Stiffness Gauge (e.g., GeoGauge™)
- ☐ Seismic testing such as the Portable Seismic Pavement Analyzer based on spectral analyses of surface waves
- ☐ Other (please indicate): ______________________________

50. If you answered "Yes" to question No. 46, what percentage of construction projects in your agency involve modulus based compaction control?

    ☐ <10%
    ☐ 10–30%
    ☐ 30–60%
    ☐ >60%

51. Does your agency use any non-traditional compaction technique (e.g., South African "slushing" technique) during the construction of unbound aggregate base/subbase?

    ☐ Yes (please name the technique and give brief detail below)
    ☐ No

    Comments:

52. Does your agency perform FWD tests to assess the structural condition of unbound aggregate base and subbase layers in existing in-service pavements?

    ☐ Yes
    ☐ No
    ☐ Other (please explain): ____________________

53. If your answer to the previous question was "Yes," what backcalculation program/software/approach does your agency use to determine granular base/subbase modulus from the FWD test results? (Please indicate below.)

    ____
    ____

54. What tests are commonly used by your agency to evaluate the field-performance of existing pavements with unbound aggregate base and subbase layers? (Please list all.)

    ____
    ____
    ____

Category 4: Recycling Aggregates and Recycled Granular Materials

Note: Recycled Granular Materials in this questionnaire refer to Recycled Concrete Aggregates (RCA) and Reclaimed Asphalt Pavement (RAP) materials only.

55. Does your agency commonly recycle unbound aggregate materials from base and subbase layers of existing pavements for application in new and rehabilitated pavement construction?

    ☐ Yes
    ☐ No
    ☐ Other (please explain): ____________________

56. If your answer to the previous question was "Yes," what tests are used by your agency for evaluating the quality of these recycled aggregates? (Please check all that apply.)

- ☐ Na_2SO_4/$MgSO_4$ Soundness Test
- ☐ Los Angeles Abrasion and/or Micro Deval Test
- ☐ Sieve Analysis
- ☐ Percent Deleterious Materials
- ☐ Other (please indicate): ______________________________

____

57. What other tests are used by your agency to characterize recycled aggregates from existing base and subbase courses for acceptance and design? (Please list all.)

- ☐ ______________________________
- ☐ ______________________________
- ☐ ______________________________

58. Is the use of recycled aggregates from existing base and subbase courses incorporated into your agency specifications?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ______________________________

59. Does your agency allow contractors to use locally available "marginal or out of specification" aggregates for unbound aggregate base and subbase layer applications?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ______________________________

60. What other recycled granular materials are approved for use by your agency in unbound aggregate layer construction? (Please check all that apply.)

- ☐ Reclaimed Asphalt Pavement (RAP)
- ☐ Recycled Concrete Aggregates (RCA)
- ☐ Other (please indicate): ______________________________
- ☐ None of the above (please skip to Category 5 of the survey)

61. What tests are used by your agency for evaluating the material quality of recycled granular materials (RCA, RAP, and/or others from question No. 58) for base and subbase applications? (Please check all that apply.)

- ☐ Na_2SO_4/$MgSO_4$ Soundness Test
- ☐ Los Angeles Abrasion and/or Micro Deval Test
- ☐ Sieve Analysis
- ☐ Percent Deleterious Materials
- ☐ Other (please indicate): ______________________________

____

62. What other tests are used by your agency to characterize recycled granular materials (RCA, RAP, and/or others from question No. 58) for acceptance and design? (Please list all.)

- ☐ ____________________
- ☐ ____________________
- ☐ ____________________

63. Does your agency have environmental concerns regarding the use of recycled granular materials (RCA, RAP, and/or others from question No. 58) in unbound aggregate base and subbase layers?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

64. If your answer to the above question was "Yes," what environmental issues is your agency particularly concerned about (e.g., leaching, etc.)?

- ☐ ____________________
- ☐ ____________________
- ☐ ____________________

65. Does your agency require strength, deformation and modulus testing and characterization of recycled aggregates (from existing base and subbase courses) as well as recycled granular materials (RCA, RAP, and/or others from question No. 58)?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

66. If you answered "Yes" to the previous question, are these characterization tests the same as those used for virgin aggregates?

- ☐ Yes
- ☐ No (please indicate below how the test methods are different)

Comments:

Category 5: Climatic Effects and Drainage

67. Are climatic effects on pavement subgrade performance a major concern for your agency?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

68. If your answer to the above question was "Yes," please check all factors that contribute to this concern:

- ☐ Groundwater table (GWT) is often shallow (can be less than 5 ft deep) under the pavements
- ☐ Native soils primarily fine-grained (e.g., silts, clays, etc.) and may get wet of optimum due to upward movement of moisture from GWT
- ☐ In-service pavement subgrades are often under "wet of optimum" moisture conditions

- ☐ Seasonal fluctuations cause significant changes in subgrade soil properties
- ☐ Subgrades stay frozen for extended periods (one month or longer)
- ☐ More than 10 freeze-thaw cycles per year are experienced at the subgrade level
- ☐ Spring thaw weakening and timing of spring load restrictions
- ☐ Subgrade soils are primarily frost-susceptible (i.e., silty soils)

69. Are aggregate materials selected for use in granular base/subbase applications by your agency tested for climatic effects (e.g., soil water characteristic curve, freeze-thaw durability, suction characteristics of fines, etc.)?

- ☐ Yes (please indicate the test types) ____________________
- ☐ No
- ☐ Other (please explain): ____________________

70. Does the pavement design procedure used by your agency consider the effects of climatic changes on unbound aggregate layer performance?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

71. If your answer to the above question was "Yes," what unbound aggregate layer properties are adjusted in your pavement design procedure to account for detrimental climatic effects on pavement performance?

- ☐ Layer structural coefficients
- ☐ Resilient modulus
- ☐ Shear strength
- ☐ Other (please indicate): ____________________

72. Are there different gradations specified by your agency for unbound aggregate applications targeting drainable vs. low permeability aggregate layers?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

73. If drainage is one of the primary functions of your flexible pavement unbound aggregate base/subbase layer, what approach is adopted by your agency to facilitate the drainability of dense-graded base courses?

- ☐ Limiting the maximum allowable percent fines (material passing sieve No. 200)
- ☐ Increasing the maximum aggregate size
- ☐ Adjusting the constructed layer gradation toward a more open-graded layer
- ☐ Other (please explain): ____________________

74. Does your agency distinguish between crushed and uncrushed aggregate types while constructing open-graded drainage layers?

- ☐ Yes
- ☐ No
- ☐ Other (please explain): ____________________

75. How is the effectiveness of an open-graded aggregate drainage layer measured by your agency?

- ☐ In-situ permeability measurements
- ☐ Laboratory tests to measure the permeability of aggregate samples
- ☐ Empirical correlations to estimate the permeability from aggregate physical properties like gradation, dry density, specific surface, and void ratio (or porosity)
- ☐ Other (please explain): ______________________________

____

____

76. For pavement structures with aggregate drainage layers, is it common practice in your agency to include a filter layer underneath to protect the drainage layer from clogging?

- ☐ Yes (open graded aggregates commonly used to construct the filter layer)
- ☐ Yes (geosynthetics commonly used as the mode of filtration)
- ☐ No (no extra layer constructed for filtration purposes)

77. How common is it for your agency to construct subsurface drainage systems like “edge-drains” under unbound aggregate base and subbase layers?

- ☐ Yes, very common
- ☐ Yes, for specific projects when required by the design
- ☐ No, not common at all

This is the End of Questionnaire

Thank you for your Cooperation

APPENDIX B

Respondent Information

SURVEY RESPONDENT INFORMATION
NCHRP PROJECT 20-5
TOPIC 43-03

PRACTICES FOR UNBOUND AGGREGATE PAVEMENT LAYERS

State	Name	Title	Agency/Organization
AK	Steve Saboundjan	State Pavement Engineer	Alaska DOT/PF
AL	Robert Shugart	Materials Engineer	Alabama DOT
AR	Michael Benson	Materials Engineer	Arkansas State Highway and Transportation Department
AZ	Paul Burch	State Pavement Design Engineer	Arizona Department of Transportation
CA	Hector Romero		California Department of Transportation
CO	Jay Goldbaum	Pavement Design Program Engineer	Colorado Department of Transportation
DC	Wasi Khan	Materials Engineer	District of Columbia (DC) DOT
DE	Greg Hainsworth	Materials Engineer	Delaware DOT
FL	David Horhota	State Geotechnical Materials Engineer	Florida DOT
GA	Georgene Geary	State Materials and Research Engineer	Georgia DOT
NC	Jack Cowsert	State Materials Quality Engineer	North Carolina DOT
ID	Michael Santi	Materials Engineer	Idaho Transportation Department
IL	Sheila Beshears	Ms.	Illinois Department of Transportation
IN	Ronald Walkers	Manager, Office of Materials Engineer	Indiana NDOT
KS	Andrew Gisi	Geotechnical Engineer	Kansas Department of Transportation
LA	Bert Wintz	Field Quality Assurance Administrator	Louisiana DOTD Materials and Testing Section
MD	Dan Sajedi	Soils and Aggregate Division Chief	Maryland State Highway
ME	Karen Gross	Assistant Engineer – Pavement Design and Quality	Maine Department of Transportation
MN	Tim Andersen	Pavement Design Engineer	Minnesota Department of Transportation
MO	John Donahue	Construction and Materials Liaison Engineer	Missouri DOT
WY	Rick Harvey	State Materials Engineer	Wyoming DOT
MS	James Williams	State Materials Engineer	Mississippi Department of Transportation
MT	Dan Hill	Pavement Analysis Engineer	Montana DOT
ND	Ron Horner	Materials and Research Engineer	North Dakota DOT
NH	Denis Boisvert	Chief of Materials Technology	New Hampshire DOT
NJ	Eileen Sheehy		New Jersey DOT
NM	Bryce Simons		New Mexico Department of Transportation
NV	Charlie Pan	Principal Bituminous Engineer	Nevada Department of Transportation
NY	Bob Burnett	Director, GEB	New York State DOT
OH	Lloyd Welker	Administrator Office of Materials Management	Ohio Department of Transportation

State	Name	Title	Agency/Organization
OK	Vincent Reidenbach		Oklahoma DOT
OR	Justin Moderie	Pavement Design Engineer	Oregon Department of Transportation
PA	Timothy Ramirez	Engineer of Tests	PA Department of Transportation
RI	Robert Snyder	Sr. Civil Engineer (Materials)	Rhode Island DOT
SC	Andrew Johnson	State Pavement Design Engineer	South Carolina DOT
SD	Joe Feller	Chief Materials and Surfacing Engineer	South Dakota DOT
TX	Mark McDaniel	Transportation Engineer	Texas Department of Transportation
UT	Howard Andersen	Quality Assurance and Aggregate Engineer	Utah DOT
VA	Mohamed Elfino	Assistant Division Engineer	Virginia Dept. of Transportation
WA	Michael Polodna	Structural Materials Testing Engineer	Washington State Dept. of Transportation
WI	Steven Krebs	Chief Materials Management Engineer	Wisconsin Department of Transportation
WV	Thomas Medvick	Pavement Engineer	West Virginia DOT/DOH Materials Division
AB, Canada	Chick McMillan	Director-Surface Engineering and Aggregates	Alberta Transportation
NL, Canada	Ken Pike	Manager, Materials Engineering	Transportation and Works, Newfoundland and Labrador
ON, Canada	Stephen Senior	Head, Soils and Aggregates Section	Ministry of Transportation
SK, Canada	Ania Anthony	Senior Surfacing Engineer	Saskatchewan Ministry of Highways & Infrastructure

APPENDIX C
Survey Responses

COMPILATION OF SURVEY RESPONSES NCHRP PROJECT 20-5 TOPIC 43-03

PRACTICES FOR UNBOUND AGGREGATE PAVEMENT LAYERS

Background and Purpose

The Transportation Research Board (TRB) is preparing a Synthesis on "Practices for Unbound Aggregate Pavement Layers." This is being done for NCHRP, under the sponsorship of the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration (FHWA).

Granular aggregate base and subbase layers are very important in pavement construction and performance. Properly designed and constructed bases have the potential to improve pavement performance and longevity while also addressing today's issues like the costs of other pavement materials, the need to save energy, and to reduce green-house gas emissions associated with the construction and reconstruction of pavements. A synthesis is being undertaken concerning the full range of granular aggregate base and subbase issues for both flexible and rigid pavement systems.

States have diverse specifications and construction practices for unbound aggregate pavement layers; sharing this information among the states will most likely lead to better design and construction practices. Information is being gathered through literature review, survey of the members of the AASHTO Highway Subcommittee on Materials (including Canadian Provinces), and selected interviews. Gaps in knowledge and current practices will be noted, along with research needs to address these gaps. This synthesis will ultimately provide information for harmonization of specifications (particularly on a regional basis) to ultimately benefit both States and material producers without adverse impacts on pavement performance.

This questionnaire is being sent to state departments of transportation. Your cooperation in completing the questionnaire will ensure the success of this effort. If you are not the appropriate person at your agency to complete this questionnaire, please forward it to the correct person.

