TRANSPORTATION TECHNOLOGY SUPPORT
FOR DEVELOPING COUNTRIES

COMPENDIUM 10

Compaction of Roadway Soils

Compactación de suelos viales

Le compactage des sols routiers

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Notice

The project that is the subject of this report was approved by the Governing Board of the National Research Council, whose members are drawn from the councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the committee responsible for the report were chosen for their special competence and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

Cover photo: Steel-wheeled vibratory roller is used in construction work in Sennar, Sudan (courtesy of TRRL, United Kingdom).

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The development of agriculture, the distribution of food, the provision of health services, and the access to information through educational services and other forms of communication in rural regions of developing countries all heavily depend on transport facilities. Although rail and water facilities may play important roles in certain areas, a dominant and universal need is for road systems that provide an assured and yet relatively inexpensive means for the movement of people and goods. The bulk of this need is for low-volume roads that generally carry only 5 to 10 vehicles a day and that seldom carry as many as 400 vehicles a day.

The planning, design, construction, and maintenance of low-volume roads for rural regions of developing countries can be greatly enhanced with respect to economics, quality, and performance by the use of low-volume road technology that is available in many parts of the world. Much of this technology has been produced during the developmental phases of what are now the more developed countries, and some is continually produced in both the less and the more developed countries. Some of the technology has been documented in papers, articles, and reports that have been written by experts in the field. But much of the technology is

Descripción del proyecto

En las regiones rurales de países en desarrollo, el desarrollo de la agricultura, la distribución de víveres, la provisión de servicios de sanidad, y el acceso a información por medio de servicios educacionales y otras formas de comunicación, dependen en gran parte de los medios de transporte. Aunque en ciertas áreas los medios de ferrocarril y agua desempeñan un papel importante, existe una necesidad universal y dominante de crear sistemas viales que provean un medio asegurado pero relativamente poco costoso para el movimiento de gente y mercancías. La mayor parte de esta necesidad se solucionaría con la construcción de caminos de bajo volúmen que generalmente moverían únicamente de 5 a 10 vehículos por día y que pocas veces moverían tanto como 400 vehículos por día.

El planeamiento, diseño, construcción y mantenimiento de caminos de bajo volúmen para regiones rurales de países en desarrollo pueden ser mejorados, con respecto al costo, calidad, y rendimiento, por el uso de la tecnología de caminos de bajo volúmen que se encuentra disponible en muchas partes del mundo. Mucha de esta tecnología ha sido producida durante las épocas de desarrollo de lo que ahora son los países más desarrollados, y alguna se produce contínuamente en estos países así como en los países menos desarrollados. Parte de la tecnología se ha documentado en disertaciones, artículos, e informes que han sido escritos por expertos en el campo. Pero mucha de la tecnología no está documentada y existe principalmente en la memoria de aquellos que han desa-

Description du projet

Dans les régions rurales des pays en voie de développement, l'exploitation agricole, la distribution des produits alimentaires, l'accès aux services médicaux, l'accès aux matériaux et aux marchandises, à l'information et aux autres services, dépendent en grande partie des moyens de transport. Bien que les transports par voie ferrée et par voie navigable jouent un rôle important dans certaines régions, un besoin dominant et universel éxiste d'un réseau routier qui puisse assurer avec certitude et d'une façon relativement bon marché, le déplacement des habitants, et le transport des marchandises. La plus grande partie de ce besoin peut être satisfaite par la construction de routes à faible capacité, capables d'accommoder un trafic de 5 a 10 véhicules par jour, ou plus rarement, jusqu'à 400 véhicules par jour.

L'utilisation des connaissances actuelles en technologie, qui sont accéssibles dans beauundocumented and exists mainly in the minds of those who have developed and applied the technology through necessity. In either case, existing knowledge about low-volume road technology is widely dispersed geographically, is quite varied in the language and the form of its existence, and is not readily available for application to the needs of developing countries.

In October 1977 the Transportation Research Board (TRB) began this 3-year special project under the sponsorship of the U.S. Agency for International Development (AID) to enhance rural transportation in developing countries by providing improved access to existing information on the planning, design, construction, and maintenance of low-volume roads. With advice and guidance from a project steering committee, TRB defines, produces, and transmits information products through a network of correspondents in developing countries. Broad goals for the ultimate impact of the project work are to promote effective use of existing information in the economic development of transportation infrastructure and thereby to enhance other aspects of rural development throughout the world.

In addition to the packaging and distribution of technical information, personal interactions with users are provided through field visits, con-

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rrollado y aplicado la tecnología por necesidad. En cualquier caso, los conocimientos en existencia sobre la tecnología de caminos de bajo volúmen están grandemente esparcidos geográficamente, varian bastante con respecto al idioma y su forma, y no se encuentran fácilmente disponibles para su aplicación a las necesidades de los países en desarrollo.

En octubre de 1977 el Transportation Research Board (TRB) comenzó este proyecto especial de tres años de duración bajo el patrocinio de la U.S. Agency for International Development (AID) para mejorar el transporte rural en los países en desarrollo acrecentando la disponibilidad de la información en existencia sobre el planeamiento, diseño, construcción, y mantenimiento de caminos de bajo volúmen. Con el consejo y dirección de un comité de iniciativas para el proyecto, el TRB define, produce, y transmite productos informativos a través de una red de corresponsales en países en desarrollo. Las metas generales para el impacto final del trabajo del proyecto son la promoción del uso efectivo de la información en existencia en el desarrollo económico de la infraestructura de transporte y de esta forma mejorar otros aspectos del desarrollo rural a través del mundo.

Además de la recolección y distribución de la

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coup de pays, peut faciliter l'étude des projets de construction, tracé et entretien, de routes à faible capacité dans les régions rurales des pays en voie de développement, surtout en ce qui concerne l'économie, la qualité, et la performance de ces routes. La majeure partie de cette technologie a été produite durant la phase de développement des pays que l'on appelle maintenant développés, et elle continue à être produite à la fois dans ces pays et dans les pays en voie de développement. Certains aspects de cette technologie ont été documentés dans des articles ou rapports écrits par des experts. Mais une grande partie des connaissances n'existe que dans l'esprit de ceux qui ont eu besoin de développer et appliquer cette technologie. De plus, dans ces deux cas, les écrits et connaissances sur la technologie des routes à faible capacité, sont dispersés géographiquement, sont écrits dans des langues différentes, et ne sont pas assez aisément accessibles pour être appliqués aux besoins des pays en voie de développement.

En octobre 1977, le Transportation Research Board (TRB) initia ce projet, d'une duré de 3 ans, sous le patronage de l'U.S. Agency for International Development (AID), pour améliorer le transport rural dans les pays en voie de développement, en rendant plus accessible la documentation existante sur la conception, le tracé, la construction, et l'entretien des routes à faible capacité. Avec le conseil, et sous la conduite d'un comité de direction, TRB définit, produit, et transmet cette documentation à l'aide d'un réseau de correspondants dans les pays en voie de développement. Nous espérons que le résultat final de ce projet sera de favoriser l'utilisation de cette documentation, pour aider au développement économique de l'infrastructure des transports, et de cette façon mettre en valeur d'autres aspects d'exploitation rurale à travers le monde.

ferences in the United States and abroad, and other forms of communication.

Steering Committee

The Steering Committee is composed of experts who have knowledge of the physical and social characteristics of developing countries, knowledge of the needs of developing countries for transportation, knowledge of existing transportation technology, and experience in its use.

Major functions of the Steering Committee are to assist in the definition of users and their needs, the definition of information products that match user needs, and the identification of informational and human resources for development of the information products. Through its membership the committee provides liaison with project-related activities and provides guidance for interactions with users. In general the Steering Committee gives overview advice and direction for all aspects of the project work.

The project staff has responsibility for the preparation and transmittal of information products, the development of a correspondence network throughout the user community, and interactions with users.

Information Products

Three types of information products are prepared: compendiums of documented information on relatively narrow topics, syntheses of knowledge and practice on somewhat broader

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información técnica, se provee acciones recíprocas personales con los usuarios por medio de visitas de campo, conferencias en los Estados Unidos de Norte América y en el extranjero, y otras formas de comunicación.

Comité de iniciativas

El comité de iniciativas se compone de expertos que tienen conocimiento de las características físicas y sociales de los países en desarrollo, conocimiento de las necesidades de transporte de los países en desarrollo, conocimiento de la tecnología de transporte en existencia, y experiencia en su uso.

Las funciones importantes del comité de iniciativas son las de ayudar en la definición de usuarios y sus necesidades, de productos informativos que se asemejan a las necesidades del usuario, y la identificación de recursos de conocimientos y humanos para el desarrollo de los productos informativos. A través de sus miembros el comité provee vínculos con actividades relacionadas con el proyecto y también una guía para la interacción con los usuarios. En general el comité de iniciativas proporciona consejos y dirección general para todos los aspectos del trabajo de proyecto.

El personal de proyecto es responsable de la preparación y transmisión de los productos informativos, el desarrollo de una red de corresponsales a través de la comunidad de usuarios, y la interacción con los usuarios.

Productos informativos

Se preparan tres tipos de productos informativos: los compendios de la información documentada sobre temas relativamente limitados, la síntesis del conocimiento y práctica sobre temas

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En plus de la dissémination de cette documentation technique, des visites, des conférences aux Etats Unis et à l'étranger, et d'autres formes de communication permettront une interaction constante avec les usagers.

Comité de direction

Le comité de direction est composé d'experts qui ont à la fois des connaissances sur les caractéristiques physiques et sociales des pays en voie de développement, sur leurs besoins au point de vue transports, sur la technologie actuelle des transports, et ont aussi de l'expérience quant à l'utilisation pratique de cette technologie.

Les fonctions majeures de ce comité sont d'abord d'aider à définir les usagers et leurs besoins, puis de définir leurs besoins en matière de documentation, et d'identifier les ressources documentaires et humaines nécessaires pour le développement de cette documentation. Par l'intermédiaire des ses membres, le comité pourvoit à la liaison entre les différentes fonctions relatives au projet, et dirige l'interaction avec les usagers. En général, le comité de direction conseille et dirige toutes les phases du projet.

Notre personnel est responsable de la préparation et de la dissémination des documents, du développement d'un réseau de correspondants pris dans la communauté d'usagers, et de l'interaction avec les usagers.

La documentation

Trois genres de documents sont preparés: des recueils dont le sujet est relativement limité, des subjects, and proceedings of low-volume road conferences that are totally or partially supported by the project. Compendiums are prepared by project staff at the rate of about 6 per year; consultants are employed to prepare syntheses at the rate of 2 per year. At least one conference proceedings will be published during the 3-year period. In summary, this project aims to produce and distribute between 20 and 30 publications that cover much of what is known about low-volume road technology.

Interactions With Users

A number of mechanisms are used to provide interactions between the project and the user community. Project news is published in each issue of *Transportation Research News.* Feedback forms are transmitted with the information products so that recipients have an opportunity to say how the products are beneficial and how they may be improved. Through semiannual visits to developing countries, the project staff acquires first-hand suggestions for the project work and can assist directly in specific technical problems. Additional opportunities for interaction with users arise through international and in-country conferences in which there is project participation. Finally, annual colloquiums are held for students from developing countries who are enrolled at U.S. universities.

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un poco más amplios, y los expedientes de conferencias de caminos de bajo volúmen que están totalmente o parcialmente amparados por el proyecto. El personal de proyecto prepara los compendios a razón de unos 6 por año; se utilizan consultores para preparar las síntesis a razón de 2 por año. Se publicará por lo menos un expediente de conferencia durante el período de tres años. En breve, este proyecto pretende producir y distribuir entre 20 y 30 publicaciones que cubren mucho de lo que se conoce de la tecnología de caminos de bajo volúmen.

Interacción con los usuarios

Se utilizan varios mecanismos para proveer las interacciones entre el proyecto y la comunidad de usuarios. Se publican las noticias del proyecto en cada edición de la *Transportation Research News.* Se transmiten, con los productos informativos, formularios de retroacción para que los recipientes tengan oportunidad de decir cómo benefician los productos y cómo pueden ser mejorados. A través de visitas semianuales a los países en desarrollo, el personal del proyecto adquiere directamente de fuentes originales sugerencias para el trabajo del proyecto y puede asistir directamente en problemas técnicos específicos. Surgen oportunidades adicionales para la interacción con los usuarios a través de conferencias internacionales y nacionales en donde participa el proyecto. Finalmente, se organizan diálogos con estudiantes de países en desarrollo que están inscriptos en universidades norteamericanas.

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synthèses de connaissances et de pratique sur des sujets beaucoup plus généraux, et finalement des comptes-rendus de conférences sur les routes à faible capacité, qui seront organisées complètement ou en partie par notre projet. Environ 6 recueils par an sont preparés par notre personnel. Deux synthèses par an sont écrites par des experts pris à l'extérieur. Les comptes-rendus d'au moins une conférence seront écrits dans une période de 3 ans. En résumé, l'objet de ce projet est de produire et disséminer entre 20 et 30 documents qui couvriront l'essentiel des connaissances sur la technologie des routes à faible capacité.

Interaction avec les usagers

Un certain nombre de mécanismes sont utilisés pour assurer l'interaction entre le personnel du projet et la communauté d'usagers. Un bulletin d'information est publié dans chaque numéro de *Transportation Research News.* Des formulaires sont joints aux documents, afin que les usagers aient l'opportunité de juger de la valeur de ces documents et de donner leur avis sur les moyens de les améliorer. Au cours de visites semi-annuelles dans les pays en voie de développement notre personnel obtient de première main des suggestions sur le bon fonctionnement du projet et peut aider à résoudre sur place certains problèmes techniques spécifiques. En outre, des conférences tenues soit aux Etats Unis, soit à l'étranger, sont l'occasion d'un échange d'idées entre notre personnel et les usagers. Finalement, des colloques annuels sont organisés pour les étudiants des pays en voie de développement qui étudient dans les universités américaines.

Foreword and Acknowledgments

This compendium is the twelfth product of the Transportation Research Board's project on Transportation Technology Support for Developing Countries under the sponsorship of the U.S. Agency for International Development. The objective of this book is that it provide useful and practical information for those in developing countries who have direct responsibility for compaction of roadway soils. Feedback from correspondents in developing countries will be solicited and used to assess the degree to which this objective has been attained and to influence the nature of later products.

Acknowledgment is made to the following publishers for their kind permission to reprint the selected text portions of this compendium: American Association of State Highway and Transportation Officials, Washington, D.C.; Australian Road Research Board, Victoria; Idaho Transportation Department, Boise; McGraw-Hill Publishing Co., Inc., New York, N.Y.; Ontario Ministry of Transportation and Communications, Canada; Permanent International Association of Road Congresses, Paris; Roads and Transportation Association of Canada, Ottawa; The Asphalt Institute, College Park, Md.; State of Vermont Agency of Transportation; and U.S. Agency for International Development, Washington, D.C.

Prefacio y agradecimientos

Este compendio es el duodécimo producto del proyecto del Transportation Research Board sobre Apoyo de Tecnología de Transporte para Países en Desarrollo bajo el patrocinio de la U.S. Agency for International Development. El objetivo de este libro es el de proveer información útil y práctica para aquellos en países en desarrollo quienes tienen responsabilidad directa para la compactación de suelos viales. Se pedirá a los corresponsales en los países en desarrollo información sobre los resultados, para utilizarse en el asesoramiento del grado al cual se ha obtenido ese objectivo y para influenciar la naturaleza de productos subsequentes.

Se reconoce a los siguientes editores por el permiso dado para reimprimir las porciones de texto seleccionadas para este compendio. American Association of State Highway and Transportation Officials, Washington, D.C.; Australian Road Research Board, Victoria; Idaho Transportation Department, Boise; McGraw-Hill Publishing Co., Inc., New York, N.Y.; Ontario Ministry of Transportation and Communications, Canada; Permanent International Association of Road Congresses, Paris; Roads and Transportation Association of Canada, Ottawa; The Asphalt Institute, College Park, Md.; State of Vermont Agency of Transportation; y U.S. Agency for International Development, Washington, D.C.

Avant-propos et remerciements

Ce recueil représente le douxième volume du projet du Transportation Research Board sur la Technologie des transports à l'usage des pays en voie de développement. Ce projet est placé sous le patronage de l'U.S. Agency for International Development. L'objet de ce recueil est de réunir une documentation pratique et utile qui puisse aider les personnes responsables du compactage des sols. La réaction des correspondants des pays en voie de développement sera sollicitée et utilisée pour évaluer à quel point le but proposé de ce projet a été atteint, et pour influencer la nature des ouvrages à venir.

Nous remercions les éditeurs qui ont gracieusement donné leur permission de reproduire les textes sélectionnés pour ce recueil: American Association of State Highway and Transportation Officials, Washington, D.C.; Australian Road Research Board, Victoria; Idaho Transportation Department, Boise; McGraw-Hill Publishing Co., Inc., New York, N.Y.; Ontario Ministry of Transportation and Communications, Canada; Permanent International Association of Road Congresses, Paris; Roads and Transportation Association of Canada, Ottawa; The Asphalt Institute, College Park, Md.; State of Vermont Agency of Transportation; et U.S. Agency for International Development, Washington, D.C.

Appreciation is also expressed to libraries and information services that provided references and documents from which final selections were made for the selected texts and bibliography of this compendium. Special acknowledgment is made to the U.S. Department of Transportation Library Services Division and to the Library and Information Service of the U.K. Transport and Road Research Laboratory (TRRL). Any photographs provided by TRRL have been reproduced by permission of Her Majesty's Stationery Office.

Finally, the Transportation Research Board acknowledges the valuable advice and direction that have been provided by the project Steering Committee and is especially grateful to W. R. Hudson, University of Texas at Austin, Adrian Pelzner, U.S. Forest Service, and John P. Zedalis, USAID, who provided special assistance on this particular compendium.

También se reconoce a las bibliotecas y servicios de información que proveen las referencias y documentos de los cuales se hacen las selecciones finales para los textos seleccionados y la bibliografía en este compendio. Se hace un especial reconocimiento a la Library Services Division del U.S. Department of Transportation y el Library and Information Service del U.K. Transport and Road Research Laboratory (TRRL). Las fotografías proveídas por TRRL fueron reproducidas con la autorización de Her Majesty's Stationery Office.

Finalmente, el Transportation Research Board agradece el consejo y dirección valiosos provistos por el comité de iniciativas, con especial reconocimiento a los señores W. R. Hudson, University of Texas at Austin, Adrian Pelzner, U.S. Forest Service, y John P. Zedalis, USAID, que prestaron ayuda especial para este compendio en particular.

Nos remerciements aussi aux bibliothèques et bureaux de documentation qui nous ont fourni les documents et les références utilisés dans les textes choisis et bibliographie de ce recueil. Nous remercions spécialement la U.S. Department of Transportation Library Services Division et les Library and Information Service of the U.K. Transport and Road Research Laboratory (TRRL). Les photos fournies par le TRRL ont été reproduites avec la permission de Her Majesty's Stationery Office.

Finalement, le Transportation Research Board reconnait la grande valeur de la direction et de l'assistance des membres du comité de direction et les remercie de leur concours et de la façon dont ils dirigent le projet, spécialement Messieurs W. R. Hudson, University of Texas at Austin, Adrian Pelzner, U.S. Forest Service, et John P. Zedalis, USAID, qui ont bien voulu prêter leur assistance à la préparation de ce recueil.

Overview

Background and Scope

Compaction is a major process in the successful construction of all earthworks. Compaction is the densification of material by means of mechanical manipulation, or mechanical stabilization, as described in *Compendium 7: Road Gravels.* Compaction is equally important in the soils below the surface and base-course materials that were the subject of that compendium. Compendium 10 discusses the concepts of compaction, the tests used to determine the amount of compaction required and then achieved, and the types of equipment used in the process.

The soil in fills must be sufficiently dense and strong to resist consolidation under its own weight or sliding along its slopes. The subgrade must not change volume excessively during wet periods or from frost action. The soil of the compacted subgrade and base courses must resist densification and deformation under repeated wheel loads.

Compaction is the result of mechanical effort expended on a soil. The amount of compaction achieved per unit of effort will vary depending on (a) the type of soil, (b) the moisture content of the soil, and (c) the method of applying the mechanical effort. The degree of compaction required will depend on the proposed use of the soil mass being compacted. In each case the

Vista General

Antecedentes y alcance

La calidad de compactación es uno de los factores más importantes en la buena construcción de todo terraplenado. La compactación se define como la densificación de material por medio de manipulación mecánica, o estabilización mecánica, como se describe en el *Compendio 7: Gravas.* La compactación tiene igual importancia en los suelos debajo de las capas de base y superficie que fueron el tema de aquel compendio. El Compendio 10 estudia los conceptos de compactación, los ensayos utilizados para determinar la cantidad de compactación necesitada y luego lograda, y el equipo utilizado en el proceso.

El suelo de los rellenos deberá ser lo suficientemente denso y fuerte para resistir la consolidación bajo su propio peso o deslizamientos a lo largo de sus pendientes. El volúmen de la subrasante no deberá cambiar excesivamente durante los períodos de lluvia o de helada. El suelo de las capas compactadas de base y de subrasante deberá resistir la densificación y deformación bajo las repetidas cargas por rueda.

La compactación es el resultado de un esfuerzo mecánico realizado sobre un suelo. La cantidad de compactación que se logre por unidad de esfuerzo varía de acuerdo con (a) el tipo de suelo, (b) el contenido de humedad del

Exposé

Historique et objectif

Le compactage est une opération majeure dans la réussite de travaux de terrassements. On entend par compactage la densification des matériaux par le moyen d'une manipulation mécanique ou d'un traitement chimique ainsi que nous l'avons expliqué dans le *recueil no. 7: Les graviers.* Le compactage est aussi important pour les matériaux de la couche de base que pour ceux situés en dessous, l'étude de ces matériaux formant le sujet du recueil no. 7. Dans le recueil no. 10, nous allons présenter les théories du compactage, les essais utilisés pour déterminer le degré de compaction nécessaire et ensuite obtenu, et le genre de matériel utilisé pour en arriver à ces fins.

Le sol des déblais doit être suffisamment dense et solide pour résister au tassement sous son propre poids, ou au glissement le long des pentes. La couche de forme ne doit pas changer de volume d'une façon excessive durant les consistency of the results of the compactive effort is of particular importance.

The purpose of a road is to provide a safe usable passageway at an acceptable level of service. Because the use of a low-volume road differs from that of an arterial highway, the materials incorporated into a low-volume road can differ from the materials required to construct a trunk road. However, the compaction applied must aid the soil in meeting the requirement of resisting further densification or deformation under the number and weight of the expected wheel loads. Otherwise, the performance of the road under traffic may require excessive maintenance.

Specifications detail the work to be done, the materials to be used, and the quality that must be achieved. Compaction is therefore the result of enforcement of the specifications. Although the development of materials specifications, including compaction requirements, is beyond the scope of this compendium, it is obvious that the specifications determine the construction cost of the road. The quality of the materials and workmanship required by the specifications can also influence, to a great degree, the future maintenance costs of the road. The agency responsible for the design and specifications for low-volume roads should ensure that the specifications define the material and workmanship re-

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suelo, y (c) el método de aplicar el esfuerzo mecánico. El grado de compactación que se requiere depende del uso al cual se pondrá el suelo compactado. En cada caso individual es de particular importancia la consistencia en los resultados de este esfuerzo compactivo.

El propósito de un camino es de ofrecer una vía de paso segura y utilizable a un nivel aceptable de servicio. Ya que el uso de un camino de bajo volúmen difiere de la de una carretera arterial, los materiales incorporados en un camino de bajo volúmen pueden diferir de los materiales necesarios para construir un camino principal. Sin embargo, el esfuerzo compactivo que se aplique deberá ayudar el suelo en lograr la resistencia necesaria contra más densificación o deformación bajo el número y peso de cargas por rueda anticipadas. De otro modo el rendimiento del camino bajo el tráfico puede exigir excesiva conservación.

Las especificaciones describen en detalle el trabajo que se debe realizar, los materiales a utilizarse, y la calidad que se deberá lograr. Por lo tanto, la compactación es el resultado de la observación de las condiciones de las especificaciones. El desarrollo de las especificaciones para materiales, incluyendo los requisitos para la compactación, está fuera del alcance de este compendio, pero está claro que las especificaciones determinan el costo de construcción del camino. La calidad de los materiales y la mano de obra exigidos en las especificaciones también pueden influir, en gran parte, los costos futuros de conservación. La agencia encargada del diseño y las especificaciones para caminos de bajo volúmen deberá asegurarse de que las especificaciones definan el material y mano de obra necesarios para lograr el nivel de servicio requerido. Las especificaciones que definen normas de-

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périodes de pluie ou de gel. Le sol compacté des couches de base et de forme doit pouvoir résister à la densification et aux déformations dûes à l'influence répétée des charges roulantes.

Le compactage est le résultat d'une action mécanique exercée sur un sol. Le compactage obtenu par unité d'effort est variable selon: (a) la nature du sol, (b) sa teneur en eau, et (c) la méthode utilisée pour exercer cet effort mécanique. Le degré de compactage nécessaire dépend lui même de l'emploi final du sol à compacter. Dans chaque cas, l'uniformité relative des résultats de l'effort compactif est d'une importance particulière.

La raison d'être d'une route est de fournir un passage utilisable et sûr, à un un niveau de service acceptable. Puisque les modalités d'emploi d'une route à faible trafic sont différentes de celles d'une route à grande circulation, les matériaux employés pour sa construction peuvent aussi ne pas être les mêmes que ceux qui sont obligatoires dans la construction d'une artère principale. Toutefois, le compactage doit aider le sol à résister davantage à la densification et aux déformations causées par le nombre et le poids des charges roulantes. Autrement, la tenue de la route soumise à la circulation peut exiger un entretien excessif.

Les spécification du cahier des charges expliquent en détail le travail à faire, les matériaux à utiliser et la qualité qui doit être obtenue. Le compactage est donc le résultat de l'application de ces spécifications. Bien que l'étude du développement de spécifications pour les matériaux et de normes de compactage dépasse l'envergure de ce recueil, nous pouvons cependant supputer que ces spécifications vont déterminer le coût de la construction de la route. La qualité des matériaux et de l'exécution des travaux que demandent les spécifications, peut aussi inquired to achieve the necessary level of service. Specifications defining unnecessarily high standards will result in a larger capital outlay with no corresponding benefits.

Rationale for This Compendium

Two distinct methods of compaction control—conventional and statistical—are currently in use. The conventional method involves the selection of representative samples. The engineer or inspector selects samples that are representative of the lift being inspected. These samples are tested and the acceptance or rejection of the compacted area is based on the test results. The statistical method involves the random selection of several samples. The test results of these samples are statistically evaluated to determine whether the compaction represented by the samples is within a desired range. Acceptance is based on that evaluation. Most specifications in use in developing countries were written under the assumption that representative sampling will be used. In the United States, however, many governmental agencies are currently exploring the use of statistical sampling. (Three texts on this subject are included in the list of additional references for this compendium.) Both methods have strong advocates; however, space limitations preclude documentation of the desirability or drawbacks of either method. The tests described in Com-

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masiado altas darán como resultado un mayor gasto sin los beneficios correspondientes.

Exposición razonada para este compendio

Hay dos métodos diferentes de control de compactación utilizados actualmente—el método tradicional y el estadístico. El método tradicional implica la selección de muestras representativas. El ingeniero o inspector selecciona muestras que son representativas de la capa que se está inspeccionando. Se ensayan estas muestras, y los resultados del ensayo determinan la aceptación o rechazo del área compactada. El método estadístico implica la selección al azar de varias muestras. Los resultados del ensayo de estas muestras se evalúan estadísticamente para determinar si la compactación representada por las muestras se encuentra dentro de los límites deseados. Su aceptación se basa en estas evaluaciones. Casi todas las especificaciones actualmente utilizadas en países en desarrollo se crearon con la suposición de que se utilizarían muestras representativas. Sin embargo, en los Estados Unidos de América muchas de las agencias del gobierno están estudiando el método de muestreo estadístico. (Tres textos sobre este tema se incluyen en las referencias adicionales de la bibliografía de este compendio.) Los dos métodos tienen partidarios; sin embargo las limitaciones de espacio en el compendio impiden la documentación de los pro y contra de cada método. Los ensayos que se describen en el Compendio 10 son ensayos básicos y son adecuados para utilizar con los dos tipos de control de compactación.