General: Use of Unbound Aggregate Base and Subbase Layers

1) **Is it common practice for your agency to incorporate unbound aggregate layers into the design and construction of pavement structures?** ***(Note that this survey focuses on unbound aggregate layers in asphalt, concrete and composite pavements only, and does not include unbound aggregate layer applications in unsurfaced pavements and gravel roads). If your answer to this question is "No," you do not need to complete this survey. Please include comments in the textbox regarding why your agency does not prefer to construct unbound aggregate layers as pavement base and subbase courses***

   [46] Yes - **100%**
   [0] No - **0%**
   46 Respondents

2) **What types of unbound aggregate layers are commonly constructed by your agency? (Please check all that apply.)**

   [44] Base course - **95.7%**
   [30] Subbase course - **65.2%**
   [11] Open graded drainage layer - **23.9%**
   [21] Pavement working platforms for subgrade stability applications - **45.7%**
   46 Respondents

3) **What types of pavement structures designed and constructed by your agency commonly incorporate unbound aggregate layers? (Please check all that apply.)**

[46] Flexible pavements (comprising hot mix or warm mix asphalt surface layer) - **100%**
[32] Rigid pavements (comprising portland/hydraulic cement concrete slabs) - **69.9%**
[14] Rehabilitated pavements (asphalt overlay over concrete, etc.) - **30.4%**
[10] Others such as composite pavements, inverted pavements, etc. (please specify) - **21.7%**
46 Respondents

"Others" responses

- All of our pavements have had a subbase beneath them since at least the 1950s.
- Composite pavements
- Foamed Asphalt base (FASB)
- Have done one inverted pavement, but are considering more.
- Inverted pavement
- We have one inverted pavement project underway right now
- Composite (listed 3 times)
- Composite bases with a layer of crushed stone over a soil cement layer

4) **What primary functionalities are unbound aggregate layers intended to serve in pavement systems designed and constructed by your agency? (Please check all that apply.)**

[43] Dense graded base courses as primary structural layers - **93.5%**
[11] Open graded layers under rigid pavements for uniform support and providing drainage - **23.9%**
[24] Pavement construction platforms to protect weak subgrade layers from excessive rutting under heavy construction equipment loading - **52.2%**
[4] Others (please specify): - **8.7%**
46 Respondents

"Others" responses

- Control expansion
- Gravel Subbase is most commonly used
- On a geotextile or geogrid for pavement construction platform
- These are the main three, but the first is the primary use, with occasional uses for the other two.

Category 1: Material Selection and Construction Practices

5) **How frequently does your agency check the acceptance of materials obtained from commonly used and/or approved aggregate sources?**

[18] Prior to the use on every major construction project - **39.1%**
[2] More than twice every year - **4.3%**
[1] Twice every year - **2.2%**
[5] Once every year - **10.9%**
[3] Less than once every year - **6.5%**
[17] Other (please explain): - **37%**
46 Respondents

"Others" responses

- 1/2,000 tons on every project
- Acceptance is done on delivered materials on every major project
- Aggregate base is approved by stockpile to be used in each project.
- Gradation and density every 2500 tons
- QA every 10 days interval during production and delivery to the project site
- QC/QA on each project
- Subbase material is placed in stockpiles and every pile is sampled and tested.

- The aggregate base is checked for acceptance for any project that has least 500 cu. yd or more.
- Soundness and durability are done annually, gradations and densities are performed every 1,000 CY
- Aggregates are accepted from a Certified Aggregate Producer. INDOT audits these plants on an annual basis.
- QA on a project basis—aggregate sources are glacio-fluvial deposits and often a source is used infrequently.
- Aggregate soundness is annual. Other properties are at the start of the project and typically every 2000 tons thereafter.
- Once a year for contractor furnished sources—Make state leased sources available to the contractor, dig test pits, perform quality tests, and make information available to contractors before the pit or quarry is opened.
- Unbound aggregate is tested under ODOT certification program before shipment to the jobsite for gradation. Source materials are quality testing from once to 4 times a year.
- Quarries are qualified annually or biennially depending on the characteristics of the materials, and then acceptance samples are taken from the roadway during construction.
- All aggregate sources and each material from a source are sampled and requalified every two years.
- Once the source is tested and approved for durability, we run gradation and deleterious materials acceptance samples approx. every 1,000 cu. yards of material.

6) What tests are used by your agency for evaluating quality aspects of virgin aggregate materials for pavement base and subbase applications? (Please check all that apply)

[26] Na_2SO_4/$MgSO_4$ Soundness Test - **56.5%**
[39] Los Angeles Abrasion and/or Micro Deval Test - **84.8%**
[43] Sieve Analysis - **93.5%**
[30] Percent Deleterious Materials - **65.2%**
[22] Other (please indicate): - **47.8%**
46 Respondents

<u>“Others” responses</u>

- Absorption and Specific Gravity
- Absorption and Specific Gravity, by either T84/85 or by TP77
- Atterberg Plasticity Index
- Freeze Thaw
- Idaho IT-15 Idaho Degradation
- LAR & -#200 Insoluble Residue
- Limerock Bearing Ratio (LBR)
- Liquid Limit, Fracture Face, CBR, Dry-rodded Weight
- PI
- Petro Number
- Plastic Index
- Plasticity Index, Fractured Coarse Aggregate Particles
- R-Value
- Sand equivalent, durability, R value
- Thin & Elongated, Crushed Fragments, & Unit Weight
- Volume Swell (MT-305)
- WSDOT Degradation Test, Sand Equivalent
- Washington Degradation
- Atterberg limits
- Plasticity, unconfined freeze-thaw, permeability
- ODOT TM 208 degradation test. It checks the soundness of aggregate in wet conditions by agitating crushed aggregate with bubblers under water, and then performing a sand equivalency type evaluation.
- Texas Triaxial strength, Plasticity Index with liquid limit separate, wet-ball mill (hardness), aggregate type (morphology)

7) Does your agency permit the use of uncrushed aggregates in pavement base and subbase applications?

[20] Yes - **43.5%**
[20] No - **43.5%**
[6] Other (please explain): - **13%**
46 Respondents

<u>"Others" responses</u>

- A minimum of 30% fractured coarse aggregate is required.
- Natural rough surfaced gravel
- Subbase only
- Allowed in subbase but not in base
- Only with a stabilized sand clay gravel material
- Spec Requirement: Aggregates shall consist of Granular material of which 30% of the particles retained on the No. 4 sieve shall contain one or more fractured faces.

8) What is the maximum aggregate particle size (D_{max}) in inches allowed by your agency in the following constructed unbound aggregate layers? *46 respondents*

[44] Dense graded base:
¾ of the lift thickness (1 agency)
1 in. (11 agencies)
1.5 in. (21 agencies)
1.75 in. (1 agency)
2 in. (4 agencies)
2 in. for graded aggregate base; 3.5 in. for coquina shell base; 1.5 in. for sand-clay base (1 agency)
2.5 in. max is allowed, but 1 in. max is almost always used (1 agency)
3 in. (2 agencies)
3.5 in. (1 agency)
4 in. (1 agency)

[36] Dense graded subbase
¾ of the lift thickness (1 agency)
0.75 in. (1 agency)
1 inch (1 agency)
1.5 in. (11 agencies)
1.75 in. (1 agency)
2-in. (8 agencies)
2.5 in. (1 agency)
3 in. (5 agencies)
3 in. in the top of the layer, 4 in. if a lower layer (1 agency)
4 in. (2 agencies)
5 in. (1 agency)
5.9 (150 mm) (1 agency)
6 in. (2 agencies)

[16] Open graded base as drainage layers
0.375 in. (1 agency)
0.75 in. (3 agencies)
1 in. (4 agencies)
1.5 in. (2 agencies)
2 in. (2 agencies)
2.25 in. (1 agency)
3 in. (1 agency)
8 in. (1 agency)
N/A (1 agency)

[12] Open graded subbase as drainage layers
0.375 in. (1 agency)
0.75 in. (1 agency)
1 in. (3 agencies)
1.5 in. (2 agencies)
2.25 in. (1 agency)

3 in. (1 agency)
4 in. (1 agency)
8 in. (1 agency)
N/A (1 agency)
[4] Other (please list)
Break run 6 in. (1 agency)
Dense graded top course - 0.5 in. (1 agency)
N/A (2 agencies)

9) **What is the maximum amount of fines (material finer than 0.075 mm or passing sieve No. 200) allowed by your agency for aggregates to be used for unbound aggregate base and subbase course applications?** *46 respondents*

[44] Dense graded base (% fines)
5 (2 agencies)
6 (1 agency)
7 (1 agency)
7.5 (1 agency)
8 (6 agencies)
9 (1 agency)
10 (8 agencies)
11 (2 agencies)
12 (11 agencies)
13 (1 agency)
15 (2 agencies)
18 (2 agencies)
20 (1 agency)
None—typ. 10–20% (1 agency)
Silicate aggregates—11%, carbonate aggregates—15% (1 agency)
specification state “well graded to dust” (1 agency)
12% for granite-derived or recycled concrete graded aggregate; 20% for marine limestone-derived graded aggregate; 30% for coquina shell base; 33% for sand-clay base (1 agency)
Up to 20% in theory. However, fines are controlled by Sand Equivalency testing and a statement “of the fraction passing the 1/4 in. sieve, 50% to 60% shall pass the No. 10 sieve.” We are in the initial stages of adding a No. 200 sieve wash requirement. (1 agency).

[35] Dense graded subbase (% fines)
6 (2 agencies)
7 (1 agency)
8 (4 agencies)
9 (1 agency)
10 (7 agencies)
12 (3 agencies)
13 (1 agency)
15 (6 agencies)
18 (2 agencies)
20 (1 agency)
25 (1 agency)
34 (1 agency)
10% (Class 4 & 5), 15% (Class 6) (1 agency)
Fines are limited by a maximum 10% passing the #100 sieve (1 agency)
None—typically 10–20% (1 agency)
Not Specified (1 agency)
Silicate aggregates—11%, carbonate aggregates—15% (1 agency)

[17] Open graded base/subbase as drainage layers (% fines)
1.5 (1 agency)
2 (4 agencies)

3 (2 agencies)
5 (4 agencies)
7 (1 agency)
10 (1 agency)
5% max passing #4 (1 agency)
3% passing the #100 sieve (1 agency)
N/A (1 agency)
Not specified but typical (1 agency)

10) Does your agency specify different allowable percent fines (material finer than 0.075 mm or passing sieve No. 200) for aggregates having nonplastic and plastic fines?

[1] Yes (please give the specified values below) - **2.2%**
[1] Base, Nonplastic (%): 8% fines
[0] Base, Plastic (%)
[0] Subbase, Nonplastic (%):
[0] Subbase, Plastic (%):
[45] No - **97.8%**
46 Respondents

11) What is the maximum value of plasticity index (PI) allowed by your agency for the fines fraction of aggregate materials to be used in the following unbound pavement layers? *46 respondents*

[44] Dense graded base:
0 (5 agencies)
3 (2 agencies)
4 (1 agency)
5 (1 agency)
6 (18 agencies)
6 for graded aggregate base and coquina base; 9 for sand-clay base (1 agency)
7 (1 agency)
8 (1 agency)
10 (1 agency)
12 (1 agency)
15 (1 agency)
AASHTO M147 (1 agency)
LL (1 agency)
N/A (4 agencies)
No specification (3 agencies)
SE 40 minimum (1 agency)
We have no PI requirement. However, the sand equivalency of "not less than 30" has helped eliminate plastic fines (1 agency)

[33] Dense graded subbase:
0 (5 agencies)
4 (1 agency)
5 (3 agencies)
6 (11 agencies)
8 (1 agency)
9 (1 agency)
12 (1 agency)
15 (1 agency)
LL (1 agency)
N/A (3 agencies)
No specification (3 agencies)
SE 35 minimum (1 agency)
We have no PI requirement. However, the sand equivalency of "not less than 25" has helped eliminate plastic fines. (1 agency)

[12] Open graded base as drainage layers:
0 (2 agencies)
2 (1 agency)
6 (3 agencies)
LL (1 agency)
N/A (3 agencies)
No specification (2 agencies)

[7] Open graded subbase as drainage layers:
0 (2 agencies)
6 (1 agency)
LL (1 agency)
N/A (2 agencies)
No specification (1 agency)

12) Does your agency have a list of approved aggregate types and sources for base and subbase course applications?

[21] Yes (please reference document/web site) - **45.7%**
[25] No - **54.3%**
[0] Other (please explain):
46 respondents

Document/website

- **Georgia:** QPL2- http://www.dot.ga.gov/doingbusiness/Materials/qpl/Documents/qpl02.pdf
- **Florida:** ftp://ftp.dot.state.fl.us/fdot/smo/website/sources/aggregatesource.pdf
- **Oregon:** ftp://ftp.odot.state.or.us/techserv/construction/TrainingManuals/MFTP/2011/09_section_4a.pdf
- **Mississippi:** http://sp.gomdot.com/Materials/Pages/Producer-Supplier.aspx
- **Alabama:** http://www.dot.state.al.us/mtweb/Testing/MSDSAR/doc/QMSD/Li01.pdf
- **Louisiana:** http://www.dotd.la.gov/highways/construction/lab/qpl/qpl%2002%20aggregates.pdf
- **Washington State:** http://www.wsdot.wa.gov/Business/MaterialsLab/ASA.htm
- **North Carolina:** https://apps.dot.state.nc.us/vendor/approvedproducts/
- **Indiana:** www.in.gov/indot
- **South Carolina:** www.scdot.org/doing/ConstructionDocs/pdfs/Materials/2%20QPL%20102411.pdf
- **Ohio:** http://www.dot.state.oh.us/Divisions/ConstructionMgt/Materials/Aggregate1/S1069%20Aggregate%20Producer%20Suppliers.pdf
- **Pennsylvania:** http://www.dot.state.pa.us/Internet/ConstrBulletins.nsf/frmBulletin14info?OpenFrameset (Material Code 203, Material Class C2A)
- **Arkansas:** http://arkansashighways.com/materials_division/Division%20300%20Bases/303020%20Aggregate%20Suppliers.pdf
- **New Jersey (for Base only):** http://www.state.nj.us/transportation/eng/materials/qualified/QPLDB.shtm

13) If you answered "Yes" to Question 12, does your agency frequently allow new materials into the list of approved aggregate types and sources for base and subbase course applications?

[14] Yes - **73.7%**
[4] No - **21.1%**
[1] Other (please explain: *As needed*) - **5.3%**
19 Respondents *(out of 21 selecting YES to Question #12)*

14) Does your agency have specific guidelines regarding the transportation and storage (stockpiling) of aggregate materials for base and subbase construction?

[17] Yes (please reference document/web site) - **37.8%**
[25] No - **55.6%**
[4] Other (please explain): - **8.9%**
46 respondents

Document/website

- **Montana:** Transport Bulk materials in vehicles that do not cause material loss or segregation
- **Indiana:** Indiana Test Method 211
- **Missouri:** MoDOT Spec. 1001.10
- **Wyoming:** Require moisture to be added in a pugmill
- **Georgia:** SOP-1 http://www.dot.ga.gov/doingbusiness/TheSource/sop/sop01.pdf
- **Texas:** Standard Specifications Item 247.4 Construction
- **Nevada:** Standard Specifications for Construction
- **Kansas:** Subsection 1100 of Standard Specifications
- **Alabama:** http://www.dot.state.al.us/mtweb/Testing/testing_manual/doc/pro/ALDOT175.pdf
- **Louisiana:** http://www.dotd.la.gov/highways/specifications/
- **Pennsylvania:** ftp://ftp.dot.state.pa.us/public/bureaus/design/pub408/pdf%20for%20printing%202011%203/106.pdf (See Sections 106.05 & 106.06)
- **Arizona:** http://azdot.gov/Highways/ConstGrp/Contractors/Useful_Information.asp (Division I—section 106.09 and 106.10)
- **New Hampshire:** http://www.nh.gov/dot/org/projectdevelopment/highwaydesign/specifications/documents/2010_Division_300.pdf
- **New York:** Section 304 in https://www.dot.ny.gov/main/business-center/engineering/specifications/updated-standard-specifications-us and GCP-17 in https://www.dot.ny.gov/divisions/engineering/technical-services/geotechnical-engineering-bureau/manuals

"Other" responses

- **Mississippi:** Samples for job acceptance are obtained from the job site
- **Florida:** Source Quality Control Plan is required to designate methods
- **Oregon:** We require plant mixing and adding water at the source.
- **Louisiana:** Look in Section Ten, 1003.03, on page 767, also part three.

15) What is the maximum construction lift thickness for unbound aggregate layers permitted by your agency?

[17] 6 in. - **37%**
[12] 8 in. - **23.9%**
[4] 10 in. - **8.7%**
[6] 12 in. - **13%**
[6] Other (please indicate): - **15.2%**
[1] No such restrictions - **2.2%**
46 respondents

"Other" responses

- 3 in., 6 in. when a vibratory steel drum roller is used
- 4 in.
- 9 in.
- Depends on the field compaction equipment being used by the contractor
- Placement thickness will be shown on plans
- Up to 24 in. total, in lifts of 3-6 in.
- Conventional dense-graded base is 4 in. by design, but will allow thicker lifts on case-by-case basis. Also have 18" rock base option, which is basically shot rock or reclaimed PCC balded and rolled into place with no sieve control other than for fines—MoDOT Spec 303.

16) Does your agency allow the construction of multiple unbound aggregate layers placed on top of each other (e.g., dense graded base over an open graded drainage layer)?

[24] Yes - **52.2%**
[12] No - **26.1%**
[10] Other (please explain): - **21.7%**
46 respondents

<u>"Other" responses</u>

- Base courses constructed in 2 lift (typically 6 in. & same material)
- Only with a geotextile separation fabric.
- We would have multiple lifts of dense graded base.
- Yes, but infrequent and discouraged.
- Occasionally—generally there is only one 6 to 8 in. crushed stone base
- Special provision
- Not really but if the compaction equipment is weak then we would require the buildup of one layer using multiple compacted layers
- Open graded drainage layer over a dense graded base or a dense graded base over a dense graded subbase
- In general, this does not apply. Special applications where large rock is used to bridge soft soils, dense graded base is allowed, but a placement of a geotextile is recommended prior to dense graded base placement.
- We used to allow unbound aggregate Open-Graded Subbase to be placed over unbound aggregate dense-graded subbase, but now have discontinued the unbound open-graded subbase in lieu of treated permeable base courses as the drainable layer above the dense-graded unbound aggregate layer beneath rigid pavements.

17) If you answered "Yes" to the above question, do you separate the two unbound aggregate layers by any kind of constructed aggregate separation (i.e., filter) layer?

[4] Yes - **16.7%**
[16] No - **66.7%**
[4] Other (please explain): - **16.7%**
24 Respondents *(out of 24 selecting YES to Question #16)*

<u>"Other" responses</u>

- Choker material is placed over crushed bedrock.
- Open graded aggregates are typically separated from soil layers with a filter fabric.
- Sometimes as required by the project special
- Sometimes-filter, geogrid, etc.

18) Does your agency allow the construction of unbound aggregate layers over or under pavement layers stabilized/treated with lime, fly ash, cement, or bitumen?

[30] Yes - **65.2%**
[10] No - **21.7%**
[6] Other (please explain): - **13%**
46 respondents

<u>"Other" responses</u>

- Allowed but seldomly used
- It is neither required nor discouraged. No one does it.
- This might be something called out in design, but no policy
- This situation has not been encountered to date.
- Yes, under
- Not as a rule—could occur in a type of sandwich layer construction if vertical grade was changing.

19) If you answered "Yes" to the above question, please give typical thicknesses of such constructed layers:

[29] Thickness (in.) of stabilized layer
3-4 in. (1 agency)
4 in. (3 agencies)
4 to 12 in. (1 agency)
6 in. (8 agencies)
6 to 8 in. (2 agencies)
6 to 12 in. (2 agencies)
6+ in. (1 agency)

7 in. for cement stabilized, 8 in. for lime stabilized (1 agency)
8 in. (1 agency)
12 in. (4 agencies)
12 to 16 in. (1 agency)
12 to 24 in. (1 agency)
18 in. (1 agency)
Not specified (site specific design thickness) (1 agency)
Variable (1 agency)

[26] Thickness (in.) of unbound aggregate layer
3 to 8 in. (1 agency)
4 in. (2 agencies)
4 to 6 in. (1 agency)
4 to 12 in. (1 agency)
> 4 in. (1 agency)
6 in. (9 agencies)
6 to 8 in. (3 agencies)
6 to 10 in. (1 agency)
8 to 10 in. (1 agency)
10 to 12 in. (1 agency)
12 in. (2 agencies)
Based on pavement structural design (1 agency)
Variable (2 agencies)

Category 2: Unbound Aggregate Base Characterization and Design

20) Who is responsible for testing/characterizing aggregate materials and providing input properties for the design of pavements with unbound granular layers?

[39] Geotechnical/Materials laboratory - **84.8%**
[1] University laboratory (under research subcontract) - **2.2%**
[8] Field Engineer - **17.4%**
[12] Other (please indicate) - **26.1%**
46 respondents

<u>"Other" responses</u>

- Assigned Structural Coefficient
- Consultants for project specific designs
- Engineering Division
- Layer coefficients provided in Design Standards
- Modulus is standard at 20,000 psi and layer coefficient of 0.10 for AASHTO 1993
- Regional Materials section/lab
- We currently use a single layer coefficient under AASHTO 93 independent of the aggregate source.
- Standard sections utilized
- Pavement sections are designed with "assumed" aggregate properties. Testing is performed to ensure minimum specifications are met.
- The materials laboratory and field engineer test aggregate materials for acceptance. The Pavement Design Engineer decides the input properties of aggregate materials for pavement design.
- Pavement Engineer specifies minimum quality levels, and the Contractor is required to provide material that meets or exceeds these requirements.
- Pavement Engineer in each district or designer with assistance from the district materials laboratory.

21) What laboratory tests are conducted by your agency for strength, deformation and modulus characterization of unbound aggregates used in base and subbase course applications? (Please check all that apply.)

[20] Strength index tests (e.g., CBR, Hveem stabilometer, etc.) - **43.5%**
[6] Triaxial shear strength tests - **13%**

[12] Repeated load triaxial tests for resilient modulus (standard tests such as AASHTO T 307, NCHRP 1-28, etc.) - **26.1%**
[2] Repeated load triaxial tests for permanent deformation behavior - **4.3%**
[21] Other (please indicate) - **45.7%**
46 respondents

“Other” responses

- Basis for design is backcalculation of moduli from FWD data.
- Depends on the existing soil being tested.
- DynaFlect, FWD
- Grain Size Analysis correlated with Mri
- None of the above (8 agencies)
- None. Will be looking at as we look more at DARWin-ME
- RLTT, for research purposes
- Standard structural number of 0.14
- Back-calculated FWD data
- Gradation and proctor for subbase. Gradations for base.
- Use uniform relative strength coefficient based on full-scale tests done by Clemson University in the late 1960s.

22) What field tests are conducted by your agency for strength, deformation and modulus characteristics of in-service unbound aggregate layers? (Please check all that apply.)

[0] Plate Load tests - 0%
[12] Dynamic Cone Penetrometer (DCP) tests - 26.1%
[30] Falling Weight Deflectometer (FWD) tests - 65.2%
[4] Light Weight Deflectometer (LWD) tests - 8.7% (MD, LA, OK, IN)
[1] Soil Stiffness Gauge tests (e.g., GeoGauge, etc.) - 2.2% (MD)
[17] Other (please indicate) - 37%
46 respondents

“Other” responses

- Compaction/Density (4 agencies)
- None for acceptance, only gradation and density
- None of the above (7 agencies)
- Roller Pattern/control strip
- We have FWD capabilities but used in a limited capacity
- We have done all of these tests, but only at the research level.
- FWD of in-service pavements but not on unsurfaced roads and not specifically to look at aggregate characteristics.

23) How often are these laboratory/field tests conducted to characterize aggregate materials for use in granular base and subbase layers?

[5] Once on limited aggregate types/materials commonly used by the agency - **10.9%**
[27] On a project-need basis - **58.7%**
[0] Once on all agency-approved aggregate sources - **0%**
[3] At regular intervals on agency-approved aggregate sources to establish a database - **6.5%**
[11] Other (please indicate) - **23.9%**
46 respondents

“Other” responses

- As needed
- FWD conducted only on local roads, per local agency request
- None (6 agencies)
- None on new materials. Nearly every project for in-service materials.
- Only when the information is needed.
- Assumed values

24) Is the structural contribution of open graded aggregate drainage layers taken into account in pavement thickness design by your agency?

[11] Yes - **23.9%**
[11] No - **23.9%**
[19] Open Graded Aggregate Drainage Layers are Not Used - **41.3%**
[5] Other (please explain) - **10.9%**
46 respondents

"Other" responses
- N/A (4 agencies)
- Used only on widening to match existing design

25) What method is used by your agency to design pavements with unbound aggregate base and subbase layers?

[4] 1972 AASHTO Design Guide - **8.7%**
[1] 1986 AASHTO Design Guide - **2.2%**
[28] 1993 AASHTO Design Guide - **60.9%**
[14] Mechanistic-Empirical Pavement Design Guide (MEPDG) - **30.4%**
[5] Agency-Specific Mechanistic Procedure: - **10.9%**
AKFPD: AK Flexible Pavement Design
Cal-ME
Modified Shell Method (M-E)
Winflex
MnPAVE, mechanistic procedure developed in Minnesota for flexible pavements - program can be found on MnDOTs Pavement Design website
[5] Agency-Specific Empirical Procedure: - **10.9%**
Idaho R-Value
MTO standard section tables
R value
https://www.dot.ny.gov/divisions/engineering/design/dqab/cpdm
Flexpave, MN Investigation 183 that is based on R-values and pavement deflection, RigidPave, rigid design based on a modified 1981 AASHTO Interim Guide Procedures, both programs can be found on MnDOTs Pavement Design website
[10] Other (please indicate) - **21.7%**
Beginning to use MEPDG
Defer to PMU
Moving toward MEPDG
Not sure
PerRoad
Contact Jeff Lambert
Getting ready to move to MEPDG
Standard sections
Our official policy calls for the use of the 1993 AASHTO Design Guide, but we also check with the MEPDG for information and back-up purposes
FPS21 (flexible pavement design system) a mechanistic-empirical system based on SCI or deflections
46 respondents *(although 67 selections were made, indicating some agencies use more than one method)*

26) What aggregate properties and/or characteristics are used as inputs for the design of pavements with granular base/subbase layers by your agency? (Please check all that apply.)

[18] Percent passing sieve sizes (gradation) and/or maximum aggregate particle size - **39.1%**
[9] Particle shape and angularity (crushed or uncrushed) - **19.6%**
[12] Compaction characteristics; i.e., optimum moisture content and maximum dry density- **26.1%**
[2] Shear strength properties (e.g., friction angle, CBR, etc.) - **4.3%**
[21] Resilient modulus - **45.7%**
[22] Other (please indicate): - **47.8%**
46 respondents

"Other" responses

- Current policy assigns a structural number to the subbase layers for flexible pavement designs.
- Defer to PMU
- Gravel Equivalence (G.E.) or back-calculated FWD modulus
- Hveem R-Value (3 agencies)
- Modulus from backcalculation of FWD data
- With respect to compliance with specifications.
- Contact Jeff Lambert
- Not sure
- Structural layer coefficient (10 agencies)
- Modulus is standard at 20,000 psi and layer coefficient of 0.10 for AASHTO 1993. Gradation, angularity, compaction characteristics, and resilient modulus are used for MEPDG.
- All subbases meeting our specifications are considered equal. It is not an efficient use of time to design pavements AFTER the contractor has chosen his subbase source.

27) What approach is adopted by your agency for assigning resilient modulus values to unbound aggregate base and subbase layers?

[10] Modulus is not used in pavement design - **21.7%**
[26] Single modulus is assigned to the entire layer - **56.5%**
[0] Stress-dependency of aggregate layer modulus is considered during pavement design - **0%**
[1] Based on research by the International Center for Aggregate Research (ICAR), anisotropy (directional dependency) of aggregate layer modulus is considered - **2.2%**
[9] Other (please indicate) - **19.6%**
46 respondents

"Other" responses

- Contact Jeff Lambert
- Defer to PMU
- MEPDG default values
- Back-calculated FWD modulus from MnROAD & various county projects around the state
- Established value
- Nil
- Not sure
- Characterization of unbound aggregate materials is being done for MEPDG local calibration. Modulus is not considered in current design methodology.