Los ensayos necesarios para lograr la calidad adecuada de compactación comienzan durante la búsqueda pre-constructiva de fuentes de

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fluencer largement le prix de revient de l'entretien de cette route. Les responsables du calcul et des spécifications de routes économiques devraient toujours s'assurer que les spécifications sont précisement celles qui sont nécessaires pour le niveau de service désiré. Des normes qui demandent inutilement un standard trop élevé, auront pour résultat une mise de fonds initiale plus élevée sans bénéfices correspondants.

Objectif de ce recueil

Deux méthodes différentes de contrôle du compactage sont actuellement utilisées: la méthode classique et la méthode statistique. La méthode classique est celle qui utilise la sélection d'échantillons représentatifs. L'ingénieur ou l'inspecteur choisit des échantillons, ou prélèvements, qui sont représentatifs de la couche à inspecter. On fait des essais sur ces prélèvements, et, suivant les résultats, on accepte ou on rejette le compactage de cette couche. Pour la méthode statistique, on prélève des échantillons au hasard, et on les soumet à des essais. On fait ensuite une analyse statistique des résultats pour déterminer si le compactage de ces échantillons est compris entre les limites désirées. La réception du compactage dépend de cette évaluation. La plupart des normes en vigueur dans les pays en voie de développement ont été developpées en supposant que la méthode des échantillons représentatifs serait utilisée. Par contre, aux Etats Unis, plusieurs organismes du pendium 10 are basic tests and are suitable for use with either type of compaction control.

The testing necessary to achieve adequate compaction begins during the preconstruction search for materials sources, i.e., the investigation of in situ materials, borrow areas, and gravel pits. ***Compendium 2: Drainage and Geological Considerations in Highway Location, Compendium 6: Investigation and Development of Materials Resources,*** and ***Compendium 7: Road Gravels*** contain selected texts that describe the types of laboratory and/or simplified field testing carried out during the preconstruction phase. These tests aid in the development of proper compaction requirements during the construction phase. Compendium 10 describes many of the actual laboratory testing procedures referenced in earlier compendiums.

Normally, the laboratory testing phase continues through the actual construction of a roadway as the construction personnel submit samples of the in-place materials for acceptance tests. During construction, additional on-site testing takes place to determine immediate acceptance or rejection of the compactive effort. However, in low-volume road construction both

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materiales, es decir, la investigación de los materiales in situ, áreas de préstamo y canteras de grava. El ***Compendio 2: Consideraciones de drenaje y geológicas en la ubicación de carreteras,*** el ***Compendio 6: Investigación y desarrollo de recursos de materiales,*** y el ***Compendio 7: Gravas*** contienen textos seleccionados que describen los tipos de ensayos de laboratorio y/o de campo simplificados que se realizan durante la etapa de pre-construcción. Estos ensayos ayudan en el desarrollo de correctos requerimientos de compactación durante la etapa de construcción. El Compendio 10 describe muchos de los verdaderos procedimientos de ensayo en el laboratorio que fueron nombrados en compendios previos.

Generalmente la etapa de ensayo en el laboratorio continúa durante la construcción del camino. El personal de la obra envía muestras del material in situ para ensayos de aceptación. Durante la construcción se realizan ensayos adicionales en la obra para la inmediata aceptación o rechazo del esfuerzo compactivo. Sin embargo, en la construcción de caminos de bajo volúmen las actividades de ensayo en el laboratorio y en el campo pueden ser mínimas, o quizás no se realice ningún ensayo en el laboratorio a favor de un control completo en el campo. Los procedimientos para ensayos individuales son iguales antes de y durante la construcción, con control de compactación representativo o estadístico.

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gouvernement sont en train d'étudier l'utilisation de la méthode d'échantillonage statistique (Trois textes sur ce sujet sont inclus dans la liste de références supplémentaires de ce recueil). Les deux méthodes ont leurs défenseurs, malheureusement, nous n'avons pas la place de documenter le pour et le contre de ces deux méthodes. Les essais que nous décrivons dans ce recueil sont des essais de base, qui peuvent être utilisés pour le contrôle du compactage, selon l'une ou l'autre méthode.

Les essais nécessaires à la réalisation d'un compactage correct commencent avant le début de la construction, lors de la prospection des matériaux routiers, c'est à dire l'inventaire in situ des lieux d'emprunt et des gravières. Nos ***recueils no. 2: Drainage and Geological Considerations in Highway Location, no. 6: Investigation and Development of Material Resources,*** et ***no. 7: Road Gravels,*** contiennent des textes qui décrivent les genres d'essais en laboratoire et/ou en chantier qui doivent être faits durant ce stade de pré-construction. Ces essais aident à établir les valeurs de compactage qui seront nécessaires au stade de la construction. Le recueil no. 10 va décrire plusieurs essais en laboratoire que nous avions mentionnés dans des recueils précédents.

Normalement, on fait des essais en laboratoire tout le temps de la construction de la route, car le personnel de construction doit soumettre les échantillons de matériaux pour la réception. En outre, durant la construction, on fait des essais sur le chantier afin de déterminer immédiatement la réception ou le rejet du compactage. Toutefois, quand on construit une route economique, il se peut que les essais en laboratoire et sur le chantier soient réduits au minimum, ou même que les essais en laboratoire soient négligés entièrement en faveur de ceux sur le chantier. Le processus d'expérimentation est le même, que les essais soient faits avant ou pendant la construction, et que le contrôle du compactage soit fait d'après la méthode représentative ou statistique.

Les dimensions des échantillons pour beaulaboratory and field testing activities may in fact be minimal, or the laboratory testing may be neglected entirely in favor of complete field control. The procedures for individual tests remain the same whether the tests are made before or during construction and whether the compaction control is representative or statistical.

The sizes of the samples for many of the individual tests are described in this compendium and in ***Design Manual; Soil Mechanics, Foundations and Earth Structures*** (Text 5, Compendium 2). The quantity of samples to be taken during construction, or the control testing frequency, is not heavily documented. Occasionally rules of thumb, i.e., one density and moisture content test per layer for 500 cubic yards of in-place subbase or base material, appear. They are quickly modified, however, by the statement that testing frequency may vary for individual projects in accordance with (a) project size and job conditions such as uniformity of materials at the source, (b) the methods and equipment used, and (c) weather conditions. Most agencies that have tried to predetermine generalized control testing frequency have found that it is impractical for a large portion of their projects; thus, they do not formalize their general guidelines.

In addition to formal testing, adequate observation and inspection of the actual construction operations and processes must be carried out to be sure the achievement of compaction quality can be obtained during construction with an acceptable degree of consistency. Compendium 10 therefore includes selected texts that describe the criteria for this type of inspection and the types of compaction equipment that are suitable for the densification of various types of soil.

Los tamaños para las muestras de muchos de los ensayos individuales se describen en este compendio y en ***Design Manual; Soil Mechanics, Foundations and Earth Structures*** (Manual de diseño; Mecánicas del suelo, fundamentos, y estructuras de tierra, Texto 5, Compendio 2). La cantidad de muestras a tomarse durante la construcción, es decir las veces que se deberán hacer ensayos de control, no está excesivamente documentada. Aparecen ocasionalmente reglas empíricas, es decir, un ensayo de densidad y contenido de humedad por cada capa de 500 yardas cúbicas de material de subbase o base en obra. Sin embargo son rápidamente modificadas por la observación de que la frecuencia de los ensayos puede variar para cada proyecto individual de acuerdo con (a) el tamaño del proyecto y las condiciones del trabajo, tales como uniformidad de materiales en la fuente, (b) los métodos y equipo que se utilizan, y (c) las condiciones meteorológicas. La mayoría de las agencias que han intentado predeterminar cuántas veces en general se deben realizar ensayos de control han descubierto que no es práctico para gran parte de sus proyectos, y por lo tanto no formalizan sus pautas generales.

Además de ensayos formales, se deben realizar observaciones e inspecciones adecuadas de las verdaderas operaciones y procesos de construcción para asegurarse de que se podrá lograr la calidad de compactación durante la construcción con un grado aceptable de consistencia. Por lo tanto, el Compendio 10

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coup de ces essais sont données dans ce recueil et dans le texte no. 5, ***Design Manual; Soil Mechanics Foundations and Earth Structures,*** du recueil no. 2. Il n'existe pas beaucoup de documents sur le nombre des échantillons à soumettre aux essais durant la construction, c'est à dire la fréquence des essais de contrôle de qualité. Quelquefois, on décide empiriquement que l'on doit faire un essai de densité et de teneur en eau par couche, pour 500 yards cubes de matériaux de base ou de fondation mis en place. Cette régle empirique est rapidement modifiée cependant, si nous observons que la fréquence des essais peut varier d'un projet à un autre selon: (a) la grandeur du projet et les conditions de travail telles que l'uniformité des matériaux du gîte, (b) les méthodes et le matériel utilisés, et (c) les conditions atmosphériques. Le plus souvent, beaucoup d'entreprises ou organismes qui ont essayé de déterminer à l'avance la fréquence des essais se sont aperçus que ce n'était pas pratique pour une grande partie de leurs projets, et n'ont donc pas établi un standard officiel.

En plus des essais standards, il faut aussi observer et inspecter de façon adéquate durant toutes les phases de la construction de la route, afin d'être sûr que la qualité du compactage soit obtenue à un degré d'uniformité acceptable. Dans cette optique, ce recueil contient des textes choisis qui décrivent les critères pour ce genre d'inspection et les engins de compactage qui conviènnent à la densification de différents types de sols.

Un des gros problèmes de la réussite du compactage est la différence entre les résultats des essais faits sur des échantillons recueillis lors de la prospection, et ceux des essais faits

A major problem in achieving proper compaction is the difference between the test results of exploratory samples and the actual construction samples. Both samples are supposed to be representative of the materials used in the construction. In fact, the excavation, transport, dumping, spreading, and compaction of the material tend to alter the composition of the soil being tested. Each test is made using a small volume of soil; the results, assuming no testing errors, indicate only the composition and compaction of the actual soil tested. Test results will therefore differ for soil taken in any other area, no matter how close to the original site. Nonetheless, proper evaluation of test results is still the most useful tool available for the achievement of proper compaction.

Discussion of Selected Texts

The first text, ***Significance of Quality Control,*** is a paper that appeared in the ***Proceedings of the Golden Jubilee Convention*** (Canadian Good Roads Association, 1964). It describes (a) the purpose of inspection and testing, (b) the general procedure for quality control, (c) the value of inspection and testing, (d) the nature of samples, (e) the value of test results, (f) the purpose of specifications, and (g) the effectiveness of job control.

The text points out the importance of quality control during the construction of a road and the need for well-trained personnel to conduct the control tests. It stresses the fact that most of the control tests used in the highway field are arbi-

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incluye textos seleccionados que describen los criterios para este tipo de inspección y los tipos de equipo de compactación que son adecuados para la densificación de varios tipos de suelos.

Uno de los problemas más importantes en la realización de la calidad de compactación es la diferencia entre los resultados de ensayo de muestras exploratorias y los resultados de ensayo de las verdaderas muestras de la obra. Se supone que ambas son representativas de los materiales utilizados en la construcción. La realidad es que la excavación, transporte, descarga, esparcido y compactación del material tienden a alterar la composición del suelo que se está ensayando. Cada prueba se realiza con una cantidad pequeña de suelo, y los resultados, suponiendo que no hay errores en el ensayo, únicamente indican la composición y compactación del suelo que se está ensayando. Los resultados siempre cambiarán para suelos de otras áreas por muy cerca que estén del área original. Sin embargo, la correcta evaluación de resultados de ensayo es el agente más útil disponible para la realización de la compactación apropiada.

Presentación de los textos seleccionados

El primer texto, ***Significance of Quality Control*** (La importancia del control de calidad, ***Proceedings of the Golden Jubilee Convention,*** Canadian Good Roads Association, 1964), describe (a) el objetivo de la inspección y ensayo, (b) el procedimiento general para el control de calidad, (c) la importancia de la inspección y ensayo, (d) las características de muestras, (e)

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sur les échantillons pris lors de la construction de la route. En théorie, ces deux sortes d'échantillons sont supposées représenter les matériaux utilisés en construction. En pratique, l'excavation, le transport, le déversement, le régalage et le compactage du matériau ont tendance à modifier la composition du sol soumis à l'essai. Chaque essai est fait en utilisant une petite quantité de matériau; les résultats, en supposant qu'aucune erreur ne soit commise, indiquent seulement la composition et le compactage du matériau soumis à l'expérimentation. Les résultats seront donc différents pour un matériau extrait d'un autre endroit, même s'il est extrait d'un endroit très proche de l'emplacement original. Malgré tout, une évaluation correcte des résultats est quand même l'outil le plus utile que l'on possède pour déterminer la réussite du compactage.

Discussion des textes choisis

Le premier texte, ***Significance of Quality Control*** (L'importance du contrôle de qualité) est une communication publiée dans le ***Proceedings of the Golden Jubilee Convention*** (Canadian Good Roads Association, 1964). On y décrit (a) le but de l'inspection et des essais, (b) le processus général de contrôle de qualité, (c) la valeur de l'inspection et des essais, (d) la nature des échantillons (e) la valeur des résultats des essais, (f) le but des spécifications, et (g) l'efficacité du contrôle du travail.

trary in the sense that (a) they do not represent actual field conditions at all times or in all locations, (b) they use different chemical or physical means to accelerate obtaining the results, or (c) they measure characteristics that permit the evaluation of the sample by comparison with other soils of similar nature rather than the evaluation of the actual physical properties of the soil under study.

The second text, ***Chapter 7 – Compaction,*** is excerpted from ***Highway Materials*** (Krebs/Walker, McGraw-Hill Book Company, 1971). It provides an analysis of the compactive effort on various types of soils and the tests that indicate the degree of compaction of these soils. Although the text was prepared for undergraduate civil engineering students, it draws material from many source references that would otherwise be included in this compendium. It is included as a general review of the art and techniques of densification of subgrades, embankments, subbases, bases, and gravel surfaces. It explains basic theories that must be understood in order for the reader to benefit from the more detailed texts included in Compendium 10.

The soils engineer may find many of the basic concepts familiar. However, the general highway engineer often loses sight of the complex interrelationships among soil properties, moisture content, compactive effort, and control tests that must be properly balanced in order to achieve a satisfactory end product at a reasonable cost.

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la importancia de los resultados de ensayo, (f) el objetivo de especificaciones, y (g) la eficacia del control del trabajo.