28) How does your agency determine the resilient modulus of unbound aggregate materials for use in granular base and subbase layers?
[10] Resilient modulus testing in the laboratory - **21.7%**
[23] Empirical correlations with index properties like CBR, gradation parameters, etc. - **50%**
[14] In-place modulus measurement of constructed layers by deflection-based methods such as FWD, LWD, etc. - **30.4%**
[15] Other (please indicate) - **32.6%**
46 respondents

"Other" responses

- Defer to PMU
- Do not use resilient modulus (6 agencies)
- FWD on Local jobs & grain size analysis on state jobs
- Modulus is standard at 20,000 psi and layer coefficient of 0.10 for AASHTO 1993
- Resilient modulus for base materials used for research purposes
- Resilient modulus is only used for the subgrade layer in the pavement design.
- Standard inputs correlated from AASHTO recs and typical aggregate properties
- Assumed values in design

29) Does your agency have specific guidelines for including new and/or locally available "marginal" aggregates in the thickness design of unbound base and subbase layers?

[6] Yes - **13%**
[38] No - **82.6%**
[2] Other (please explain): - **4.3%**
46 respondents

"Other" responses

- Treated base should be used where a "binder" should be incorporated.
- No—the modulus of the material is either determined or estimated from lab testing (Strength in comparison to typical materials).

Category 3: Compaction, Quality Control and Field Performance

30) How are aggregate compaction characteristics established in the lab by your agency?

[42] Drop-hammer based methods like the standard and modified Proctor tests - **91.3%**
[0] Static compaction methods - **0%**
[2] Vibratory compaction methods - **4.3%**
[0] Gyratory compaction methods - **0%**
[0] Kneading compaction methods - **0%**
[7] Other (please indicate) - **15.2%**
46 respondents

"Other" responses

- CT method
- DCP, modified Penetration Index, can be found on the MnDOTs Grading & Base website
- Proctor
- We don't establish compaction characteristics
- Method based compaction
- Not in laboratory in field
- Generally done in the field, but occasional proctor test to verify or resolve field dispute or conflict

31) List typical equipment used for placement and construction of unbound aggregate base/subbase layers in your state (please check all that apply).

[26] Dump truck - **56.5%**
[5] Material Transfer Device - **10.9%** (CA, NV, IL, MS, OH)
[39] Trucks and graders - **84.8%**
[24] Aggregate spreaders - **52.2%**
[4] Other (please indicate below) - **8.7%**
46 respondents

"Other" responses

- For big projects we may ask for the drop box
- Any of the above, depending on the job size
- Compactors, rare use of aggregate spreader
- Truck and grader for small quantity

32) Please describe briefly the preferred method of placement and construction of unbound aggregate base and subbase layers in your state.

- Use of drop box rather use of dozer
- Jersey spreader

- Deliver the aggregate to the grade, uniformly spread the uncompacted material, then compact with proper equipment until minimum compaction levels are achieved
- Up to Contractor
- Preferred would be an auto-grader, but commonly it is dump trucks and graders.
- Aggregate Spreaders, Subsection 305.3c Standard Specifications
- Plant-mixed aggregate base, moisture conditioned at the plant, and spread by graders and compacted in 6-in. lifts.
- Uniform spreading of layers
- Placement in 8 in.–12 in. lifts followed by compaction.
- Trucks and Graders
- We do not specify placement method, although spreaders are common. However, we do sample and accept the base from the roadway after placement, with emphasis on sampling areas that appear to be segregated. Contractors and suppliers must be aware and account for breakdown during compaction.
- Use of dump trucks and spreaders
- Specified depth w/near optimum moisture and mechanical densification (rolling w/pneumatic and/or steel roller in vibratory mode.
- Preferred by whom? DOTD or the contractors? We do not require specific equipment. Contractor given flexibility but must meet density requirements.
- Don't have a specified or preferred method.
- None, only concerned about avoiding segregation.
- Tucks and graders
- Material is placed by belly dumps in a windrow, blade processed while moisture conditioned, spread, and compacted to grade.
- ftp://ftp.dot.state.fl.us/LTS/CO/Specifications/SpecBook/2010Book/200.pdf
- Use of spreader box
- Contractor preference
- Truck delivery, bladed to grade, watered and compacted, finish grading, watered surface and rolled finish.
- Spread by excavator or dozer and compact with vibratory steel drum rollers in horizontal layers
- Central mixed and placed with aggregate spreader.
- Do not dictate construction methods. Contractor choice of methods. Must perform. No locations where method of placement has affected pavement life/performance.
- Graders
- Trucks and graders are most commonly used. Transfer devices or aggregate spreader to remix material help with aggregate segregation.
- Aggregate spreader and vibratory compaction
- Placement, grading or spreading and compaction
- Placed by a dump truck, spread by a grader and moisture conditioned (as needed) by a water truck. Each layer is then compacted using vibratory rollers.
- Trucks, graders, rollers
- Dump, spread, roll, final grade
- Windrow aggregate with a belly dump trailer and spread with a grader. Compact in maximum 8 in. lifts to 98% of modified Proctor.
- Specifications require adjustable, self-propelled mechanical spreaders capable of placing and screeding material without segregation with automatic grading machine trimmer.
- Base layers are placed by windrowing, using a grader to work the material to the desired crossfall while compacting with wobbly compaction equipment. Compaction requirements are based on a specified set of compaction equipment.
- Trucks and graders
- Not one preferred method. Different ways have been used and our specifications are not written in a manner to specify any one method. End result.
- Windrowed material is spread out with graders in lifts less than 6 in., and compacted
- Contractor controlled. Acceptance on lot-by-lot basis. Must meet compaction, grade requirements
- Pug-milled base material placed by mechanical spreader
- Belly dumps and graders, the method of placement is determine by the contractor
- Aggregate Spreaders
- T & G

33) How is moisture content of constructed/compacted unbound aggregate base and subbase controlled in the field?

[2] According to quarry reported moisture content - **4.3%**
[14] Sampled during construction/compaction for laboratory testing - **30.4%**
[35] Measured through field testing using (Please indicate) - **76.1%**

- 0
- AASHTO T255
- CT 226 Moisture content
- Compaction is controlled based on a control strip method.
- GDT21 http://www.dot.ga.gov/doingbusiness/TheSource/gdt/gdt021.pdf
- Indiana Test Method 506
- Nuclear compaction and within 2% of optimum
- Nuclear density method (16 agencies)
- Nuclear gauge/speedy moisture
- Sample taken from in-place material
- Speedy moisture tester
- Compacted density, moisture content
- Moisture pan & density gauge
- Nuclear device checks, but QA samples taken and dried in oven
- Nuclear or burn off method
- Using surface nuclear gauge
- Specified Density Method—not less than 65% of optimum moisture of standard proctor, Quality Compaction Method—not less than 5% of dry weight, modified Penetration Index Method - not less than 5% of dry weight, Can be found in the Standard Specifications for Construction (spec 2211) found on the MnDOT Grading & Base website

[6] Other (please indicate) - 13%
46 respondents

<u>“Other” responses</u>

- Controlled totally by contractor.
- Material is required to be damp.
- Visual
- Control of moisture content is the responsibility of the contractor; base accepted on gradation and compaction level.
- Measurement is often following compaction. Compaction control is often left to the contractor as a QC function.
- Based on visual inspection and ability to achieve a practical target density and resulting constructed density per lot placed. Procedures are outlined within the Materials Procedure governing placement and compaction of aggregate bases.

34) What is an acceptable variation from optimum moisture content for constructed/compacted unbound aggregate base and subbase allowed by your agency?

- ±2 % over optimum
- <2%
- Not specified
- Aggregate shall be maintained substantially at optimum moisture.
- N/A
- ±3% of optimum
- Plus or minus 2% of optimum
- += 2%
- ±2% is desired, but required provided dry density is attained.
- Not specified
- Do not specify.
- As required to produce a density of 100% T180
- Minus 2% to plus 1%
- Plus or minus 2% of optimum MC

- Plus 2% to minus 4%
- We don’t specify the optimum moisture content, simply that 95% of max. dry density is achieved.
- 95% density
- No moisture requirement
- Compaction controlled. No variation in moisture content specified.
- Need to obtain target density in the field
- ±2% points
- No requirement.
- Not less than 1% below optimum but as much as 2% above.
- Enough to obtain the specified compaction, but not too much to cause pumping and rutting
- 3%
- Varies but generally ±2%
- ±2%
- 0 to 5% above optimum
- There is no allowable variation for moisture. Acceptance is based on acceptable density.
- 2% of optimum
- Don’t have one.
- Not specified
- Plus or minus 2%
- No requirement on moisture content
- Optimum moisture is not specified for base and subbase.
- Required to be damp.
- N/A
- Approx 2%
- There is no target moisture content and therefore no tolerance established.
- No specified value
- +1%, -2%
- 2%
- 3 and -1 % of optimum moisture content
- No set variation; however experience shows that we end up with -2% to +1% of optimum w

35) What is the primary approach used by your agency for evaluating degree of compaction and construction quality control of unbound aggregate base and subbase layers?

[14] Gradation - **30.4%**
[3] Proof-rolling - **6.5%**
[41] Measurement of constructed layer density - **89.1%**
[3] Field measurement of constructed layer modulus - **6.5%**
[0] Continuous compaction control by means of Intelligent Compaction (IC) equipment - **0%**
[4] Other (please indicate) - **8.7%**
46 respondents

“Other” responses

- DCP, modified Penetration Index
- Non-movement under compaction equipment.
- Nuclear density gauge
- Control strip. Comparing density achieved to that achieved by the specified compaction equipment/effort.

36) If your answer to the previous question was “Measurement of constructed layer density,” what method is commonly used by your agency for measuring constructed layer densities in the field?

[39] Nuclear density methods (ASTM D 2922 / AASHTO T310 or T238) - **88.6%**
[7] Sand cone method (ASTM D 1556 / AASHTO T191) - **15.9%**
[1] Balloon method (ASTM D 2167) - **2.3%**
[0] Oil or water method - **0%**
[0] Drive cylinder method (ASTM D 2937) - **0%**

[0] Rapid method (ASTM D 5080) - **0%**
[0] Electrical density measurement method (ASTM D 7698) - **0%**
[5] Other (please indicate) - **11.4%**
44 respondents

“Other” responses

- AASHTO TP 68
- CT 216
- GDT59 http://www.dot.ga.gov/doingbusiness/TheSource/gdt/gdt059.pdf
- Illinois Modified AASHTO 310
- N/A

37) Is it common practice for your agency to construct “test strips” to establish roller patterns and check for compaction density growth of unbound aggregate base and subbase layers?

[13] Yes - **28.3%**
[31] No - **67.4%**
[2] Other (please explain) - **4.3%**
46 respondents

“Other” responses

- No, but currently considering.
- Yes, but usually part of project, start of placement.

38) If your answer to the above question was “Yes,” do you primarily use nuclear density measurement method for this purpose?

[12] Yes - **92.3%**
[0] No - **0%**
[1] Other (please explain) - **7.7%**
13 respondents

“Other” responses

- We are transitioning from the nuclear gauge to the LWD for stiffness and both are used on the test strip.

39) What is the primary mode of nuclear gauge operation used by your agency for density checks on constructed unbound aggregate base and subbase layers?

[33] Direct Transmission - **76.7%**
[7] Backscatter - **16.3%**
[5] Not applicable (nuclear density measurement not used) - **11.6%**
43 respondents

40) If your agency uses the “drop-hammer based” methods to establish compaction characteristics of aggregates in the laboratory, please select all applicable options from the list below regarding the minimum compaction requirements in the field:

	90%		**95%**		**100%**		**Other**		**Responses**
	%	#	%	#	%	#	%	#	#
Flexible Base Standard Proctor	4.3%	1	47.8%	11	21.7%	5	26.1%	6	23
Flexible Base Modified Proctor	9.1%	2	27.3%	6	18.2%	4	45.5%	10	22
Flexible Subbase Standard Proctor	4.8%	1	52.4%	11	23.8%	5	19.0%	4	21
Flexible Subbase Modified Proctor	12.5%	2	50.0%	8	12.5%	2	25.0%	4	16
Rigid Base/Subbase Standard Proctor	4.3%	1	43.5%	10	26.1%	6	26.1%	6	23
Rigid Base/Subbase Modified Proctor	11.1%	2	44.4%	8	5.6%	1	38.9%	7	18

41) If you selected "Other" as one of the responses to the above question, please specify the answer below (submit your answer in the format: Flexible Base-Standard Proctor-xx %).

- Flexible Base Modified Proctor-98%
- 98% of modified proctor
- Compact to within 97 percent of optimum for flexible and rigid

42) If your agency does NOT use "drop-hammer based" methods to establish compaction characteristics of aggregates in the laboratory, please specify the relative compaction requirements for the following unbound aggregate base/subbase applications:

	90%		95%		100%		Other		Responses
	%	#	%	#	%	#	%	#	#
Flexible Base	11.1%	1	22.2%	2	0%	0	66.7%	6	9
Flexible Subbase	0%	0	25%	2	0%	0	75%	6	8
Rigid Base/Subbase	0%	0	29.6%	8	11.1%	3	59.3%	16	27

43) If you selected "Other" as one of the responses in the above question, please specify the details below:

97% for the unbounded aggregate on top 12"
Type A Subbase requires 95%, Type A Base requires 100%; regardless of pavement type
98% of control strip density

44) Is there common interest among engineers in your agency to implement non-nuclear density measurement methods for construction quality control of unbound aggregate base/subbase layers?

[24] Yes (please select all possible reasons) - **52.2%**
[17] Safety concerns
[19] Nuclear certification too expensive
[20] Nuclear certification too inconvenient
[3] Non-nuclear methods provide better results
[19] No - **41.3%**
[8] Other (please explain): - **17.4%**
46 respondents

<u>"Other" responses</u>

- Don't know
- Speed of testing.
- We have investigated non-nuclear methods, but have not seen equivalent performance so far
- If served as better and simpler, we would use
- Reduction in staffing levels to support program
- Trying to find reliable, practical measurement of strength, not just density
- MnDOT does not use nuclear density measurement for quality acceptance, contractors can use nuclear density measure methods for process or quality control
- Some. Our agency has not started to use mechanistic design as the staple for our pavements yet. But we are starting to use it for many cases and the need to develop inputs for these methods will lead to more of a need to utilize methods to control field measurements that mirror these inputs.

45) Has your agency ever participated in demonstration projects involving continuous compaction control of unbound aggregate base/subbase layers using Intelligent Compaction (IC) techniques?

[17] Yes - **37%**
[27] No - **58.7%**
[2] Other (please explain): - **4.3%**
46 respondents

"Other" responses

- Doing that summer of 2012 (UT)
- Research yes, but not within a road that remained property of the state (DE)

46) Has your agency actively implemented IC techniques to construct in-service pavements with unbound aggregate base/subbase layers?

[1] Yes - **2.2%** (TX)
[42] No - **91.3%**
[3] Other (please explain): - **6.5%**
46 respondents

"Other" responses

- **Georgia:** Have plans to let another demonstration project in April 2012
- **Louisiana:** It is been looked at and tried on a few projects
- **Indiana:** We have just let our first contract for QC/QA Soil Embankment that requires the Contractor to implement IC Technology.

47) If your answer to the above question was "Yes," does your agency have a specification for continuous compaction control using IC techniques and construction of unbound aggregate base/subbase layers?

[1] Yes - **100%** (TX)
[0] No

48) Does your agency implement modulus-based compaction control for the construction of unbound aggregate base/subbase layers?

[2] Yes - **4.3%** (IN, GA – demo only)
[44] No - **95.7%**
46 respondents

49) If you answered "Yes" to the previous question, what method is used for measuring the field modulus of constructed unbound aggregate layers?

[0] Falling Weight Deflectometer (FWD)
[1] Light Weight Deflectometers (LWD) - **Indiana**
[0] Soil Stiffness Gauge (e.g., GeoGauge)
[0] Seismic testing such as the Portable Seismic Pavement Analyzer based on spectral analyses of surface waves
[1] Other (please indicate) – **Georgia: Intelligent Compaction-Demo only**
2 respondents

50) If you answered "Yes" to question No. 48, what percentage of construction projects in your agency involve modulus based compaction control?

[ ] <10%
[ ] 10–30%
[1] 30–60% - **Indiana**
[ ] >60%
1 respondent

51) Does your agency use any non-traditional compaction technique (e.g., South African "slushing" technique) during the construction of unbound aggregate base/subbase?

[2] Yes (please name the technique and give brief details) - **4.3%** [NM, RI]
[44] No - **95.7%**
46 respondents

"Details for Yes" responses

- **Rhode Island:** Use test strip to obtain a maximum density value by testing the same spot until the density no longer increases. Compare other values to that value.

- **New Mexico:** As indicated above, we are doing one inverted project in which we will be utilizing the slushing effort as a part of the effort.

52) Does your agency perform FWD tests to assess the structural condition of unbound aggregate base and subbase layers in existing in-service pavements?

[27] Yes - **58.7%**
[11] No - **23.9%**
[8] Other (please explain): - **17.4%**
46 respondents

"Other" responses

- Defer to PMU
- FWD is used for determination of in-place embankment resilient modulus
- For research purposes only
- For specific purposes but not routinely
- Non-standard testing, but we have performed in the past
- Only when needed by research
- Used on a limited capacity
- Project-specific and research projects

53) If your answer to the previous question was "Yes," what back calculation program/software/approach does your agency use to determine granular base/subbase modulus from the FWD test results? (Please indicate below.)

- Deflexus AARA program designed for our State
- ELMOD
- DELMAT A forward calculation based program (DELMAT) developed by Makbul Hossain of our staff
- Evercalc
- Darwin, Evercalc, BACKFA, and/or AASHTO 1993 equations
- Calback
- SCDOT-specific, similar to 1993 AASHTO
- Contact LTRC Doc Zhang (225)767-9162
- ModTag
- Modulus
- Illi-Pave
- Modulus 6.0
- MODULUS6.0
- FWD AREA Program
- Modulus
- Elmod
- ELMOD
- Internally developed at Ohio DOT
- Modtag
- I don't know
- We utilize the "modulus" backcalculation software
- DARWin Software multiplied by C = 0.25 correction factor
- ELMOD, Looking at Modtag, others
- Unknown
- Dynatest's ELMOD 6

54) What tests are commonly used by your agency to evaluate the field-performance of existing pavements with unbound aggregate base and subbase layers? (Please list all.)

- FWD
- We run IRI for HPMS and have an annual visual rating program that uses cracking, rutting, raveling, etc.
- Pavement Condition Surveys
- FWD

- Primarily a visual assessment, but FWD and coring are also done.
- FWD, DCP
- Visual assessment, coring, indirect tensile testing of cores, TSR of cores, bulk and component properties of cores, DCP, geoprobes (for samples), and FWD. Considering using a recently purchased APA for testing in-service HMAC cores.
- IRI, FWD
- None
- FWD—project specific and research projects
- FWD analysis
- Defer to PMU
- FWD
- Dyna-flect, FWD LWFWD, DCP
- FWD
- DCP, FWD
- FWD, Depth Check and Line Sampling
- FWD
- Annual condition surveys
- Layer thickness based on cores
- FWD
- FWD
- DCP, FWD, GPR
- Rideability
- FWD
- DCP, FWD
- No testing on the unbound layer of the pavement section. Measure thickness of constructed road and assign structural number to the various layers based upon accepted layer coefficients.
- FWD
- FWD with cores taken to evaluate layer thicknesses
- Not really tests. We have a pavement evaluation system
- None
- FWD along with sampling the existing materials and testing for R-Value, gradation and soil classification
- FWD on asphalt or concrete pavements
- I don't know
- We use FWD to test all of MDT's pavements on a 5-year rotation (i.e., it takes 5 years to complete all roads). We also annually access the condition of our pavements using automated pavement distress collection vans. The vans characterize pavement condition using ride (international roughness index), alligator cracking, longitudinal and transverse cracking, and rut depth.
- FWD, Ride Quality (IRI), Automated Pavement Distress Surveys
- Field performance is based on characteristics such as ride, strength, and surface distresses.
- Nil
- SPT, Compaction and FWD
- Benkelman Beam and Falling Weight Deflectometer on a trial basis at this time
- Surficial distress observations, rutting, smoothness, pavement condition evaluation
- None
- FWD
- FWD
- FWD
- No specific test, however rut depth and IRI are collected as part of PMS

Category 4: Recycling Aggregates and Recycled Granular Materials

Note: Recycled Granular Materials in this questionnaire refer to Recycled Concrete Aggregates (RCA) and Reclaimed Asphalt Pavement (RAP) materials only.

55) Does your agency commonly recycle unbound aggregate materials from base and subbase layers of existing pavements for application in new and rehabilitated pavement construction?

[24] Yes - **52.2%**
[15] No - **32.6%**

[7] Other (please explain) - **15.2%**
46 respondents

"Other" responses

- Occasionally with Full Depth Reclamation
- RAP is used a lot. RCA not as much due to environmental runoff issues and breakdown
- As needed
- Base only
- If applicable—however, we do not reconstruct very often
- Left in place and regarded for rehabilitation projects
- Used occasionally

56) If your answer to the previous question was "Yes," what tests are used by your agency for evaluating the quality of these recycled aggregates? (Please check all that apply.)

[9] $Na_2SO_4/MgSO_4$ Soundness Test - **19.6%**
[14] Los Angeles Abrasion and/or Micro Deval Test - **30.4%**
[24] Sieve Analysis - **63.0%**
[15] Percent Deleterious Materials - **32.6%**
[8] Other (please indicate): - **45.7%**
24 respondents

"Other" responses

- Limerock Bearing Ratio after removal
- PI, R value
- Plastic Index
- Rare opportunity and case-by-case
- Asphalt content
- Restrictions on RCA that can be found in spec 2211 & 3138 Standard Specifications
- We recycle from our own pavement so assume quality equal to original. Sieve analysis is used for control.
- If used as granular fill then no acceptance criteria - if used as base then would meet new requirements.

57) What other tests are used by your agency to characterize recycled aggregates from existing base and subbase courses for acceptance and design? (Please list all.)

Only 5 responses—PI, pH, R-value, % recycled pavement, % bitumen content, our primary method of recycling base course is by pulverizing it and mixing it with the existing hot mix asphalt surface, Our only requirement is that the resulting blended mix consists of no more than 50% RAP by volume.

58) Is the use of recycled aggregates from existing base and subbase courses incorporated into your agency specifications?

[21] Yes - **45.7%**
[22] No - **47.8%**
[3] Other (please explain): - **6.5%**
46 respondents

"Other" responses

- They are not prohibited, but they would need to meet same requirements as virgin aggregate.
- Not specified but common knowledge is that this is an acceptable substitution as no cost betterment.
- The final mixture of recycled aggregates from existing base and subbase courses must meet specifications.

59) Does your agency allow contractors to use locally available "marginal or out of specification" aggregates for unbound aggregate base and subbase layer applications?

[2] Yes - **4.3%**
[38] No - **82.6%**

[6] Other (please explain): - **13%**
46 respondents

"Other" responses

- Do not readily do, but has been done due to economic considerations.
- In some locations, agency specifies marginal aggregate sources for sub-base.
- Only if allowed. They would not be allowed as a substitute.
- Recycled aggregates must meet the requirements of the virgin aggregate for gradation.
- Lower quality "Stone Embankment" material is sometimes substituted during design for a sub-base.
- The gradation does not change when recycled materials are used. The virgin materials must still meet specifications, before the recycled material is added.

60) What other recycled granular materials are approved for use by your agency in unbound aggregate layer construction? (Please check all that apply.)

[31] Reclaimed Asphalt Pavement (RAP) - **67.4%**
[37] Recycled Concrete Aggregates (RCA) - **80.4%**
[10] Other (please indicate): - **21.7%**
[2] None of the above (please skip to Category 5 of the survey) - 4.3**%**
46 respondents

"Other" response

- Air cooled blast furnace slag
- Glass
- Glass, blast furnace slag
- On occasion RAP is used in shoulder areas as a substitute for aggregate base
- Blast furnace slag
- Glass
- Glass cullet
- Glass, whiteware, slags
- Glass cutlet, aggregate blended with oil field waste, others that meet specification requirements

61) What tests are used by your agency for evaluating the material quality of recycled granular materials (RCA, RAP, and/or others from question No. 60) for base and subbase applications? (Please check all that apply.)

[8] $Na_2SO_4/MgSO_4$ Soundness Test - **32%**
[12] Los Angeles Abrasion and/or Micro Deval Test - **48%**
[22] Sieve Analysis - **88%**
[11] Percent Deleterious Materials - **44%**
[3] Other (please indicate): - **12%**
25 respondents

"Other" response

- Sand equivalent (2 agencies)
- Dry rodded weight, fractured face and permeability

62) What other tests are used by your agency to characterize recycled granular materials (RCA, RAP, and/or others from question No. 60) for acceptance and design? (Please list all)

- Atterberg limits (5 agencies)
- R-value (3 agencies)
- Degradation-sand equivalent (3 agencies)
- RAP ac content (3 agencies)
- Micro-Deval
- Micro-Deval on RAP

- DCP
- FWD
- Dynaflect
- 50% RAP blend with virgin, max
- Visual observation of stockpiles
- LBR for RCA
- CBR for RCA
- RCP-pH
- Durability
- Sulfates
- Fractured faces
- pH
- RAP limited to shoulders
- Freeze/thaw soundness for RCA
- Must meet same criteria as virgin aggregate

63) Does your agency have environmental concerns regarding the use of recycled granular materials (RCA, RAP, and/or others from question No. 60) in unbound aggregate base and subbase layers?

[6] Yes - **24%**
[17] No - **68%**
[2] Other (please explain): - **8%**
25 respondents

64) If your answer to the above question was "Yes," what environmental issues is your agency particularly concerned about (e.g., leaching, etc.)?

- RCP leachate (3 agencies)
- RAP leachate
- pH levels
- Leaching (2 agencies)
- Only use RCP
- Toxicity
- Hydroxide levels

65) Does your agency require strength, deformation and modulus testing and characterization of recycled aggregates (from existing base and subbase courses) as well as recycled granular materials (RCA, RAP, and/or others from question No. 60)?

[7] Yes - **28%**
[16] No - **64%**
[2] Other (please explain): - **8%**
25 respondents

66) If you answered "Yes" to the previous question, are these characterization tests the same as those used for virgin aggregates?

[7] Yes - **100%**
[0] No (please indicate how the test methods are different)
7 respondents

Category 5: Climatic Effects and Drainage

67) Are climatic effects on pavement subgrade performance a major concern for your agency?

[28] Yes - **60.9%**
[16] No - **34.8%**

[2] Other (please explain): - **4.3%**
46 respondents

<u>"Other" response</u>

- Freeze-thaw on silty subgrade caused problems (pavement heave) on some projects
- Only changes in subgrade moisture

68) If your answer to the above question was "Yes," please check all factors that contribute to this concern: ***28 respondents***

[16] Groundwater table (GWT) is often shallow (can be less than 5 ft deep) under the pavements - **57.1%**
[22] Native soils primarily fine-grained (e.g., silts, clays, etc.) and may get wet of optimum due to upward movement of moisture from the GWT - **78.6%**
[13] In-service pavement subgrades are often under "wet of optimum" moisture conditions—**46.4%**
[24] Seasonal fluctuations cause significant changes in subgrade soil properties - **85.7%**
[17] Subgrades stay frozen for extended periods (one month or longer) - **60.7%**
[19] More than 10 freeze-thaw cycles per year are experienced at the subgrade level - **67.9%**
[17] Spring thaw weakening and timing of spring load restrictions - **60.7%**
[16] Subgrade soils are primarily frost-susceptible (i.e., silty soils) - **57.1%**

69) Are aggregate materials selected for use in granular base/subbase applications by your agency tested for climatic effects (e.g., soil water characteristic curve, freeze-thaw durability, suction characteristics of fines, etc.)?