El texto indica la importancia del control de calidad durante la construcción de un camino y la necesidad de tener personal bien instruido para realizar los ensayos de control. Subraya que casi todos los ensayos de control que se utilizan en el campo vial son arbitrarios en el sentido de que (a) no representan las verdaderas condiciones del campo en todo momento en todas las ubicaciones, (b) utilizan distintos medios químicos o físicos para obtener resultados acelerados, o (c) miden las características que permiten la evaluación de la muestra en comparación con otros suelos similares, en vez de la evaluación de las propiedades físicas verdaderas del suelo que se está estudiando.

El segundo texto, ***Chapter 7, Compaction*** (Capítulo 7, Compactación, ***Highway Materials,*** Krebs/Walker, McGraw-Hill Book Company, 1971), proporciona un análysis del esfuerzo compactivo sobre varios tipos de suelos y los ensayos que indican el grado de compactación de estos suelos. Aunque el texto se preparó para estudiantes de ingeniería civil no graduados, extrae material de muchas referencias de origen que de otra manera hubieran sido incluidas en este compendio. Es incluido como un repaso general del arte y las técnicas de densificación de subrasantes, terraplenes, subbases, bases y superficies de grava. Explica las teorías básicas que el lector debe comprender para aprovechar totalmente los textos más específicos incluidos en el Compendio 10.

El ingeniero de suelos probablemente conocerá muchos de los conceptos básicos. Sin em-

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Le texte met l'emphase sur l'importance du contrôle de qualité pendant la construction de la route et la nécessité d'avoir un personnel qualifié pour diriger les essais. On souligne le fait que la plupart des essais de contrôle de qualité sont arbitraires dans le sens que (a) ils ne représentent pas les conditions sur le chantier à toutes les périodes ou à tous les emplacements, (b) ils utilisent des moyens mécaniques ou physiques pour accélerer les résultats, ou (c) ils mesurent les caractéristiques qui permettent d'évaluer le prélèvement par rapport à d'autres de même nature, plutôt que d'évaluer les propriétés physiques du sol que l'on est en train d'étudier.

Le deuxième texte, ***Chapter 7 — Compaction,*** (Chapitre 7 — Compaction) est extrait du livre ***Highway Materials*** (Krebs/Walker, McGraw-Hill Book Company, 1971). L'effort compactif sur différents sols et les essais qui indiquent leur degré de compacité sont donnés. Bien que ce livre soit un manual à l'usage des étudiants en génie civil, il contient beaucoup de références qui auraient été incluses dans ce recueil. Nous le considérons donc comme une revue générale de l'art et des techniques de densification des sous-sols, talus, sous-couches, couches de base et surfaces en matériaux graveleux. Ce texte contient et explique la théorie de base que le lecteur doit posséder afin d'être capable de mettre pleinement à profit les textes plus détaillés du recueil 10.

L'ingénieur des sols sera sans doute familiarisé avec beaucoup de ces concepts de base. Toutefois, l'ingénieur routier que nous qualifieront de généraliste, souvent perd de vue les rapports étroits et complexes qui existent entre les caractéristiques des sols, leur teneur en eau, l'effort compactif et les essais de contrôle, qui

The third text contains two excerpts from *Soils Manual for the Design of Asphalt Pavement Structures* (The Asphalt Institute, MS-10, Second Edition, March 1978). The first excerpt — *Chapter IV, Significance of Tests on Soil Materials* – notes that, although most highway engineers and field soils technicians are acquainted with the basic tests performed in soils laboratories, they are not thoroughly familiar with the test methods and/or the significance and interpretation of the test results due to their lack of experience with the tests. Four basic laboratory soil tests are described: mechanical analysis, specific gravity, consistency tests and indices, and the moisture-density test. Each test description includes (a) the significance of the test, (b) a synopsis of the test method, (c) typical test results, and (d) influences of the method of testing, i.e., the common sources of errors in the test results.

The second excerpt is ***Chapter VII, California Bearing Ratio of Laboratory-Compacted Soils.*** The California Bearing Ratio (CBR) is a widely used method of comparing the relative bearing values of base, subbase, and subgrade materials. The CBR is the load required to force a piston into the soil to a certain depth, which is expressed as a percentage of the load required to force the piston the same depth into a standard sample of crushed stone. Penetration loads for the crushed stone have been standardized. The resulting bearing value is known as the Califor-

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bargo, el ingeniero de carreteras en general muchas veces pierde de vista las interrelaciones complejas entre propiedades de suelo, contenido de humedad, esfuerzo compactivo, y ensayos de control que se deberán equilibrar correctamente para obtener un producto final satisfactorio a un costo razonable.

El tercer texto contiene dos extractos de *Soils Manual for the Design of Asphalt Pavement Structures* (Manual de suelos para el diseño de estructuras de pavimento de asfalto, The Asphalt Institute, MS-10, Second Edition, March 1978). El primer extracto, *Chapter IV—Significance of Tests on Soil Materials* (Capítulo IV — Significado de ensayos de materiales de suelo), observa que aunque muchos ingenieros viales y técnicos de suelos del campo conocen los ensayos básicos que se realizan en los laboratorios de suelos, no están totalmente familiarizados con los métodos de ensayo y/o el significado e interpretación de los resultados de ensayo a causa de una falta de experiencia con los ensayos. Se describen cuatro ensayos básicos de laboratorio para suelos: análisis mecánico, gravedad específica, ensayos e indices de consistencia, y el ensayo de humedad-densidad. Cada descripción incluye (a) el significado del ensayo, (b) un sinopsis del método de ensayo, (c) resultados típicos de ensayo, y (d) influencias del método de ensayo, es decir, los orígenes comunes de errores en los resultados de ensayo.

El segundo extracto es *Chapter VII – California Bearing Ratio of Laboratory-Compacted Soils* (Capítulo VII — Valor relativo de soporte de California de suelos compactados en el laboratorio). El valor relativo de soporte de California (CBR) es un método utilizado extensivamente para comparar los valores relativos de soporte de materiales de base, subbase, y subrasante. El CBR es la carga que se requiere para impulsar un émbolo a penetrar el suelo hasta cierta pro-

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doivent tous être correctement équilibrés, pour arriver à un résultat satisfaisant à un prix raisonable.

Le troisième texte contient deux extraits de *Soils Manual for the Design of Asphalt Pavement Structures* (Manuel des sols pour le calcul des chaussées en bitume asphaltique) publié par The Asphalt Institute, MS-10, Second Edition, March 1978. Le premier extrait, *Chapter IV – Significance of Tests on Soil Materials* (Chapitre IV — La signification des essais sur les matériaux routiers) remarque que, bien que la plupart des ingénieurs routiers et des spécialistes des sols, connaissent les essais de base effectués sur les sols en laboratoire, ils ne se sont pas nécéssairement entièrement familiarisés avec les méthodes utilisées pour faire ces essais, ou/et avec la signification et l'interprétation des résultats, à cause d'un manque d'expérience. Quatre essais de base en laboratoire sont décrits: a) signification de l'essai, (b) synopsis des méthodes employées, (c) résultats caractéristiques et (d) influence de la méthode d'essai, c'est à dire, la cause habituelle d'erreurs dans le résultats.

Le second extrait est *Chapter VII — California Bearing Ratio of Laboratory Compacted Soils* (Chapitre VII — Indice de portance californien des sols compactés en laboratoire). Le calcul de portance californien, ou essai CBR, est une méthode utilisée de façon extensive, où l'on compare les valeurs de portance relatives des matériaux de base, de sous-couche et du sous-sol. Le CBR est la charge nécessaire pour enfoncer un piston dans le sol jusqu'à une certaine complete inability to compact the soil to any acceptable density.

The text also discusses compaction methods and factors influencing compaction such as moisture content and control, soil mixing, lift heights, the use of ballast, compaction speeds and passes (which are interrelated), and weather.

Bibliography

The selected texts are followed by a brief bibliography containing reference data and abstracts for 20 publications. The first 10 describe the selected texts. The other 10 describe publications related to the selected texts. Although there are many articles, reports, and

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gundo principio es la acción de amasar, normalmente con rodillos pata de cabra; esta acción es muy útil en la compactación de materiales cohesivos, es decir, arcillas y arcillas limosas. El tercer principio es la vibración, representada por rodillos o planchas vibratorias de alta frecuencia, muy útil en la compactación de arenas y limos arenosos. El cuarto principio es el impacto, representado por pisones de baja frecuencia dirigidos a mano a utilizarse en áreas pequeñas o espacios encerrados.

No existe un solo compactador ideal para todos los suelos; sin embargo, rodillos combinados pueden utilizarse para una variedad de tipos de suelos. El resultado mínimo de un error en la selección del rodillo es un aumento en el costo de compactación, mientras que el peor resultado es la imposibilidad de compactar el suelo hasta una densidad aceptable.

El texto también examina los métodos de compactación y los factores que influyen en la compactación, tales como el contenido y control de humedad, mezclado del suelo, espesores de capas, utilización de balasto, velocidades y pasadas de compactación (que son interrelacionadas) y condiciones meteorológicas.

Bibliografía

A continuación de los textos seleccionados el lector encontrará una breve bibliografía que contiene los datos de referencia y extractos para 20 publicaciones. Las primeras diez referencias describen los textos seleccionados. Las otras diez describen publicaciones relacionadas con los textos seleccionados. Aunque existen muchos artículos, informes, y libros que podrían nombrarse, no es el propósito de esta bibliografía mencionar todas las posibles referencias que se relacionen con el tema de este compendio.

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tes classes de sol, avant et pendant l'effort compactif. On recommande aussi l'équipement correct (rouleaux) à utiliser sur les différents types de sol, et une gamme de profondeurs de couche (épaisseur non-compactée) qui devrait être utilisée pour différents types de sols.

Le dixième texte est extrait de *Earth Compaction* (Le compactage des sols) et comprend une partie des articles *3. Compaction Equipment* (Engins de compactage) et *4. Compaction Methods* (Procédés de compactaĝe) — Réimpression de *Construction Methods and Equipment,* McGraw-Hill Publishing Co., Inc., 1961). On y développe le choix de l'engin de compactage et de la profondeur des couches, introduits au texte précédent. Ce texte peut aussi servir de guide pour déterminer si l'offre d'un contracteur inclut un matériel de compactage adapté aux sols spécifiques anticipés dans un projet.

Le matériel de compactage utilise quatre principes différents (ou une combinaison de ceux-ci) pour accomplir l'objectif. Le premier principe est celui du poids statique, représenté par des rouleaux à bandages d'acier lisses, ou le compacteur à pneus, très utile pour le compactage des matériaux granuleux. Le deuxième principe est celui du pétrissage, accompli principalement par les rouleaux à pieds de mouton, très utile celui-là pour le compactage des sols cohérents comme l'argile et l'argile-limoneuse. Le troisième principe est celui de la vibration, représenté par les rouleaux ou plaques, vibrants à haute fréquence, très efficace pour compacter notemment les sables et les limons sableux. Le quatrième principe est celui de la percussion, et est representé par les pilons à basse fréquence à main, ou les engins dameurs, leur domaine d'emploi étant le compactage des emplacements réduits et des espaces restreints.

Il n'existe pas d'engin de compactage idéal pour chaque sol. Toutefois on peut utiliser des rouleaux qui combinent plusieurs fonctions pour compacter une quantité de sols divers. La conséquence minimale d'un choix d'engin de compactage erroné, est l'augmentation du prix de revient du compactage. La pire conséaccount and in-house construction projects must continually determine if compaction procedures are progressing properly. Otherwise, the formal testing effort will frequently be wasted on incompletely compacted materials.

This text presents simplified methods of determining the type of material that is actually being compacted in the field. It presents rules of thumb for determining the optimum compaction moistures for the various types of soil before and during the compactive effort. It also recommends that proper types of compactors (rollers) for use in the various types of soil and the range of lift depths (uncompacted thicknesses) that should be used for various soil types.

The tenth text is excerpted from *Earth Compaction* and includes parts of *3. Compaction Equipment* and *4. Compaction Methods* (reprinted from *Construction Methods and Equipment,* McGraw-Hill Publishing Co., Inc., 1961). It amplifies the selection of the proper compaction equipment and lift thicknesses introduced in the previous text. This text can also be used as a guide to determine if a contractor's proposal includes the proper compaction equipment for the specific soils expected on a project.

Compaction equipment uses four different principles (or combinations of same) to accomplish its objective. The first principle is static weight, as represented by smooth-steel-wheel or pneumatic-tired types of roller, most useful for compacting granular soils. The second principle is kneading action, mainly attributed to sheepsfoot rollers; this is most useful for compacting cohesive materials, i.e., clays and silty clays. The third principle is vibration, as represented by high-frequency vibratory rollers or plates, most useful for compacting sands and sandy silts. The fourth principle is impact, as represented by low-frequency hand-held tampers, or rammers, used in small areas and confined spaces.

No single compactor is ideal for every soil; however, combination rollers can be used on a variety of soil types. The minimum consequence of selecting the wrong type of roller is an increase in the cost of compaction; the extreme consequence of improper roller selection is the

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completamente compactados.

Este texto presenta métodos simplificados para determinar el tipo de material que realmente se está compactando en el campo. Presenta reglas empíricas para determinar la humedad óptima de compactación para los diversos tipos de suelo antes de y durante el esfuerzo compactivo. También recomienda los tipos correctos de compactores (rodillos) a utilizarse en los diversos tipos de suelos y los espesores de capa (espesor antes de compactar) que deberán utilizarse para varios tipos de suelos.

El décimo texto fué extraído de *Earth Compaction* (Compactación de suelos) e incluye partes de *3. Compaction Equipment* (3. Equipo de compactación) y *4. Compaction Methods* (4. Métodos de compactación) (reimprimido de *Construction Methods and Equipment,* McGraw-Hill Publishing Co., Inc., 1961). Presenta una amplificación de la selección de equipo de compactación y espesores de capa apropiados que se presentó en el texto previo. Este texto también puede utilizarse como guía para determinar si la propuesta de un contratista incluye el equipo de compactación correcto para los suelos específicos anticipados en un proyecto.

El equipo de compactación utiliza cuatro principios diferentes (o combinaciones de éstos) para realizar su objetivo. El primer principio es el peso estático, representado por rodillos de rueda lisa de acero o de llanta neumática, útil en la compactación de suelos granulares. El se-

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ing the Compaction of Soils (Essais manuels pour guider le compactage des sols) a été publié dans les *Proceedings of the 10th Annual Engineering Geology and Soils Engineering Symposium* (University of Idaho, 1972).