[7] Yes (please indicate the test types) - **15.2%**
- Expansion test
- Freeze Thaw
- Idaho IT-116, Ethylene Glycol
- LAR & -#200 insoluble residue for carbonate materials
- Soundness Testing (include AASHTO T 103, Sodium Sulfate, or Brine Freeze-Thaw)
- Freeze-thaw
- Unconfined freeze-thaw durability

[36] No - **78.3%**
[3] Other (please explain): - **6.5%**
46 respondents

<u>"Other" response</u>

- Not on a regular basis. The specifications were written to minimize these problems.
- Grain size analysis to determine % silt.
- Currently testing subbase materials for information gathering including % water absorption, freeze/thaw, British frost heave.

70) Does the pavement design procedure used by your agency consider the effects of climatic changes on unbound aggregate layer performance?

[9] Yes - **19.6%**
[27] No - **58.7%**
[10] Other (please explain): - **21.7%**
46 respondents

<u>"Other" response</u>

- Adjust Structural Coefficient based on a drainage factor
- Defer to PMU
- I don't think it does.

- MEPDG designs only
- Not sure
- To the respect that subgrade stiffness affects base stiffness (modulus ratio of 3-5).
- Yes, in that our granular materials are chosen to minimize these effects.
- Contact Jeff Lambert
- Reduction in embankment Mr due to base clearance < 3 ft
- Use soaked CBR

71) If your answer to the above question was "Yes," what unbound aggregate layer properties are adjusted in your pavement design procedure to account for detrimental climatic effects on pavement performance?

[4] Layer structural coefficients - **44.4%**
[7] Resilient modulus - **77.8%**
[1] Shear strength - **11.1%**
[2] Other (please indicate): - **22.2%**
9 respondents

<u>"Other" response</u>

- Drainage coefficient
- Minimum thickness

72) Are there different gradations specified by your agency for unbound aggregate applications targeting drainable vs. low permeability aggregate layers?

[19] Yes - **41.3%**
[24] No - **52.2%**
[3] Other (please explain): - **6.5%**
46 respondents

<u>"Other" response</u>

- Don't know
- We've tried without much success.
- Yes, but typically our "drainable" layers are a soil, not aggregate. For an unbound aggregate layer, the gradation would be gap graded, not dense graded. We don't specify "low permeability" layers—these would be based upon need for pavement structure.

73) If drainage is one of the primary functions of your flexible pavement unbound aggregate base/subbase layer, what approach is adopted by your agency to facilitate the drainability of dense-graded base courses?

[17] Limiting the maximum allowable percent fines (material passing sieve No. 200) - **37%**
[2] Increasing the maximum aggregate size - **4.3%**
[6] Adjusting the constructed layer gradation toward a more open-graded layer - **13%**
[8] Other (please explain): - **17.4%**
[23] Drainage is NOT one of the primary functions of flexible pavement unbound aggregate base/subbase layers - **50%**
46 respondents

<u>"Other" response</u>

- Fines are limited by sand equivalence and split sieve gradations.
- N/A- do not use drainage layers
- None
- We install edge drain systems for all new pavement
- Minimize micaceous mineral content
- Not for flexible pavements
- Drainage layers are not part of the pavement structure. Drainage of the unbound layer would be accomplished through edge drains using open graded aggregate wrapped in filter fabric of some type.

- A drainage layer is not often designed as a layer in TX flexible pavements, but when it is gradations are more uniform (less fines and coarse).

74) Does your agency distinguish between crushed and uncrushed aggregate types while constructing open-graded drainage layers?

[14] Yes - **30.4%**
[10] No - **21.7%**
[16] Open-graded drainage layers are not used - **34.8%**
[6] Other (please explain) - **13%**
46 respondents

<u>“Other” response</u>

- 85% two face crush
- Base is required to be crushed, subbase is assumed to be uncrushed
- N/A
- Not exactly; we require uncrushed gravel to have a minimum crush count
- Unbound open-graded drainage layers were used in past, but discontinued in lieu of treated permeable base courses. The unbound open-graded and treated permeable base required minimum crushed particles.

75) How is the effectiveness of an open-graded aggregate drainage layer measured by your agency?

[2] In-situ permeability measurements - **4.3%**
[8] Laboratory tests to measure the permeability of aggregate samples - **17.4%**
[9] Empirical correlations to estimate the permeability from aggregate physical properties like gradation, dry density, specific surface, and void ratio (or porosity) - **19.6%**
[23] Open-graded drainage layers are not used - **50%**
[9] Other (please explain) - **19.6%**
46 respondents

<u>“Other” response</u>

- DRIP Program
- Historical performance is basis
- N/A
- No testing. Rarely use them.
- None
- Not measured.
- Gradation
- Nil
- Two passes with a double drum roller and a 5 gal. pail of water passes through the material before it starts to puddle.

76) For pavement structures with aggregate drainage layers, is it common practice in your agency to include a filter layer underneath to protect the drainage layer from clogging?

[5] Yes (open graded aggregates commonly used to construct the filter layer) - **10.9%**
[12] Yes (geosynthetics commonly used as the mode of filtration) - **26.1%**
[9] No (no extra layer constructed for filtration purposes) - **19.6%**
[22] Open-graded drainage layers are not used - **47.8%**
46 respondents

77) How common is it for your agency to construct subsurface drainage systems like “edge-drains” under unbound aggregate base and subbase layers?

[12] Yes, very common - **26.1%**
[21] Yes, for specific projects when required by the design - **45.7%**
[14] No, not common at all - **30.4%**
46 respondents

APPENDIX D

Review of Current Resilient Modulus Models

REVIEW OF CURRENT RESILIENT MODULUS MODELS

Resilient models of granular materials increase with increasing stress states (stress-hardening), especially with confining pressure and/or bulk stress, and slightly with deviator stress (Lekarp et al. 2000a). Resilient behavior of unbound aggregate materials can be reasonably characterized by using stress dependent models which express the modulus as nonlinear functions of stress states. Such a characterization model must include in the formulation the two triaxial stress conditions, i.e., the confining pressure σ_3 and the deviator stress σ_d or, the applied mean pressure p and the deviator stress q, to account for the effects of both confinement and shear loading. The model parameters are traditionally obtained from the multiple regression analyses of the repeated load triaxial test data. In the following subsections, currently available models are discussed in detail.

Confining Pressure Model

Seed et al. (1967) introduced a simple model for the resilient modulus relating it to confining stresses. They conducted repeated load triaxial tests on sands and gravels, and expressed the results in the form:

$$M_R = K_1(\sigma_3)^{K_2} \tag{D-1}$$

where σ_3 is confining pressure and K_1 and K_2 are regression analysis constants from experimental data. This model, however, did not give high correlation coefficients.

K-θ Model

One of the most popular models was developed by Hicks and Monismith (1971). This model, known as the K-θ model, has been the most widely used for modeling modulus as a function of stress state applicable to granular materials.

$$M_R = K(\theta)^n \tag{D-2}$$

where θ is bulk stress $= (\sigma_1 + 2\sigma_3)$ or $(\sigma_d + 3\sigma_3)$, σ_d is deviator stress $= (\sigma_1 - \sigma_3)$ and K, n are regression analysis constants obtained from experimental data. Even though it is a popular model, the K-θ model has a shortcoming since it fails to adequately distinguish the effect of shear behavior.

The impact of neglecting shear stress was illustrated by Uzan (1985) and the K-θ model predicted an increasing resilient modulus as axial strains increased in contrast to the test data that showed a decrease in resilient modulus. According to Brown and Pappin (1981), the K-θ model is not able to handle volumetric strains and therefore can only be applicable to a very limited stress range when confining pressure (σ_3) is less than deviator stress (σ_d). In addition, Nataatmadja (1989) reported that this model was not dimensionally satisfied as K had the same dimension with resilient modulus (M_R). Despite of this weakness, the K-θ model is still being used frequently for granular materials because of its simplicity.

Shackel's Model

After conducting repeated load triaxial tests on a silty-clayey soil, Shackel (1973) developed the following resilient modulus model in terms of octahedral shear stress and octahedral normal stress

$$M_R = K_1\left[\frac{(\tau_{oct})^{K_2}}{(\sigma_{oct})^{K_3}}\right] \tag{D-3}$$

where K_i are material regression constants obtained from triaxial test data. He proposed that his model was valid for both granular materials and cohesive soils. Since the model was defined in terms of stress invariants, it was considered to be one of the early advanced nonlinear models.

$$\sigma_{oct} = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3) = \frac{1}{3}I_1 \tag{D-4}$$

$$\begin{aligned}\tau_{oct} &= \frac{1}{3}\left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2\right]^{\frac{1}{2}} \\ &= \frac{\sqrt{2}}{3}(I_1^2 - 3I_2)^{\frac{1}{2}}\end{aligned} \tag{D-5}$$

where I_1 is the first stress invariant and I_2 is the second invariant.

Bulk-Shear Modulus Model

Boyce (1980) developed a nonlinear material model based on the secant bulk modulus (K) and the shear modulus (G). He found the influence of mean normal stress to resilient strain and the relationships were given as:

$$K = K_i p^{(1-n)} \tag{D-6}$$

$$G = G_i p^{(1-n)} \tag{D-7}$$

where K_i is an initial value of bulk modulus, G_i is an initial value of shear modulus and n is a constant less than 1.0. Boyce (1980) also updated his model to satisfy Maxwell's reciprocity theorem. Accordingly, the second order partial derivatives of a stress potential function are independent of the order of differentiation of volumetric and deviatoric stress components. Expressions of the moduli were given as follows:

$$K = \frac{K_i p^{(1-n)}}{1-\beta\left(\frac{q}{p}\right)^2} \quad \text{(D-8)}$$

$$G = G_i p^{(1-n)} \quad \text{(D-9)}$$

where β is $(1-n)\frac{K_i}{6G_i}$, p is mean stress, q is deviator stress. In this model, the volumetric strains and deviatoric strains are related to mean normal stress (p) and deviatoric stress (q) as follows:

$$\varepsilon_v = \left(\frac{1}{K_i}\right)p^n\left[1-\beta\left(\frac{q}{p}\right)^2\right] \quad \text{(D-10)}$$

$$\varepsilon_q = \left(\frac{1}{3}G_i\right)p^n\left(\frac{p}{q}\right) \quad \text{(D-11)}$$

where ε_v and ε_q are the volumetric and shear strains, respectively. This model can successfully predict measured strains from the initial bulk and shear moduli and the applied stress states.

Uzan Model

Since the K-θ model was not sufficient to describe the shear behavior of granular materials, Uzan (1985) made a modification to this model. An additional deviator stress component that includes the effect of shear behavior was shown to be in good agreement with test results.

$$M_R = K_1(\theta)^{K_2}(\sigma_d)^{K_3} \quad \text{(D-12)}$$

where θ is bulk stress $= (\sigma_1 + 2\sigma_3)$ or $(\sigma_d + 3\sigma_3)$, σ_d is deviator stress $= (\sigma_1 - \sigma_3)$, and K_1, K_2, and K_3 are regression analysis constants obtained from experimental data. Considering in the formulation both bulk and deviator stresses, the Uzan model overcomes the deficiency of the K-θ model that did not include shear effects and fits better with the test data than the K-θ model. This was shown to be especially important when confining stress values applied on the specimen were larger than the applied deviator stresses during testing.

Lade and Nelson Model

Lade and Nelson (1987) proposed an elastic material model based on energy conservation for closed-loop strain path. In this model, isotropic and nonlinear assumption was used in the elastic behavior of granular materials. With the assumption of energy conservation, the work during any arbitrary closed path stress cycle was written as:

$$W_{\text{cycle}} = \oint_{\text{cycle}} dW = \oint_{\text{cycle}} \left(\frac{I_1}{9K}dI_1 + \frac{dJ_2}{2G}\right) = 0 \quad \text{(D-13)}$$

where K is bulk modulus, G is shear modulus, I_1 is the first stress invariant, and J_2 is the second deviatoric invariant. The first order partial differential equation from Equation 5.24 is derived as follows:

$$\frac{I_1}{9K^2}\frac{\partial K}{\partial\sqrt{J_2}} = \frac{\sqrt{J_2}}{G^2}\frac{\partial G}{\partial I_1} \quad \text{(D-14)}$$

After substituting $K = \frac{E}{3(1-2\nu)}$ and $G = \frac{E}{2(1+2\nu)}$ into Equation (D-14), the equation can be expressed in terms of E (Young's modulus).

$$\frac{1}{\sqrt{J_2}}\frac{\partial E}{\partial\sqrt{J_2}} = R\frac{1}{I_1}\frac{\partial E}{\partial I_1} \quad \text{(D-15)}$$

where $R = \frac{6(1+\nu)}{(1-2\nu)}$. The final form of the stress-dependent modulus equation was proposed as follows:

$$E = Mp_a\left[\left(\frac{I_1}{p_a}\right)^2 + R\frac{J_2}{p_a}\right]^{\lambda} \quad \text{(D-16)}$$

where p_a is atmospheric pressure and M and λ are material constants. This Lade and Nelson model did not give good results due to the energy conservation principles adopted in this hyperelastic material model formulation since energy dissipates when granular materials are subjected to repeated loading.