Dans le texte choisi no. 1 nous avons pû lire que le contrôle du compactage consiste en une inspection continuelle à l'aide d'essais officiels, pour s'assurer que l'exécution des travaux et les matériaux sont en accordance avec les spécifications. Malheureusement, beaucoup d'inspecteurs pensent que l'inspection continuelle consiste en le prélèvement d'échantillons a posteriori pour la réception. Cela n'est pas vrai. L'inspecteur de construction et le chef des travaux éxecutés en régie directe, ou pour le compte de l'administration, doivent continuellement déterminer si le compactage progresse de façon satisfaisante. Autrement, les essais officiels en laboratoires seront fréquemment gaspillés sur des matériaux pas complètement compactés.

Ce texte présente des méthodes simplifiées pour déterminer le genre de matériau actuellement compacté en chantier. Il présente des méthodes empiriques pour déterminer la teneur en eau optimale de compactage pour les differensequently, the situation as a whole is very much open to criticism for it considerably hinders exchange of information between countries and the progress of road technology.

The committee, therefore, is making recommendations on those matters that appear essential to ensure that certain tests are carried out uniformly in all countries. These recommendations are in the form of a proposal presented at the 1979 Vienna conference. The tests included are (1) particle-size distribution by sieving, (2) Los Angeles test, (3) sand equivalent, (4) polished-stone-value, (5) quality of fine material passing a 0.075-mm sieve, (6) density measurements (three tests), (7) aggregate shape (two tests), (8) sample reduction to provide the test sample, and (9) sensitivity to freezing.

The ninth text, ***Hand-Feel Tests for Guiding the Compaction of Soils,*** appeared in the ***Proceedings of the 10th Annual Engineering Geology and Soils Engineering Symposium*** (University of Idaho, 1972). Selected Text 1 pointed out that compaction control consists of continuous inspection with the use of formal tests to check the compliance of workmanship and materials with the specifications. Unfortunately, many inspectors feel that taking samples of the completed work for acceptance purposes is continuous inspection. This is not so. The construction inspector and the supervisors of force-

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en varios países. Algunas de estas diferencias son el resultado de distintos conceptos de las funciones de caminos. Otras diferencias no tienen base firme. En consecuencia, la situación puede censurarse en que obstruye el intercambio de información entre países y el avance de la tecnología vial.

Por lo tanto, el comité presenta recomendaciones sobre los temas que parecen esenciales para asegurar que ciertos ensayos se lleven a cabo en forma uniforme en todos los países. Estas recomendaciones toman forma de una propuesta que se presentó en la conferencia de 1979 en Viena. Los ensayos que se incluyen son (1) distribución de tamaños de partícula por tamizado, (2) ensayo de Los Angeles, (3) equivalente de arena, (4) ensayo de valor de piedra pulida, (5) calidad del material fino que pasa por un tamiz de 0,075 mm, (6) medidas de densidad (3 ensayos), (7) forma del agregado (2 ensayos), (8) reducción de una muestra para proveer la muestra de ensayo, y (9) sensitividad a la helada.

El noveno texto se titula ***Hand-Feel Tests for Guiding the Compaction of Soils*** (Ensayos por tacto para dirigir la compactación de suelos, ***Proceedings of the 10th Annual Engineering Geology and Soils Engineering Symposium,*** University of Idaho, 1972).

El Texto Seleccionado 1 indica que el control de compactación consiste en una contínua inspección con el uso de ensayos formales para asegurarse del cumplimiento de los requisitos de las especificaciones en lo que respecta a mano de obra y materiales. Desafortunadamente, muchos inspectores piensan que la toma de muestras del trabajo completado, para determinar aceptación o rechazo, es inspección contínua. Esto no es cierto. El inspector de construcción, y los supervisores de proyectos de construcción hechos por el departamento vial y de trabajos por administración deben determinar contínuamente si la compactación se está llevando a cabo correctamente. De otra forma ocurrirá que el esfuerzo de realizar ensayos formales será malgastado en materiales no

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ods to Be Used for Testing Aggregates – draft (Recommandations pour l'exécution des essais de granulats — proposition), est extrait de ***Technical Committee Report on Testing of Road Materials,*** Permanent International Association of Road Congresses, XVI World Road Congress, Vienna, 1979. Le rapport constate qu'il y a une très grande hétérogénéité des procédures d'essai en usage à travers le monde. Certaines de ces différences proviennent d'un concept différent du fonctionnement des chaussées, d'autres, au contraire, n'ont aucun fondement sérieux. Par conséquent, la situation dans l'ensemble, laisse la porte ouverte aux critiques, car elle entrave considérablement le transfert réciproque de l'information d'un pays à un autre, et le progrès de la technique routière.

Compte tenu de cette situation, le Comité des Essais de Matériaux Routiers a pris pour objectif de proposer des recommandations sur les sujets qui semblent être essentiels pour assurer l'uniformité de certains essais dans tous le pays. Ces recommandations ont été soumises à la conférence de Vienne en 1979. Les essais inclus dans ces recommandations sont (1) analyse granulométrique par tamisage, (2) essai Los Angeles, (3) equivalent de sable, (4) essai de polissage accéléré, (5) détermination de la quantité de fines passant au tamis de 0,075 mm, (6) mesures gravimétriques (3 essais), (7) mesure de la forme des granulats (2 essais), (8) préparation d'un échantillon pour essai, et (9) sensibilité au gel.

Le neuvième texte, *Hand-Feel Tests for Guid-*

and a paper on *Constant Dry Weight (C.D.W.) Test Procedure* (State of Vermont Agency of Transportation, 1979). The report introduces one of the most common problems in construction compaction control, i.e., how to make quick and reliable decisions in the field about the state of compaction of subgrades, embankments, and pavement courses. Several field compaction test methods are described and a new (in 1968) test procedure is introduced. The principle of the constant dry weight compaction method is that the volume of a fixed weight of soil is inversely proportional to its dry density, regardless of its moisture content. No correction for stone content of the in situ dry density is necessary because each test is self-contained.

Because the report stresses the theoretical aspects of the test, this text also includes additional information in the form of a letter with an enclosure that describes in full the field procedure currently in use in Vermont.

The eighth text, *Recommendations on Methods to Be Used for Testing Aggregates (draft),* is an excerpt from the *Technical Committee Report on Testing of Road Materials* (Permanent International Association of Road Congresses, XVI World Road Congress, Vienna, 1979). The report indicates that there is a very wide variety of test procedures in use in various countries. Some of these differences are the result of different concepts of road functions. Other differences have no sound basis. Con-

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El séptimo texto consiste en dos secciones, un informe titulado *The Constant Dry Weight Method—A No-Weighing Field Compaction Test* (El método de peso seco constante — Un ensayo sin pesado de compactación en el campo, Report RP 141, Department of Highways, Ontario, 1968) y un artículo sobre *Constant Dry Weight (C.D.W.) Test Procedure* (Procedimiento de ensayo de peso seco constante — C.D.W., State of Vermont Agency of Transportation, 1979). El informe presenta uno de los problemas más comunes en el control de compactación en la construcción, es decir, cómo llegar a decisiones rápidas y seguras en el campo sobre el estado de compactación de subrasantes, terraplenes, y capas de pavimento. Se describen varios métodos de ensayo de compactación en el campo y se introduce el procedimiento de un ensayo nuevo (1968). El principio del método de compactación C.D.W. es que el volúmen de un peso fijo de suelo es inversamente proporcional a su densidad seca, independiente de su contenido de humedad. No es necesaria una corrección, por contenido de piedra, de la densidad seca in situ ya que cada ensayo es independiente.

Ya que el informe da importancia a los aspectos teóricos del ensayo, este texto seleccionado también incluye información adicional en una carta con adjunto que describe totalmente el procedimiento de campo que hoy en día se utiliza en el estado de Vermont.

El octavo texto, *Recommendations on Methods to Be Used for Testing Aggregates – Draft* (Recomendaciones sobre los métodos a utilizarse para ensayar agregados — horrador, *Technical Committee Report on Testing of Road Materials,* Permanent International Association of Road Congresses, XVI World Road Congress, Vienna, 1979), indica que existe una gran variedad en los procedimientos de ensayo utilizados

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Le septième texte contient deux parties, un rapport intitulé *The Constant Dry Weight Method — A No Weighing Field Compaction Text* (La méthode du poids sec constant — un essai sans pesage de compactage sur le chantier) Report RP 141, Department of Highways, Ontario, 1968, et une communication nommée *Constant Dry Weight (C.D.W.) Test Procedure* (Méthode d'essai du poids sec constant CDW), State of Vermont Agency of Transportation 1979. Le rapport aborde un des problèmes les plus courants du contrôle du compactage en construction: comment décider sur le champs, et en toute fiabilité, de l'état de compactage des couches de forme, remblais et couches de la chaussée. Plusieurs méthodes d'essais de compactage sur le chantier sont décrites, et une nouvelle (1968) méthode d'essai est présentée. La méthode d'essai du poids sec constant, est basée sur le principe que le volume d'un poids fixe de sol est inversement proportionnel à sa densité sèche, quelle que soit sa teneur en eau. Il n'est pas nécessaire de faire une correction pour la teneur en pierre de la densité sèche in-situ car chaque essai est indépendant.

Nous avons décidé d'ajouter une documentation supplémentaire, en ce cas une lettre et une pièce jointe, qui décrivent en détail la méthode sur le chantier actuellement utilisée au Vermont, car le rapport lui même met plutôt l'emphase sur l'aspect théorique de l'essai.

Le huitième texte, *Recommendations on Meth-*

compacted by 56 uniformly distributed blows from the rammer."

The test procedures excerpted in this text are (a) density of soil and soil-aggregate in place by nuclear methods (shallow depth) T238-76 and (b) moisture content of soil and soil-aggregate in place by nuclear methods (shallow depth) T239-76. Other selected texts in this compendium make references to these nuclear tests. In reviewing many articles and papers on the use of nuclear testing equipment for publication in Compendium 10, it was found that conclusions about the reliability, true cost, total time savings, and practical use of nuclear testing varied significantly. Although the latest techniques for use of nuclear test equipment are included here (T238-76 and T239-76), no recommendations are offered for or against such use.

The seventh text consists of two sections, a report entitled ***The Constant Dry Weight Method–A No-Weighing Field Compaction Test*** (Report RP 141, Department of Highways, Ontario, 1968)

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la designación de AASHTO la letra "T" significa ensayo (test), los primeros dos o tres dígitos indican el número del ensayo y los últimos 2 dígitos después del guión indican el último año en que se introdujo un cambio en el procedimiento del ensayo.

La importancia de la fecha del último cambio puede demostrarse por el siguiente ejemplo. El Texto Seleccionado 2 se refiere al ensayo Standard Proctor como T99. Sin embargo, la descripción del ensayo se deriva de T99-61. En el método B de este ensayo, el texto indica que cuando se utiliza un molde de 6 pulgadas, los golpes se aumentan a 55 por capa. Sin embargo, el AASHTO T99-74 incluye lo siguiente en la descripción del método B ". . . compactando cada capa con 56 golpes del pisón uniformemente distribuídos."

Los procedimientos de ensayo extraídos para este texto son (a) densidad de suelo y suelo-agregado en obra por métodos nucleares (poca profundidad) T238-76, y (b) contenido de humedad de suelo y suelo-agregado en obra por métodos nucleares (poca profundidad) T239-76. Hay otros textos seleccionados en este compendio que se refieren a estos ensayos nucleares. Al repasar muchos artículos e informes sobre el uso de equipo de ensayo nuclear, para publicación en el Compendio 10, se descubrió que las conclusiones sobre la fiabilidad, costo verdadero, total de tiempo ahorrado, y uso práctico de ensayo nuclear varían significativamente. Aunque se incluyen aquí las últimas técnicas para el uso de equipo de ensayo nuclear (T238-76 y T239-76), no se presentan recomendaciones en pro o contra de tal uso.

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ods of Sampling and Testing, AASHTO, July 1978 (Normes pour les matériaux de transport [matériaux routiers] et méthodes d'échantillonage et d'essais, Deuxième partie, Méthodes d'échantillonage et d'essais, AASHTO, 1978). La table des matières de ce livre indique le répertoire complet des méthodes d'essais qui ont été normalisées par l'American Association of State Highways and Transportation Officials (AASHTO). La table des matières est sous-divisée en trois: section sur les matériaux, section numérique, et tableau de correspondance des normes AASHTO et ASTM. On signale au lecteur que pour AASHTO la lettre "T" signifie essai, les premiers deux ou trois chiffres indiquent le numéro de l'essai, et les deux derniers après le tiret, indiquent l'année de la plus récente révision de l'essai.

L'importance de la date de révision est démontrée par l'exemple qui suit: Le texte no. 2 s'en réfère à l'essai Proctor normal T99. Toutefois, la description de l'essai est celle de l'essai T99-61. Dans la méthode B de cet essai, on indique que quand un moule de 6 in. est utilisé, les coups sont augmentés jusqu'à 55 par couche. Mais dans l'essai AASHTO T99-74, on donne pour la méthode B "chaque couche étant compactée par 56 coups du piston, distribués uniformément."

Les méthodes d'essai qui sont données dans ce texte sont (a) Densité du sol et des mélanges de sol en place par méthodes nucléologiques (investigation superficielle) T238-76 et (b) Teneur en eau du sol et des mélange de sol en place, par méthodes nucléologiques (investigation superficielle) T239-76. D'autres textes de ce recueil s'en réfèrent à ces méthodes nucléologiques. Après avoir passé en revue beaucoup d'articles et de communications sur l'utilisation d'appareils nucléaires pour voir si nous allions les publier dans ce recueil, nous nous sommes aperçus que les conclusions sur la fiabilité, prix de revient actuel, le total des économies de temps, et l'utilisation pratique de ces méthodes nucléologiques, varient de façon importante. Bien que les techniques nucléologiques les plus récentes soient incluses (T238-76 et T239-76) dans ce recueil, nous n'offriron aucune recommandation pour ou contre leur utilisation.

cates the complete repertoire of test procedures that have been standardized by the American Association of State Highway and Transportation Officials (AASHTO). The table of contents is subdivided into a subject sequence, a numerical sequence, and a tabulation showing equivalencies between AASHTO and ASTM specifications. The reader should note that in the AASHTO designation the letter "T" means test, the first two or three digits indicate the test number, and the last two digits following the dash indicate the latest year in which a revision to the test has been made.

The significance of the revision date can be shown by the following example. Selected Text 2 refers to the Standard Proctor test as T99. However the description of the test is derived from T99-61. In method B of this test, the text indicates that when a 6-inch mold is used, the blows are increased to 55 per layer. However, AASHTO T99-74 includes the following in the description of method B: " . . . each layer being

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El quinto texto, *Chapter 2 — Test Procedures for Evaluation of Tropical Soil Properties* (Capítulo 2 — Procedimientos de ensayo para la evaluación de las propiedades de suelos tropicales, *Laterite and Lateritic Soils and Other Problem Soils of the Tropics, Volume II, Instruction Manual,* USAID, 1975) describe los procedimientos que se utilizan en los ensayos de durabilidad de gruesos (D_c) y finos (D_f) de California, que se evaluaron pero no fueron detallados en el texto previo. También se dirige al hecho de que las preparaciones de manipulación y precalentamiento que se especifican para el ensayo de suelos de zonas templadas muchas veces cambiarán las propiedades de suelos tropicales. Son casi siempre irreversibles los cambios en las propiedades ingenieriles que ocurren en el precalentamiento antes del ensayo.

El texto resume los procedimientos de ensayo más adecuados en la evaluación ingenieril de suelos tropicales. Se incluyen recomendaciones y modificaciones específicas con cada ensayo para compensar por la extraordinaria naturaleza de estos suelos. Los procedimientos de ensayo que se describen incluyen (1) preparación seca de muestras de suelo, (2) preparación húmeda de muestras de suelo, (3) preparación de muestras de suelo con contenido de humedad natural, (4) análisis de tamaño de partícula, (5) límite líquido, (6) límite plástico e índice de plasticidad, (7) relaciones de densidad/humedad, (8) gravedad específica, (9) California Bearing Ratio, (10) valor equivalente de arena, (11) los ensayos de durabilidad de California, (12) el uso del medidor PVC de suelos FHA, y (13) método sugerido de ensayo de la expansión unidimensional y subpresión de suelos arcillosos.

El sexto texto fué extraído de *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II — Methods of Sampling and Testing,* (Especificaciones normalizadas para materiales de transporte y métodos de muestreo y ensayo, Segunda parte — Métodos de muestreo y ensayo, AASHTO, July 1978). El índice de este libro indica el repertorio completo de los procedimientos de ensayo que han sido normalizados por la Asociación Americana de Funcionarios de Carreteras Estatales y de Transporte (AASHTO). El índice se subdivide en una secuencia de temas, una secuencia numérica y una tabulación que indica las equivalencias entre las especificaciones de AASHTO y ASTM. El lector deberá notar que en

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grains fins (D_f), ces méthodes ayant été évaluées de façon générale dans le texte précédent. On nous fait aussi remarquer que les préparations de pré-chauffage et de manipulation, spécifiées pour les essais sur les sols des pays tempérés, souvent changent les caractéristiques des sols tropicaux.

Le texte résume les meilleures méthodes d'essais pour l'évaluation routière des sols tropicaux. Des recommandations et des modifications spécifiques sont incluses pour chaque essai, pour compenser la nature inhabituelle de ces sols. Les méthodes d'essais qui sont incluses comprennent: (1) préparation d'éprouvettes sèches, (2) préparation d'éprouvettes imbibées, (3) préparation d'éprouvettes à la teneur en eau naturelle, (4) détermination des grosseurs des grains, (5) limite de liquidité, (6) indice de plasticité, et limite de plasticité, (7) relation entre la densité et la teneur en eau, (8) poids spécifique, (9) indice portant de Californie, (10) equivalent de sable, (11) essai de durabilité de Californie, (12) emploi du dispositif de mesure PVC (Potential Volume Change: essai de gonflement) de la FHA, et (13) méthode proposée d'essai uniaxial d'expansion et de sous-pression des sols argileux.

Le sixième texte est extrait de *Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II – Meth-*

scribes the durability tests considered most appropriate for use with lateritic material.

The fifth text, ***Chapter 2 – Test Procedures for Evaluation of Tropical Soil Properties,*** is excerpted from ***Laterite and Lateritic Soils and Other Problem Soils of the Tropics, Volume II, Instruction Manual*** (USAID, 1975). It describes the procedures used in the California coarse (D_c) and fine (D_f) durability tests, which were evaluated but not detailed in the previous text. It also addresses the fact that the manipulating and preheating preparations specified for the testing of temperate soils will often change the properties of tropical soils. The changes in engineering properties that occur with preheating prior to testing are usually irreversible.

The text outlines the test procedures most suitable for use in the engineering evaluation of tropical soils. Specific recommendations and modifications are included with each test to compensate for the unusual nature of these soils. The test procedures described include (1) dry preparation of soil samples, (2) wet preparation of soil samples, (3) preparation of soil samples at natural moisture content, (4) particle size analysis, (5) liquid limit, (6) plastic limit and plasticity index, (7) moisture density relations, (8) specific gravity, (9) California Bearing Ratio, (10) sand equivalent value, (11) the California durability tests, (12) using the FHA soil PVC meter, and (13) suggested method of test of one-dimensional expansion and uplift pressure of clay soils.

The sixth text is excerpted from ***Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II – Methods of Sampling and Testing*** (AASHTO, July 1978). This book's table of contents indi-

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que hay una desviación del procedimiento normal.

El cuarto texto, ***Mechanical Durability of Lateritic Gravels from Southeast Asia; Suggested Tests and Test Standards for Highway Uses*** (Durabilidad mecánica de gravas lateríticas del Asia del Sudeste; Ensayos y normas de ensayos sugeridos para utilización vial, ***Australian Road Research,*** Australian Road Research Board, 1970), describe una investigación que se realizó para determinar cuales ensayos son los más adecuados para evaluar la durabilidad mecánica de gravas lateríticas a utilizarse en la construcción vial. El ***Compendio 7: Gravas*** contiene varios textos seleccionados que explican los problemas singulares que los constructores de caminos en los trópicos tienen con los suelos lateríticos. Estas propiedades poco comunes de la laterita invalidan algunos de los procedimientos de ensayo que se utilizan en climas templados para la selección de materiales adecuados para la construcción de caminos.

La laterita es muy sensitiva a la degradación o desintegración en una estructura vial. Los ensayos comunes de durabilidad no siempre son lo suficientemente sensitivos para determinar si una laterita en particular se desempeñará en forma satisfactoria en uno u otro de los componentes de una estructura vial. Este texto describe los ensayos de durabilidad que parecen ser los más apropiados para el material laterítico.

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la déviation du procédé normal.

Le quatrième texte, ***Mechanical Durability of Lateritic Gravels from Southeast Asia; Suggested Tests and Test Standards for Highway Uses*** (Durabilité mécanique des graviers latéritiques de l'Asie du Sud-Est; Essais et normes d'essais suggérés pour leur utilisation routière), est un rapport publié dans ***Australian Road Research,*** Australian Road Research Board, en 1970. On décrit une investigation qui fut menée pour déterminer quels genres d'essais sont les plus appropriés pour évaluer la durabilité mécanique des graviers latéritiques utilisés dans la construction routière. Le ***recueil no. 7: Les graviers,*** contient plusieurs textes choisis qui expliquent les problèmes exceptionnels que les constructeurs de routes des tropiques ont avec les sols latéritiques. Ces caractéristiques inhabituelles de la latérite invalident certaines méthodes d'essais utilisées dans les climats tempérés pour la séléction de matériaux routiers.

Dans une structure routière, la latérite est très sensible à la dégradation ou désintégration. Les essais de durabilité normaux ne sont pas toujours assez sensibles pour déterminer si un type de latérite se conduira de façon satisfaisante s'il est utilisé comme constituant d'une structure routière. Ce texte décrit les essais de durabilité considérés les plus adéquats pour employer avec des matériaux latéritiques.

Le cinquième texte, ***Chapter 2 — Test Procedures for Evaluation of Tropical Soil Properties*** (Chapitre 2 — Méthodes d'essais pour l'évaluation des caractéristiques des sols tropicaux) est extrait de ***Laterite and Lateritic Soils and Other Problem Soils of the Tropics, Volume II, Instruction Manual*** (USAID, 1975). On décrit les méthodes utilisées pour l'essai de durabilité de Californie pour les grains grossiers (D_c) et les nia Bearing Ration with the percentage omitted. The text gives a detailed explanation of the test procedure, the equipment used, the preparation of the sample, and the calculation and correction of the stress-penetration curve of the American Society for Testing and Materials (ASTM) procedure, which is the simplest of the various CBR test procedures to use.

It should be noted that the remarks in this text concerning the moisture content in subgrades under "Full-Depth Asphalt Pavements" are not a part of ASTM D 1883, nor do they apply to the low-volume roads that are the subject of this compendium. However, *Compendium 7: Road Gravels* (p. 133) contains information concerning the use of unsoaked CBR values under certain environmental conditions in tropical areas. Most design criteria assume the use of soaked CBR values. Therefore, any CBR values determined from the use of unsoaked samples should be so noted to warn subsequent users of the deviation from the normal procedure.

The fourth text, *Mechanical Durability of Lateritic Gravels from Southeast Asia; Suggested Tests and Test Standards for Highway Uses,* is a report that appeared in *Australian Road Research* (Australian Road Research Board, 1970). It describes an investigation that was conducted to determine which tests are most suitable for evaluating the mechanical durability of lateritic gravels for use in road construction. *Compendium 7: Road Gravels* contains several selected texts that explain the unique problems tropical road builders have with lateritic soils. These unusual properties of laterite invalidate some of the test procedures used in temperate climates for selection of materials suitable for road construction.

Laterite is very sensitive to degradation or disintegration in a roadway structure. The usual tests for durability are not always sensitive enough to determine if a particular laterite will perform satisfactorily in one or another of the components of a road structure. This text de-

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fundidad. Esta carga es expresada como un porcentaje de la carga que se requiere para impulsar el émbolo a penetrar hasta la misma profundidad en una muestra normal de piedra triturada. Se han normalizado las cargas de penetración para la piedra triturada. El valor resultante de soporte se llama el California Bearing Ratio (CBR) con el porcentaje omitido. El texto presenta una explicación detallada del procedimiento de ensayo, el equipo utilizado, la preparación de la muestra, y el cálculo y corrección de la curva de esfuerzo-penetración del procedimiento ASTM (American Society for Testing and Materials), que es el más simple de utilizar entre los diversos procedimientos de ensayo de CBR.

Deberá notarse que los comentarios en este texto que se relacionan con el contenido de humedad de subrasantes bajo pavimentos de asfalto de profundidad total, no forman parte de ASTM D 1883, ni son aplicables a los caminos de bajo volúmen que son el tema de este compendio. Sin embargo, el *Compendio 7: Gravas* (página 133) contiene información sobre el uso de valores CBR no empapados en ciertas condiciones del medio ambiente en áreas tropicales. Casi todos los criterios de diseño suponen el uso de valores CBR empapados. Por lo tanto, cualquier valor CBR determinado por el uso de muestras no empapadas deberá notarse como tal para advertir a los usuarios subsecuentes de

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profondeur, cette charge étant exprimée en pourcentage de la charge nécessaire pour enfoncer le piston à la même profondeur dans un échantillon de référence en pièrre concassée. Les charges de pénétration dans la pierre concassée sont normalisées. La valeur de portance qui résulte s'appelle l'indice de portance californien ou CBR. Cet indice est exprimé en omettant l'expression mathématique du pourcentage. Ce texte donne une explication détaillée de l'essai, l'équipement utilisé, la préparation des éprouvettes, et du calcul et de la correction de la courbe tension/pénétration du procédé de l'American Society for Testing and Materials (ASTM) qui est le plus simple des nombreux procédés de calcul de l'indice CBR.

Il faut préciser que les remarques qui concernent la teneur en eau des couches de forme, sur lesquelles des revêtements entièrement en bitume asphaltique, sont posés directement, ne font pas partie de la spécification ASTM D 1883, et ne sont pas applicables aux routes économiques formant le thème de ce recueil. Cependant, le *recueil no. 7: Les graviers,* à la page 133, contient des instructions sur l'utilisation de l'indice CBR avant imbibition, pour certaines conditions metéorologiques des régions tropicales. La plupart des critères de calcul présument l'utilisation des valeurs CBR après imbibition. Donc on devra signaler tout indice CBR déterminé en utilisant des échantillons avant imbibition, de façon à avertir les futurs utilisateurs de

xxviii Labor-Intensive Rural Access Roads Programme in Kenya uses long-term environmental compaction before reshaping and applying surfacing material (photo courtesy of TRRL, United Kingdom).

books that could be listed, it is not the purpose of this bibliography to contain all possible references related to the subject of this compendium. The bibliography contains only those publications from which a text has been selected or basic publications that would have been selected had there been no page limit for this compendium.

Contiene únicamente aquellas publicaciones de las cuales se seleccionó texto y publicaciones básicas que se habrían seleccionado si no hubiera un límite al número de páginas en este compendio.

quence est l'impossibilité totale de compacter le sol à une densité acceptable.

On discute aussi des méthodes de compactage, des facteurs qui influencent le compactage, tels que la teneur en eau, le contrôle de cette teneur en eau, le malaxage des sols, la hauteur des couches, le lestage, la vitesse de compactage et les passes (corrélatives), et enfin, les conditions atmosphériques.

Bibliographie

Les textes choisis sont suivis d'une brève bibliographie contenant les données de référence et les analyses de 20 publications. Les 10 premières s'en réfèrent aux textes choisis. Les dix autres décrivent des publications apparentées au thème des textes choisis. Bien qu'il y ait beaucoup d'articles, rapports et livres qui pourraient être inclus, cette bibliographie se rapporte seulement aux publications dont nous avons choisi des extraits, ou à des textes de base que nous aurions choisis aussi, s'il n'y avait pas de limite quant au nombre de pages de ce recueil.

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This section of the compendium contains selected pages from each text that is listed in the table of contents. Rectangular frames are used to enclose pages that have been reproduced from the original publication. Some of the original pages have been reduced in size to fit inside the frames. No other changes have been made in the original material except for the insertion of occasional explanatory notes. Thus, any errors that existed in the selected text have been reproduced in the compendium itself.

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Cada texto comienza con una o más páginas de material de introducción que contenía la publicación original. Este material generalmente incluye una página título, un índice, o ambos. Los asteriscos que han sido agregados al índice original significan lo siguiente:

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La pagination originale apparaît à l'intérieur de l'encadrement. La pagination du recueil est à l'extérieur de l'encadrement, soit à droite, soit à gauche de la marge extérieure des pages, et est celle qui est citée dans la table des matières et dans l'index du recueil.