Universal Octahedral Shear Stress Model

Witczak and Uzan (1988) proposed an improvement over the Uzan (1985) model by replacing the deviator stress term with octahedral shear stress. This model also used atmospheric pressure (p_a) to normalize the bulk and shear stress terms to make the model parameters dimensionless.

$$M_R = K_1 p_a\left(\frac{I_1}{p_a}\right)^{K_2}\left(\frac{\tau_{\text{oct}}}{p_a}\right)^{K_3} \quad \text{(D-17)}$$

where I_1 is first stress invariant $= (\sigma_1 + \sigma_2 + \sigma_3)$ or $(\sigma_1 + 2\sigma_3)$, τ_{oct} is octahedral shear stress $= \frac{1}{3}\{(\sigma_1-\sigma_2)^2 + (\sigma_1-\sigma_3)^2 + (\sigma_2-\sigma_3)^2\}^{1/2} = \frac{\sqrt{2}}{3}(\sigma_1-\sigma_2)$, p_a is atmospheric pressure, and K_1, K_2, and K_3 are regression constants obtained from experimental data.

Itani Model

An improved correlation between the resilient modulus and various stress state variables, such as deviator stress, mean stress, confining stress, and axial strain, was obtained from multiple regression analyses. Itani (1990) proposed the material model with a high correlation coefficient ($R^2 = 0.96$) as follows:

$$M_R = K_1 p_a \left(\frac{\sigma_\theta}{p_a}\right)^{K_2} (\sigma_d)^{K_3} (\sigma_3)^{K_4} \tag{D-18}$$

where $\sigma\theta = (\sigma_1 + \sigma_2 + \sigma_3) = (\sigma_1 + 2\sigma_3)$, $\sigma_d = \sigma_1\text{-}\sigma_3$, σ_3 is confining stress, p_a is atmospheric pressure, and K_1, K_2, K_3 and K_4 are multiple regression constants obtained from triaxial tests. With the goal of developing improved models to characterize the resilient modulus, laboratory test data from different aggregate gradations were used in this study. Itani (1990) concluded that this model was useful to predict resilient modulus, although there was a slight multi-colinearity problem. This is due to the fact that two independent triaxial stress states are expressed in three stress terms in this equation.

Crockford et al. Model

Crockford et al. (1990) developed a resilient modulus model which was expressed as a function of volumetric water content, suction stress, octahedral shear stress, unit weight of material normalized by the unit weight of water, and the bulk stress. The model was proposed as follows:

$$M_R = \beta_0 \left(\theta + 3\Psi \frac{V_W}{V_t}\right)^{\beta 1} (\tau_{\text{oct}})^{\beta_2} \left(\frac{\gamma}{\gamma_W}\right)^{\beta_4} \tag{D-19}$$

where β_0, β_1, β_2, and β_3 are material constants, ψ is suction stress, $\frac{V_W}{V_t}$ is volumetric water content, τ_{oct} is octahedral shear stress, and $\frac{\gamma}{\gamma_W}$ is unit weight of material normalized by the unit weight of water. When eliminating moisture term and the normalized unit weight term, this equation simplifies to the octahedral shear stress model of Witczak and Uzan (1988).

UT-Austin Model

UT-Austin model was developed by Pezo (1993) with a good agreement of the resilient modulus data from the repeated load triaxial test. This model predicts the response variable, axial strain, instead of the resilient modulus using the applied confining and deviator stresses. Since this model is independent of the response variables, it is very useful for any condition.

$$M_R = \frac{\sigma_D}{\varepsilon_r} = \frac{\sigma_d}{a\sigma_d^b \sigma_3^c} = \frac{1}{a}\left(\sigma_d^{1-b}\sigma_3^{-c}\right) = K_1 (\sigma_d)^{K_2} (\sigma_3)^{K_3} \tag{D-20}$$

where σ_d is deviator stress = $(\sigma_1 - \sigma_3)$, σ_3 is confining stress and K_1, K_2 and K_3 are regression analysis constants obtained from experimental data.

Lytton Model

Lytton (1995) proposed that the principles of unsaturated soil mechanics could be applied to the universal octahedral shear stress model (Witczak and Uzan 1988) because unbound aggregate materials in pavements are normally unsaturated. To evaluate the effective resilient properties of unsaturated granular materials, Lytton added a suction term to the universal octahedral shear stress model as follows:

$$M_R = K_1 p_a \left(\frac{I_1 - 3\theta f h_m}{p_a}\right)^{K_2} \left(\frac{\tau_{\text{oct}}}{p_a}\right)^{K_3} \tag{D-21}$$

where p_a is atmospheric pressure, I_1 is first stress invariant $= (\sigma_1 + \sigma_2 + \sigma_3)$, is volumetric water content, f is function of the volumetric water content, h_m is matric suction, τ_{oct} is octahedral shear stress $= ⅓\{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2\}^{1/2}$, and K_1, K_2, and K_3 are multiple regression constants obtained from triaxial tests.

NCHRP 1-37A Mechanistic Empirical Pavement Design Guide (MEPDG) Model

In the MEPDG (NCHRP 1-37A, 2004), a generalized constitutive model was adopted to characterize the resilient modulus of unbound aggregates. This equation combines both the stiffening effect of bulk stress and the softening effect of shear stress. Thus, the values of K_2 should be positive, since increasing the bulk stress produces a stiffening of the material. However, K_3 should be negative to show a softening effect. To properly find the model constants, multiple correlation coefficients determined from test results have to exceed 0.90. Note that this model is proposed for use with both unbound aggregates and fine-grained subgrade soils.

$$M_R = K_1 p_a \left(\frac{\theta}{p_a}\right)^{K_2} \left(\frac{\tau_{\text{oct}}}{p_a} + 1\right)^{K_3} \tag{D-22}$$

where is the bulk stress $= \sigma_1 + \sigma_2 + \sigma_3$, τ_{oct} is octahedral shear stress $= ⅓\{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2\}^{1/2}$, p_a is atmospheric pressure, and K_1, K_2, and K_3 are constants obtained from experimental data.

REFERENCES

Boyce, J.R., "A Nonlinear Model for the Elastic Behavior of Granular Materials Under Repeated Loading," International Symposium on Soils Under Cyclic and Transient Loading, Swansea, 1980.

Brown, S.F. and J.W. Pappin, "Analysis of Pavements with Granular Bases," *Transportation Research Record 810,* Transportation Research Board, National Research Council, Washington D.C., 1981, pp. 17–23.

Crockford, W.W., L.J. Bendana, W.S. Yang, S.K. Rhee, and S.P. Senadheera, *Modeling Stress and Strain States in Pavement Structures Incorporating Thick Granular Layers,* Final Report, Contract F08635-87-C-0039, The Texas Transportation Institute, The Texas A&M University, College Station, 1990.

Hicks, R.G. and C.L. Monismith, "Factors Influencing the Resilient Properties of Granular Materials," *Transportation Research Record 345,* Transportation Research Board, National Research Council, Washington D.C., 1971, pp. 15–31.

Itani, S.Y., *Behavior of Base Materials Containing Large-Sized Particles,* PhD Thesis, School of Civil and Environmental Engineering, Georgia Institute of Technology, Sep. 1990.

Lade, P.V. and R.B. Nelson, "Modeling the Elastic Behavior of Granular Materials," *International Journal for Numerical and Analytical Methods in Geomechanics,* Vol. 11, 1987, pp. 521–542.

Lytton, R.L., "Foundations and Pavements on Unsaturated Soils," 1st International Conference on Unsaturated Soils, Paris, France, 1995.

Nataatmadja, A., *Variability of Pavement Material Parameters under Repeated Loading,* PhD Dissertation, Monash University, Australia, 1989.

NCHRP 1-37A, *Mechanistic-Empirical Pavement Design Guide,* Draft Report, Part 2 Design Inputs, 2004.

Pezo, R.F., "A General Method of Reporting Resilient Modulus Tests of Soils—A Pavement Engineer's Point of View," Paper No: 93082, 72nd Annual Meeting of the Transportation Research Board, Washington, D.C., 1993.

Seed, H.B., F.G. Mitry, C.L. Monismith, and C.K. Chan, *NCHRP Report 35: Prediction of Flexible Pavement Deflections from Laboratory Repeated Load Tests,* Highway Research Board, National Research Council, Washington, D.C., 1967.

Shackel, B., "Repeated Loading of Soils–A Review," *Australian Road Research,* Vol. 5, No. 3, 1973, pp. 22-49.

Uzan, J., "Characterization of Granular Material," *Transportation Research Record 1022,* Transportation Research Board, National Research Council, Washington D.C., 1985, pp. 52–59.

Witczak, M.W. and J. Uzan, *The Universal Airport Pavement Design System, Report I of V: Granular Material Characterization,* Department of Civil Engineering, University of Maryland, College Park, Md., 1988.

APPENDIX E

Review of Current Permanent Deformation Models

Constitutive relationships often need to be developed to properly describe permanent deformation accumulation in unbound granular materials with number of load applications. In this section, a summary is given of the different models proposed by many researchers to predict permanent strain as a function of load and material property related factors.

Barksdale Model

Barksdale (1972) analyzed standard repeated load triaxial data to propose a linear relationship between permanent axial strain and the logarithm of number of load applications as shown below:

$$\varepsilon_p = a + b\log(N) \qquad \text{(E-1)}$$

Where ε_p is the axial permanent strain; N is the number of load applications; a and b are model parameter estimates from linear regression of laboratory experimental data.

Phenomenological Model

Monismith et al. (1975) proposed a log-log relationship between axial permanent strain and the number of load applications as shown in Equation (E-2). This model, also known as the phenomenological model, is used widely to present permanent deformation test results from laboratory experiments.

$$\varepsilon_p = AN^b \qquad \text{(E-2)}$$

where the definitions of e_p, N, A (or a) and b are the same as given above.

Note that researchers (Monismith et al. 1975; Maree 1978) have proposed a value less than unity (1.0) for the regression parameter "b" for stress conditions significantly below the shear strength of the material [30, 35]. However, a value of "b" that is less than unity (1.0) would imply a permanent deformation accumulation rate of infinity (∞) for the first load application ($N = 1$), and zero for large values of N. This also implies that the "A" parameter represents an asymptote for the accumulated permanent deformation for large values of N. Note that asymptotic permanent deformation response is typical of unbound aggregate behavior in the "plastic shakedown" range (Werkmeister 2003). Therefore, the phenomenological model can predict material behavior accurately only for stress levels below the plastic shakedown limit.

Thompson and Nauman (1993) observed that the "A" term in the phenomenological model was significantly affected by stress states ("A" values typically increased with increasing stress levels), whereas the "b" parameter varied in the range between 0.12 and 0.20 for different granular material types.

Strain Rate Model

El-Mitiny (1980) and Khedr (1985) proposed the strain rate model, which was related to the phenomenological model. The strain rate model inversely correlates the rate of permanent axial strain to the logarithm of the number of load repetitions as follows:

$$\frac{\varepsilon_p}{N} = aN^{-b} \qquad \text{(E-3)}$$

where the definitions of ε_p, N, A (or a) and b are the same as given above.

Tseng and Lytton Model

Tseng and Lytton (1989) presented a three-parameter permanent deformation model to predict the accumulation of permanent deformation through material testing. The parameters were developed from the laboratory established relationship between permanent strains and the number of load applications. The curve relationship is expressed as follows:

$$\varepsilon_a = \varepsilon_0 e^{-\left(\frac{\rho}{N}\right)^{\beta}} \qquad \text{(E-4)}$$

Where ε_a is the axial permanent strain; N is the number of load applications, ε_0, β, and ρ are material parameters that are different for each sample, and are determined based on the water content, resilient modulus, and stress states for base aggregate and subgrade soils through multiple regression analyses.

Wolff Model

Wolff (1992) developed the following model to predict permanent strain accumulation in aggregate base and subbase layers from Heavy Vehicle Simulator (HVS) test data.

$$\varepsilon_p = (mN + a)\left(1 - e^{-bN}\right) \qquad \text{(E-5)}$$

where ε_p is the axial permanent strain; N is the number of load application; and a, b, and m are model parameters. The primary feature of Wolff's model is that it accounts for the initial rapid increase in permanent deformation followed by a linear phase in which the permanent deformation increases at a steady rate. Upon differentiating the above expression to study the rate of accumulation of permanent strain $\left(\partial\varepsilon_p / \partial N \right)$, one can see that the incremental permanent deformation is equal to $(a \times b)$ for $N = 0$, and approaches "m" as $N \rightarrow \infty$.

Rutting Rate Model

Thompson and Nauman (1993) proposed a practical application of the above model. They used rut depths obtained from field measurements instead of the permanent axial strain term as follows:

$$RR = \frac{RD}{N} = aN^b \tag{E-6}$$

where RR = Rutting rate; RD = Rut depth; N = Number of load applications; a, b = Model Parameters. Thompson and Nauman (1993) successfully applied their rutting rate model to prediction of the AASHO Road Test section rutting performances.

Van Niekerk and Huurman Model

Note that none of the above discussed models accounted for the effects of stress states on the accumulation of permanent deformation in unbound aggregates. Accordingly, van Niekerk and Huurman (1995) proposed the following relationship between plastic strain and the number of load repetitions for unbound granular materials:

$$\varepsilon_p = a_1 \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{a_2} \left(\frac{N}{1000} \right)^{b_1 \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{b_2}} \tag{E-7}$$

where ε_p is the permanent or plastic strain; N is the number of load applications; σ_1 is the major principal stress, $\sigma_{1,f}$ is the major principal stress at failure; and a_1, a_2, b_1, and b_2 are model parameter estimates. This model accounts for the effect stress state on permanent deformation by incorporating the ratio between applied deviator stress, and the deviator stress at failure for a triaxial specimen. Note that as the ratio $\left(\sigma / \sigma_{1,f} \right)$ is kept constant for a particular test, the above equation is essentially the same as the phenomenological model is Equation (E-2).