Chaque texte commence par une ou plusieurs pages d'introduction qui étaient incluses dans le texte original. Ces pages sont généralement le titre, ou la table des matières, ou les deux. Des astériques ont été ajoutés à la table des matières d'origine, pour les raisons suivantes :

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en el texto seleccionado, pero otras páginas (o partes de página) en esta parte de la publicación original han sido omitidas.

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incluses dans les textes choisis, mais d'autres pages (ou portion de pages) de l'édition originale ont été omises.

**Toutes les pages dans cet extrait du document original sont incluses dans les textes choisis.

Les textes choisis, donc, incluent seulement ces extraits des documents originaux qui sont précédés d'un astérique dans les tables des matières des publications respectives.

Les lignes brisées sur les pages des textes choisis indiquent les endroits où le texte original a été omis. A certains endroits, les textes choisis contiennent des explications qui ont été insérées par notre personnel. Ces explications sont entourées d'un encadrement en pointillé, et commencent toujours par le mot NOTE.

PROCEEDINGS

of the

Golden Jubilee Convention

Canadian Good Roads Association

The Queen Elizabeth

Montreal, Quebec

October 19 to 22, 1964

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SIGNIFICANCE OF QUALITY CONTROL

Byron T. Kerr and Gilles G. Hénault
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As a continuation to this symposium on Quality Control, the following paper deals with two main topics: Inspection and Testing in Highway Construction Work, and Specifications.

We intend to comment first on the purposes of Quality Control and give the procedures usually followed to ensure the quality of workmanship and materials.

This will lead to a discussion of two important questions on which the value of inspection and testing greatly depends. These are, the nature of the samples and the application of the test results.

We shall then discuss the need for the specifications to be realistic and some ways of achieving this goal.

Finally, some suggestions are given on how inspection and testing can be effective in spite of normal variations in test results and in spite of delays due to the time required for some tests.

Purpose of Inspection and Testing

Inspection and testing performed directly on the job form an important part of quality control. Their purpose is to ensure that the material and workmanship are of a high enough standard to provide a highway which will perform satisfactorily and economically throughout its intended operating life.

This goal is attained by providing an inspection team under professional supervision which will verify and test the quality of the materials not only with respect to the requirements of the specifications but also with respect to the safe and economical solution of problems not anticipated in the design.

Continuous inspection and testing are the tools necessary to prevent unacceptable results caused by such factors as poor workmanship, changes in the sources of materials, unsuitable equipment and long exposure to adverse conditions.

General Procedure for Quality Control

The following steps are generally followed in order to obtain a good control of quality:

1) A study is made of the soil reports, specifications and plans so that the inspection team can get thoroughly acquainted with the materials, requirements, and any special problems which may be expected.

2) A review is made of the contractor's schedule and the equipment to be used for construction and for the processing materials. This enables the inspection team to prepare its own schedule so as to have enough men and facilities available, and to co-ordinate their work with that of the contractor and thus avoid delays.

3) Samples are obtained from the sources of materials proposed by the contractor for all types of work involved. The results obtained from these samples and an examination of the site enable the laboratory to determine the quality and degree of uniformity of the materials and to advise the contractor on the suitability of the material before construction starts. This also serves as a guide to the degree of quality control which will be necessary during the course of the project.

4) During construction, the laboratory must check the compliance of workmanship and materials with the requirements of the specifications. Different degrees of verification are necessary depending on the nature of the work. In highway construction, such items as the physical properties of embankment, base and pavement materials must be verified. It is equally important to verify the uniformity in the nature and compaction of the different layers placed, as well as the stability of embankments, subbase and base. Other possible causes of failure to look for are the presence of buried organic and other unsuitable materials, basins where water can collect, springs and insufficient drainage.

5) Continuous preparation and maintenance of progress charts, statistical analyses and other pertinent data should also be carried out. These records are extremely useful in showing rapidly the quality and uniformity of the finished products and they permit field decisions to be made quickly when problems arise.

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6) Detailed observations of factors affecting the quality of the work (such as weather) should also be made so that appropriate action can be taken quickly. A common example of the value of such observations would be the protection of concrete against cold temperature either by heating, insulation or other means.

Value of Inspection and Testing

Because of the great importance of control during construction, visual inspection is not enough and must be supplemented by the application of various testing techniques. Acceptance criteria are based mostly on tests which have been standardized. The behavior of existing highways has been correlated with the test results of the materials used in their construction and present-day acceptance criteria are based upon these relationships.

Thus, in the normal course of quality control various tests are performed directly at the site and others are made on samples of the materials from pits or borrow areas. The results of these standard tests performed by trained personnel form the basis for acceptance or rejection of the materials or work. Standard tests are fairer and more accurate than results obtained from visual inspection, which can vary depending on the individual and the circumstances. Control tests in common use for road materials are moisture determinations, grain size analyses, Atterberg limits, field density, Proctor, CBR, abrasion and soundness tests, and petrographic number determinations.

A similar range of control tests exists for other materials, such as concrete and asphalt.

Tests results have the greatest importance for proper quality control. They affect the design of the highway and therefore its performance and life — all factors that are of critical importance. They can also impose unnecessary expense on the contractors if invalid results are used to call up more expensive materials, equipment or methods of work.

Thus, even if quality control is performed by well trained personnel under professional supervision, there remain two important problems: namely, how representative are the samples submitted for testing, and what is the validity of test results?

Nature of Samples

The materials used by highway engineers are the normal components of the earth's crust, i.e., bedrock and soils. Everyone familiar with geology or with excavation work knows how complex can be the distribution of different soils or rock formations. The appraisal of a possible source of borrow materials should be done only by specialists familiar with the technical factors involved in the formation and transformation of the earth's crust, with experience in regional geology and with a full knowledge of the purpose of the materials. Such experts will aid considerably to obtain, at minimum practicable cost, useful information regarding the use of any deposit; because of their special training, they can best outline the number and location of samples to be taken to obtain a representative result. With their help, uneconomical deposits can be rejected early and suitable deposits can be processed as cheaply as possible to obtain a uniform product.

Moreover, for the determination of the quality and uniformity of materials either in place, in stockpiles, in bins or in trucks, various effective sampling techniques have been derived on a statistical basis. After careful verification and experimentation, these techniques have been approved and are recommended by well-known associations devoted to the testing of materials. Thus, under the supervision of experts who can select and adapt proper sampling techniques, trained personnel can secure samples which can be used, in fairness to all parties concerned, to measure the properties of any material, its homogeneity or heterogeneity and the absence or presence of segregation. Also, for the special cases where non-homogeneous materials can be used, experience and careful sampling can provide effective quality control.

For example, if an earth fill is to be built from an earth cut with a complex system of different acceptable soil formations, the samples taken for Proctor determinations may not be representative of a whole layer as placed on the job because of the non-uniform blending that takes place. In such a case, experienced personnel are especially required, because considerable judgment of the test results is necessary, together with many samples, in order to be fair to all parties concerned.

Value of Test Results

Most of the control tests used in the highway field are arbitrary in the sense that they do not represent actual field conditions at all times, or because different chemical or physical means are to be used to simulate field conditions in order to accelerate obtaining the results. Other tests, like classification tests, do not measure physical properties but permit us to relate the material to a given group of soils having significant characteristics determined through observation and experimentation, such as frost susceptibility or swelling susceptibility.

Although such tests are arbitrary, they are correlated to the field performance of the property being investigated, and therefore give acceptable and reliable results. In other words, a correlation has been established between the results of standard control tests performed on existing highways and the performance of those highways. Thus, it is possible to predict the performance of new construction in the light of test results and field conditions. This means that the results of control tests must be interpreted by a specialist who can decide if an unacceptable result is in fact significant in a particular job condition. For instance, a highly frost-susceptible material can be used if it is not within reach of frost or not critically near the base.

A specialist in materials also appreciates the limitations that certain tests have and, in these instances, would not rely entirely on the test results but would also take into consideration the performance of similar materials placed elsewhere under the same conditions, or rely on his judgment, experience and correlation with other test results. 9

So long as standard procedures are followed and the results are interpreted by experts, test results are not only valid but also fair to all parties concerned.

Specifications

Specifications in a broad sense consist of the volume of documents describing in detail the work which is to be done, the materials to be used, the quantity of materials needed and the quality that must be achieved. They must include sufficient data to ensure the execution of the contract in the best possible way for both the owner and the contractor and with the least possible friction between the owner, the contractor, the engineers and the laboratory. Specifications are especially important for the control of construction. It is sometimes forgotten by the specification writer that the end result of the work depends largely on how quality control is performed; on the other hand, how well the quality control is performed depends on how the specifications are written.

Besides a detailed description of minimum requirements, the specifications must also include details of the acceptance criteria. This is necessary to eliminate differences in interpretation which can lead to claims or losses on the part of the contractor. It is only by having very clear, exact and concise specifications that the required end results can be obtained. It is too often found that problems are encountered because of lack of precision in the specifications.

It is most important to have very clear specifications. Most highway projects are awarded on public tender. When a contractor is bidding for a job, the specifications should be so clear that he does and should not have to make hazardous interpretations of what is specified. If the specifications are not clear, contractors may have to assume or make their own interpretation of the intent of the specifications. This can produce a wide range of interpretations of the intent of the specifications, thus a wide range of prices; a contractor whose price might otherwise have been the lowest may lose the job because his interpretation of the specifications was stricter than his competitor. Such circumstances are particularly common in the interpretation of specifications on materials and this situation occurs particularly when all contractors have a technical staff who can make such interpretations of the specifications.

On the other hand, if the successful contractor makes too loose an interpretation of a specification that is not precise, we generally can expect much argument during the execution of the work, and the possibility of claims. So long as the specifications are clear, it follows that no claims should be allowed to a contractor for a lack of technical staff or misinterpretation of specifications, since this is not fair to the unsuccessful bidders.

If the specifications are clear and no different interpretations are possible, it may be expected that the bid prices will be within a very small range. If clear specifications cannot be written, some other form of contract is desirable and would give better results, but a cost-plus contract is usually not a satisfactory substitute for adequate pre-engineering.

Material specifications should also give enough latitude to the contractor so that he can, for example, use equipment which will allow him to reduce his operating costs to the benefit of both contractor and owner.

If specifications are vague, good quality control is difficult without introducing the possibility of claims from the contractor. A good portion of the claims and problems in highway construction deals with materials. To overcome this, we believe that material engineers should participate in the writing of the detailed specifications. Material engineers participating in the pedological study of the right-of-way should also participate in the design, because they are familiar with technical conditions and can be more realistic in their approach. This could prevent specifying materials of such a high quality that they cannot be found within an economic radius; similarly, it is unrealistic to specify the highest standard of material for a second-class road.

Likewise, enforcement of the specifications should be done by the same group of specialists who carried out the preliminary studies and collaborated in the design of the project and in writing the specifications. Logically, they are better equipped to handle the special problems occurring during the construction, mainly because they are already familiar with conditions and materials on the job.

Effectiveness of Job Control

Variations in test results are small enough not to be a problem. All control tests follow well-defined procedures and a well-trained technician can obtain or reproduce the same results from the same material to a great accuracy in most tests. The results of tests for which the procedures are not clearly defined should not be used for the rejection of work or materials unless a detailed study is carried out. The accuracy of test results is continuously under study by Associations of Testing Materials and efforts are being made to improve and standardize them.

All standard control tests have been checked not only for reproductiveness of results but also for realism against what average modern construction equipment can perform in the field. It is then possible to avoid asking for the impossible.

Since the specified requirements usually represent a minimum rather than an average, contractors must plan to get better average results than the minimum set forth in the specifications. The margin between the minimum specified and the average intended compensates easily for the small variations in the test results obtained when these results are done on representative samples by experienced technicians. On the other hand, statistically it is to be expected that some results can be low but still acceptable.

In the same manner, we do not feel that the time needed to perform the tests is a major problem preventing the effectiveness of quality control. Generally, only control tests that can be done quickly on the site are used to accept or reject a portion of materials or of the finished job. Other longer tests are used in evaluation of the sources of materials before the job is started and are repeated afterwards only as a routine test to make sure of the correlation between the test results and original observations. This is the case, for instance, for the soundness test for which the results cannot be obtained before 6 days or longer but the behavior of the aggregate can be predicted in a fairly accurate way by the petrographic analysis which can be done in one day.

Research is continually under way to develop and prove out new pieces of equipment that could provide faster results. Among these are the Nuclear Densimeter for soils and asphalt, rapid curing methods for concrete, the Benkelman Beam for roads and petrographic numbers for aggregates.

Most delays can be eliminated if field operations are well planned and schedules of work or changes to them are known in advance by the inspection force. Likewise, long delays can be avoided if the contractor has a good knowledge of his sources of materials and is not using sources of marginal quality.

Conclusions

The aim of quality control is to ensure for the owner a job which satisfies the intent of the specifications and safeguards his expenditure. For different reasons, quality control benefits all parties concerned: the owner, the engineer, the contractor, and the material supplier. All four have the same common interest in doing a job with which all are satisfied. 11

With established procedures for sampling and testing, experienced personnel can do representative sampling and obtain reliable test results.

Quality control must also include adequate pre-engineering, without which the design might have to be changed and the money spent be considerably higher than estimated in order to get the specified quality.

Specifications should be written in collaboration with the materials or quality control engineer. They should be concise and well defined in order to be fair both to the owner and contractor, and should leave initiative to the bidders in the choice of techniques and equipment, provided that the quality specified is obtained.

String lines define proper crown and grade of single-lane, labor-intensive embankment construction; final compaction occurs under traffic load (Mexico).

Highway Materials

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Compaction

The unconsolidated material of pavements and pavement foundations must above all remain stable during the life of the pavement structure. This means that the soil in fills must be sufficiently dense and strong to resist consolidation under its own weight or sliding along its slopes. The soil of compacted subgrade and base courses must resist densification and deformation under repeated wheel loads. Subgrade must not change volume excessively during wet periods or from frost action. Accordingly, fill, subgrade, and base cannot be haphazardly placed, but must be constructed with a suitable amount of stabilization.

Soil stabilization for pavement foundation is normally achieved by *compaction*, the densification of material by means of mechanical manipulation. Typically, a uniform layer or "lift" of 4 to 12 in. of material is compacted with several passes of heavy compaction equipment, such as a roller, until a specified unit weight is achieved. Such parameters as lift thickness, type of compaction equipment, and required density may be varied according to the material being stabilized and the component of pavement or foundation being constructed. The objective of the compaction is twofold: to provide a stable material and to provide a material whose properties are known, at least within limits. If the compaction yields a pavement component with a particular degree of stability, it is important to pavement design that this degree of stability be a known or predictable amount. As emphasized in the foregoing chapters, different soil materials are widely different in their resistance to deformation, even when they are handled similarly. Further, similar soils differ at different levels of compaction. Thus, manner and degree of compaction, nature of the soil to be stabilized, and the final product of the construction procedure are all closely related and, in turn, relate to pavement design and performance.

The subject of stabilization of highway materials also includes treatment of soil or aggregate with chemicals that provide cementation after compaction is completed. Most common among the chemicals used are lime for clay soils and portland cement for more granular materials. When such chemicals are used, the mixing, compaction, and subsequent cementation are referred to as *lime stabilization* or *portland cement stabilization*. Stabilization without chemical admixtures is usually referred to simply as *compaction*. Chemical stabilization is discussed in Chap. 8.

Engineering for soil stabilization by compaction requires establishing the compaction requirement for the material, usually in the laboratory; specifying the conditions and degree of compaction for the field; and inspecting, with tests, the product obtained in the field so as to maintain proper field control. Considering that this must be done for each material to be used and each component of the pavement foundation, including base-course materials and fill, and recognizing that gross differences exist between laboratory- and field-compaction methods, it is a remarkable achievement that the procedure is effective in producing the sound finished product found beneath most modern highways. 21

FUNDAMENTAL CONCEPTS

An objective of compaction is to rapidly pack a maximum of soil solids in a unit volume. With few exceptions, soil water content remains constant and the compaction consists of expelling air as solids assume positions or orientations favorable for tight packing. The amount of packing is measured in terms of dry unit weight γ_d, commonly referred to as *dry density*, which is a direct measure of solids per unit volume. The degree of compaction depends primarily on three important factors: (1) soil moisture content during compaction, (2) soil type, and (3) nature and amount of compactive effort. Moreover, these factors are interrelated. Thus, a highly plastic clay soil may compact best at a relatively

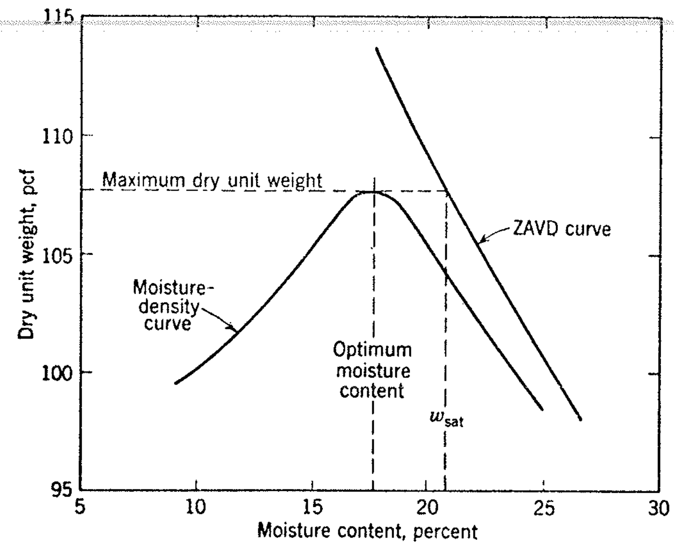


Fig. 7-1 Typical moisture-density curve developed by soil compaction and ZAVD (zero air-voids density) curve for $G_s = 2.70$.

high water content with the kneading action of a sheepsfoot or pneumatic-wheeled roller, whereas a granular soil may compact well, and to a much higher density than the clay, at a relatively low water content with a vibratory roller. In the laboratory, one may examine the effect of any one of the three factors by holding the other two constant, but their strong interrelationships should not be forgotten.

7-1 MOISTURE-DENSITY RELATIONS

It was learned long ago that moisture facilitates soil compaction unless it becomes so abundant as to occupy volume that would otherwise be taken by solids. Thus, for a particular soil type and compactive effort, as the water content of compaction increases from a low level, the achieved dry density will increase as well. Soon, however, a point is reached where increasing water content has an adverse effect, causing dry density to decrease. Thus, there is an *optimum water content* at which a *maximum dry density* may be reached for each type of soil and compaction. A typical *moisture-density curve*, attained for a single soil and compactive effort by changing compaction moisture content, is shown in Fig. 7-1.

Intuitively, the reason for the influence of moisture on dry density is simply the lubricating action of water on relatively dry soil grains, which allows them to slip or shear, one grain against another, as they move into positions of tight packing. Too much water presumably prohibits tight packing by occupying space otherwise available for solids. Theoretically, this would occur where compaction expels all the soil air and complete saturation results. In practice, however, a small amount of air remains entrapped in the solid-water system, especially in fine-grained soils. This is illustrated by comparing the moisture-density curve with a *zero air-voids density*, *ZAVD*, or saturation-moisture-content curve, shown in Fig. 7-1. The ZAVD curve, which represents dry density at zero air voids or 100 percent saturation with water, may be computed knowing the specific gravity of the soil solids and using either of the formulas below:

$$\text{ZAVD} = \frac{\gamma_w G_s}{wG_s + 1} \tag{7-1}$$

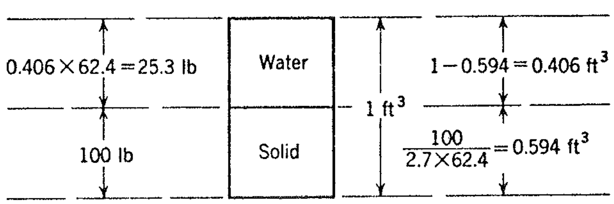
$$w_{\text{sat}} = \frac{\gamma_w}{\gamma_d} - \frac{1}{G_s} \tag{7-2}$$

Degree of saturation for any compaction water content may be computed from

$$S = \frac{w}{w_{\text{sat}}} \tag{7-3}$$

Thus, to determine the degree of saturation for a particular dry unit weight and compaction moisture content, one need only find the saturation moisture content for the dry density from the ZAVD curve and divide it into the water content of compaction.

It should be noted that phase diagrams as illustrated in Chap. 1 may be used in lieu of formulas. For example, if it is known that $\gamma_d = 100$ pcf and $G_s = 2.70$, then the saturation moisture content can be calculated as follows:



$$w_{\text{sat}} = \frac{25.3}{100} \times 100\% = 25.3\%$$

Significance of saturation moisture content Saturation water content is a valuable indication of the potential water content of a compacted soil in the field. Soil beneath highway pavements is likely to become saturated through percolating meteoric water, capillary rise of groundwater, or frost action. If saturation water content is known, the consistency of a fine-grained soil so saturated may be surmised. Thus, although cohesive soil is generally compacted at water contents less than the plastic limit, future saturation may result in material closer to the liquid limit, with consequent reductions in soil stability. A general rule of thumb is that when the saturation moisture content is closer to the liquid limit than to the plastic limit, compaction is inadequate. This is especially true when compaction dry density is low or when significant swell attends the saturation. If the degree of swell caused by saturation can be estimated from laboratory work, a new dry density $\gamma_{d_{sat}}$ can be estimated using the formula

$$\gamma_{d_{sat}} = \frac{\gamma_d}{1 + \text{swell}} \tag{7-4}$$

and a revised saturation moisture content can be determined with the ZAVD curve as before.

The utility of saturation moisture content is illustrated assuming that a soil consists of a mixture of 70 percent coarse aggregate and 30 percent fines, by weight. Such a mixture, lacking the intermediate fine aggregate for good gradation, is unlikely to be used as construction material, but it may occur as a result of an intrusion of fines from a subgrade into an open-graded aggregate placed to facilitate drainage. If the dry density of the mixture is 100 pcf, then a cubic foot of the soil contains 70 lb of coarse aggregate and 30 lb of fines. From the ZAVD curve in Fig. 7-1, the saturation water content of the soil is approximately 25 percent. This represents 25 lb of water per cubic foot of soil. The coarse aggregate may absorb as much as 3 lb of this water in rock pores, leaving 22 lb as water associated with the fines. This yields a water content for the fines of $^{22}/_{30}$ = 73 percent, a value in excess of the liquid limit for most fine-grained soils of low to intermediate plasticity. It is important to recognize that the fines will form the matrix of the soil and largely govern its behavior. Thus, mixtures of fine-grained soil and stone at low dry density may be highly unstable at saturation water content. The saturation moisture content of the soil fraction of a soil-aggregate mixture should always be examined if the fines are in sufficient amount to form a matrix in which the granular particles will float. Depending upon the gradation of the granular material, this can occur at as low as 20 percent fines where the granular material is relatively dense graded.

Establishing the moisture-density curve As will be emphasized in subsequent discussion, the moisture-density curve, including maximum dry unit weight and optimum moisture content, is influenced by a large number of variables, including soil type, nature of the compaction, level of compactive effort, and the manner in which the soil is handled before and during the compaction process. This greatly complicates the characterization of the compaction properties of soils and makes very difficult the extrapolation of laboratory-compaction results to field situations. It is nevertheless valuable to study the laboratory-compaction properties of soils prepared by standardized compaction tests as a starting point and guide to specifying field-compaction conditions.

With this objective in mind, R. R. Proctor published in 1933 a series of four articles [15] on soil compaction. He described in the second of this series a laboratory-compaction test which became known as the "standard Proctor" test. With some modification, the test has been adopted as a standard test by the American Association of State Highway Officials (AASHO T 99-61) and by ASTM (D 698-66T). The test provides for dynamic compaction using 25 blows of a hammer on each of 3 layers of soil in a cylindrical mold, as illustrated in Fig. 7-2. The compactive effort is 12,400 ft-lb/ft³, which is roughly equivalent to light

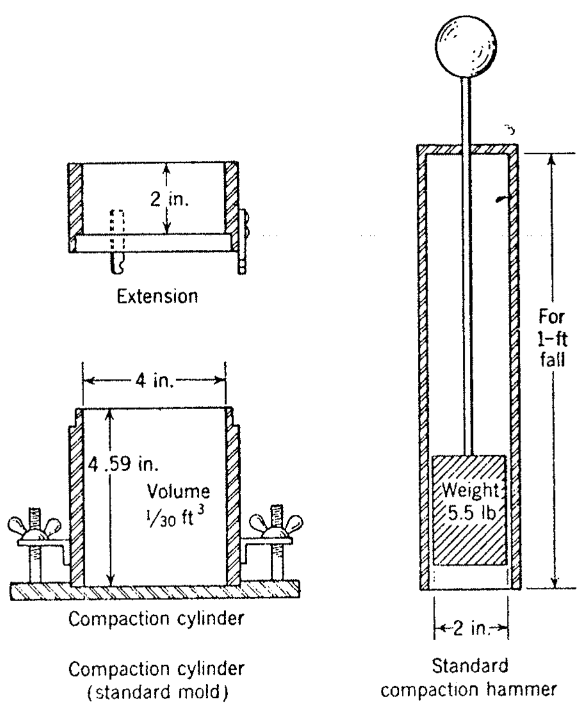


Fig. 7-2 Compaction equipment for the standard AASHO compaction test.

rollers or thorough tamping in the field. Later, a modified version of the test was developed to correlate better with the compactive efforts comparable to that obtained with heavy rollers under favorable working conditions. In the modified test, a 10-lb hammer with an 18-in. stroke is used to compact the soil in 5 layers, 25 blows per layer, giving a compactive effort of 56,300 ft-lb/ft^3 (AASHO T 180-61 and ASTM 1557-66T). The tests are commonly called "standard AASHO" and "modified AASHO" or simply T 99 and T 180. In most soils, the modified method results in maximum dry densities from 3 to 6 pcf greater than the standard.

Both AASHO tests provide for four separate subprocedures: A-4 in mold, soil passing No. 4 sieve; B-6 in mold, soil passing No. 4 sieve; C-4 in mold, soil passing a ¾-in. sieve; and D-6 in mold, soil passing a ¾-in. sieve. Procedure *A* is normally used for fine-grained subgrade soils.

The soils are normally air-dried and sieved through a No. 4 sieve. Moisture is then added and the compaction curve is developed by mixing, compacting, measuring achieved density and moisture content, disaggregating the compacted soil, incrementing the moisture content, and repeating the process until the soil is substantially wet of optimum. If the soil contains large amounts of material larger than the No. 4 sieve, and one wishes to incorporate this larger material into the compaction test, a 6-in.-diameter cylinder of the same height may be used and the blows increased to 55 per layer (method *B*). Reusing previously compacted soil during the test tends to increase dry density by a few pcf, but processing a sufficient quantity of soil to provide an unused specimen for each moisture level is usually inconvenient. There are many other shortcomings to the standard test procedures, such as air-drying the soil during sample preparation, and using a compaction process that is little like the action of field equipment; but as with many empirical soil tests, numerous studies and field correlations make the results valuable to the experienced engineer who recognizes both their limitations and their significance.

A compaction test that perhaps better simulates the field compaction of clays is the test developed by Wilson [24] known as the "Harvard Miniature" compaction test. The apparatus consists of a small cylindrical mold and a tamper employing a spring-loaded plunger. Compaction is essentially a rodding action, best described as kneading compaction. Tamping force, number of blows per layer, and number of layers can be easily adjusted to yield the desired compactive effort. The test has the advantage of using small soil samples that need not be reworked and that are suitable for unconfined compression testing without trimming. The compaction action probably resembles that obtained in the field with a sheepsfoot roller. It has the disadvantage of not having been widely correlated with standard tests and field-compaction results, but it is a valuable sample-preparation technique for research investigations.

A large number of other procedures for obtaining moisture-density curves have been developed, such as using static pressures, vibratory compaction, and kneading action. However, the test equipment is often more complex than with the standard tests and the added expense in relation to their uncertain advantage in simulating field compaction has restricted their use primarily to special investigations.

7-2 COMPACTION IN RELATION TO SOIL TYPE

Maximum dry density and optimum moisture content for a given compactive effort and compaction method are primarily influenced by soil type. Maximum dry unit weights as low as 60 pcf and as high as 145 pcf have been reported and optimum moisture contents may range from about 5 percent to 35 percent. The higher optimum moisture contents are generally