Paute Model

Paute et al. (1996) suggested the following relationship between the number of load applications and the accumulation of permanent deformation after 100 cycles, considering the maximum permanent axial strain possible, depicted as "a" in the model:

$$\varepsilon_p = a \left(1 - \left(\frac{N}{100} \right)^{-b} \right) \tag{E-8}$$

The model parameter definitions are the same as above. Note that Paute's model excluded the rapid rate part of permanent deformation accumulation between the 1st and 100th cycle. This is in accordance with the difficulty of predicting the permanent deformation development within the first 100 cycles which often corresponds to the rapid reorientation of individual particles in the aggregate matrix.

Huurman Model

Huurman (1997) combined stress level and number of load applications into one expression to predict the accumulation of permanent deformation in unbound granular materials. Equation shows the model proposed by Huurman.

$$\varepsilon_p = A \left(\frac{N}{1000} \right)^B + C \left(\exp\left(D \frac{N}{1000} \right) - 1 \right) \tag{E-9}$$

Where the parameters A, B, C, and D account for the stress dependency of permanent strains as shown below:

$$X = x_1 \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{x_2} \tag{E-10}$$

where X is a variable representing each parameter A, B, C, or D in Equation E-9; x_1 and x_2 are variables representing related coefficients a_1, a_2, b_1, b_2, c_1, c_2, d_1, and d_2, respectively.

Ullidtz Model

Ullidtz (1997) proposed a stress related permanent model which expresses the accumulated permanent strain in terms of the applied deviator stress and the number of load applications for a triaxial specimen. Equation (E-11) shows the model proposed by Ullidtz.

$$\varepsilon_p = a \left(\frac{\sigma_d}{p_0} \right)^b N^c \tag{E-11}$$

where σ_d is the axial deviator stress, p_0 is the normalizing reference stress (often p_0 = 1 psi or 1 kPa), a, b, and c are model parameter estimates obtained from regression analyses of experimental data.

Lekarp and Dawson Model

Lekarp and Dawson (1998) used the shakedown concept to investigate the effect of stress state on permanent deformation development, and proposed the following model relating the permanent strain accumulation to the maximum shear stress ratio and the length of the stress path:

$$\frac{\varepsilon_p\left(N_{\text{ref}}\right)}{\left(L/p_0\right)} = a\left(\frac{q}{p}\right)_{\max}^{b} \tag{E-12}$$

where: ε_p (N_{ref}) is the permanent axial strain at a given reference number of cycles N_{ref}, where $N_{\text{ref}} > 100$; L is the length of stress path, p is the mean normal stress $\left(p = (\sigma_1 + \sigma_2 + \sigma_3)/3\right)$; q is the deviatoric stress ($q = \sigma_1 - \sigma_3$); $(q/p)_{\max}$ is the maximum stress ratio; p_0 is the normalizing reference stress; N is the number of load applications; a and b are model parameter estimates. Although this model included several load related variables, it did not consider the stress path-direction and loading slope which can influence the permanent deformation accumulation.

REFERENCES

Barksdale, R.D., "Laboratory Evaluation of Rutting in Base Course Materials," *Proceedings of the 3rd International Conference on Asphalt Pavements, University of Michigan,* Ann Arbor, 1972, pp. 161–174.

El-Mitiny, M.R., *Material Characterization for Studying Flexible Pavement Behavior in Fatigue and Permanent Deformation,* PhD Thesis, Ohio State University, Columbus, 1980.

Huurman, M., *Permanent Deformation in Concrete Block Pavements,* Ph.D. Thesis, Delft University of Technology, Delft, the Netherlands, 1997.

Khedr, S., "Deformation Characteristics of Granular Base Course in Flexible Pavement," *Transportation Research Record 1043,* Transportation Research Board, National Research Council, Washington, D.C., 1985, pp. 131–138.

Lekarp, F. and A. Dawson, "Modeling Permanent Deformation Behavior of Unbound Granular Materials," *Construction and Building Materials,* Vol. 12, No. 1, 1998, pp. 9–18.

Monismith, C.L., N. Ogawa, and C.R. Freeme, "Permanent Deformation Characteristics of Subgrade Soils Due to Repeated Loading," *Transportation Research Record 537,* Transportation Research Board, National Research Council, Washington, D.C., 1975, pp. 1–17.

Maree, J.H., *Design Parameters for Crushed Stone in Pavements,* MS Thesis, University of Pretoria, South Africa, 1978.

Paute, J.L., P. Hornych, and J.P. Benaben, "Repeated Load Triaxial Testing of Granular Materials in the French Network of Laboratoires des Ponts et Chaussées," *Flexible Pavements, Proceedings of the European Symposium Euroflex 1993,* A.G. Correia, Ed., Balkema, Rotterdam, the Netherlands, 1996, pp. 53–64.

Thompson, M.R. and D. Nauman, "Rutting Rate Analyses of the AASHO Road Test Flexible Pavements," *Transportation Research Record 1384,* Transportation Research Board, National Research Council, Washington, D.C., 1993, pp. 36–48.

Tseng, K.-H. and R.L. Lytton, "Prediction of Permanent Deformation In Flexible Pavement Materials," *Implication of Aggregates In the Design, Construction, and Performance of Flexible Pavements,* ASTM STP 1016, ASTM, Philadelphia, Pa., 1989, pp. 154–172.

Ullidtz, P., "Modeling of Granular Materials Using the Discrete Element Method," *Proceedings of the 8th International Conference on Asphalt Pavements,* Seattle, Wash., 1997, pp. 757–769.

Van Niekerk, A.A. and M. Huurman, *Establishing Complex Behavior of Unbound Road Building Materials from Simple Material Testing,* Technical Report No. 7-95-200-16, Delft University of Technology, Delft, the Netherlands, 1995.

Werkmeister, S., *Permanent Deformation Behaviour of Unbound Granular Materials in Pavement Constructions,* PhD Thesis, Dresden University of Technology, Dresden, Germany, 2003.

Wolff, H., *Elasto-Plastic Behavior of Granular Pavement Layers in South Africa,* Ph.D. Thesis, University of Pretoria, Pretoria, Republic of South Africa, 1992.

APPENDIX F

Follow-up Survey on Resilient Modulus Testing

SURVEY RESULT BRIEF: 46 TOTAL RESPONDENTS – *14 INDICATED DOING M_r* TESTING

Idaho Transportation Department: Idaho uses a state-specific mechanistic design procedure (Winflex), which uses the Idaho R-Value as the main input parameter for base layers. They are considering a switch to MEPDG and are currently running companion tests (Idaho R-value and *Mr*) on subgrade soils and some selected base materials for the purpose of improving on Level 3 inputs/defaults. The Materials Group has a GeoComp Test System, which is used experimentally to conduct resilient modulus tests using the AASHTO T307 method. This system uses one of two triaxial chambers (for either a 4-in. diameter × 8-in. height or 6-in. diameter × 12-in. height cylindrical test specimen) manufactured by GeoComp with externally mounted transducers (location of load cell and LVDTs). Idaho uses the bulk stress model ($M_r = K1\ (\theta)^{K_2}$) to characterize stiffness as interpreted from the T307 data.

Maryland State Highway: Maryland currently uses the 1993 AASHTO Guide design procedure and selects from a range of structural layer coefficients (0.08–0.14 with 0.11 typical) as the main input parameter for base layers. The Materials Group has a GeoComp Test System that is used experimentally to conduct resilient modulus tests using the AASHTO T307 method. This system uses a triaxial chamber for 6-in. diameter × 12-in. height cylindrical test specimen, manufactured by GeoComp, with externally mounted transducers (location of load cell and LVDTs). Maryland uses the universal model ($M_r = K1p_a\ (\theta/p_a)^{K_2}\ ((\tau_{oct}/p_a) + 1)^{K_3}$) to characterize stiffness as interpreted from the T307 data. They have modeled materials from 30 quarries thus far, and are considering a switch to MEPDG. They consider the characterization of base materials critical with respect to the MEPDG, and are also using the lab data to confirm selection of structural layers coefficients used in their current designs. Additionally, they perceive the test will be a valuable discriminator when considering alternative materials such as RAP, RCP, and other sources.

Arkansas State Highway and Transportation Department: Arkansas currently uses the 1993 AASHTO Guide design procedure and selects a structural layer coefficient as the main input parameter for base layers. Although the Materials Group has an Instron Test System capable of conducting the resilient modulus test on base materials, they currently only perform such tests (AASHTO T307 method) on subgrade soils. A research project was performed by a local university to characterize base materials using resilient modulus, but this test is not a current practice in Arkansas.

Oklahoma Department of Transportation: Although the Materials Group has an MTS Test System capable of conducting the resilient modulus test on base materials, they currently only perform such tests (AASHTO T307 method) on subgrade soils. In years past, they used a Trautwein triaxial chamber for testing 6-in. diameter × 12-in. height cylindrical test specimen with externally mounted transducers (location of load cell and LVDTs), but found the process of remolding quality test specimens to be burdensome. Oklahoma currently uses the 1993 AASHTO Guide design procedure and uses a range of structural layer coefficients (0.10–0.14 based upon gradation) as the main input parameter for base layers. They are transitioning to MEPDG, and currently use 35,000 psi as an input for base layer modulus.

Virginia Department of Transportation: Virginia currently uses the 1993 AASHTO Guide design procedure and selects a structural layer coefficient as the main input parameter for base layers. Although the Materials Group has an Instron Test System capable of conducting the resilient modulus test on base materials, they currently only perform such tests (AASHTO T307 method) on subgrade soils. A research project was performed by a consultant (Instron Test System) to characterize base materials using resilient modulus (6 sources), but this test is not a current practice in Virginia.

Mississippi Department of Transportation: Mississippi currently uses the 1993 AASHTO Guide design procedure and selects a structural layer coefficient as the main input parameter for base layers. A research project was performed by a consultant (Interlaken Test System) to characterize base materials using resilient modulus, but this test is not a current practice in Mississippi. They are transitioning to MEPDG, but most likely will rely on backcalculated base layer values as input for new designs.

Utah Department of Transportation: Utah currently uses the 1993 AASHTO Guide design procedure and selects a structural layer coefficient as the main input parameter for base layers. A research project was performed by a local university (IPC Test System) to characterize base materials using resilient modulus, but this test is not a current practice in Utah.

North Carolina Department of Transportation: North Carolina currently uses the 1972 AASHTO Interim Guide design procedure and a structural layer coefficient (0.14) as the main input parameter for base layers. The Materials Group has an Instron Test System which is used experimentally to conduct resilient modulus tests using the AASHTO T307 method. This system uses a triaxial chamber for 6-in. diameter × 12-in. height cylindrical test specimen, manufactured by Karol-Warner, with externally mounted transducers (location of load cell and LVDTs). North Carolina uses the SHRP Model ($M_r = K1(S_C)^{K_2}(S_3)^{K_5}$) to characterize stiffness as interpreted from the T307 data. They are considering a switch to MEPDG, and intend to use the experimental data from their lab program to determine a single value to use as input. Presently, they are using 40,000 psi, but are fine-tuning and calibrating this value.

Missouri Department of Transportation: Missouri shifted to the MEPDG design procedure in 2004 and currently uses resilient modulus as the main input parameter for base layers. A research project was performed by a local university (MTS Test System – GCTS 6-inch triaxial cell) to characterize base materials using resilient modulus. Missouri uses the universal model ($M_r = K1p_a\ (\theta/p_a)^{K_2}\ ((\tau_{oct}/p_a)+1)^{K_3}$) to model stiffness as interpreted from the T307 research data, and have used this study to calibrate models and supplement and/or replace Level 3 lookup values since the 2004 implementation. The resilient modulus test is not considered a routine practice in Missouri.

South Dakota Department of Transportation: South Dakota currently uses the 1993 AASHTO Guide design procedure and is considering using the MEPDG. They currently do not have the capability to perform resilient modulus tests to characterize base materials. If the need arises, they would contract the work to a university or consultant.

Wisconsin Department of Transportation: Wisconsin currently uses the 1972 AASHTO Guide design procedure and is considering using the MEPDG. They currently do not have the capability to perform resilient modulus tests to characterize base materials. In preparation for the MEPDG, Wisconsin has initiated some research work in this specific area to local universities, who conduct AASHTO T307 testing.

Indiana Department of Transportation: Although the Materials Group has a GeoComp Test System capable of conducting the resilient modulus test on base materials, they currently only perform such tests (AASHTO T307 method) on subgrade soils. This is due to the fact that they do not possess a large triaxial chamber for conducting tests on large-diameter specimens. Indiana has implemented the MEPDG procedure and currently uses 30,000 psi as the main input parameter for their base layer, based primarily on backcalculated values of existing pavement structures. They recognize this general use default value is restrictive and are exploring options to add robustness through research (develop a catalog of values) or active testing.

Kansas Department of Transportation: Kansas currently uses the 1993 AASHTO Guide design procedure and selects a structural layer coefficient as the main input parameter for base layers. Although the Materials Group has an Interlaken Test System capable of conducting the resilient modulus test on base materials, they have not conducted tests for nearly 10 years.

Minnesota Department of Transportation: Minnesota DOT currently uses FlexPave (R-value inputs) and RigidPave for flexible and rigid pavement designs, respectively. The Materials Group (The Research Group) has an Interlaken Test System, which is used currently (experimentally) to conduct resilient modulus tests using the NCHRP 1-28a method for research projects (although 3 internally-mounted LVDTs are used rather than the recommended 2 LVDTs, and a rigorous QA/QC protocol has been established to readily scan data for acceptance). This system uses a triaxial chamber for 6-in. diameter × 12-in. height cylindrical test specimen, manufactured by Interlaken, with internally mounted transducers (location of load cell and LVDTs). Minnesota uses the Universal Model ($M_r = K1p_a\ (\theta/p_a)^{K_2}\ ((\tau_{oct}/p_a) + 1)^{K_3}$) to characterize stiffness as interpreted from the NCHRP 1-28a data. MnDOT currently uses MnPAVE and RigidPave for flexible and rigid pavement designs, respectively. They are developing a pavement design catalog for rigid pavements using MEPDG (through research).

Abbreviations used without definitions in TRB publications:

A4A	Airlines for America
AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation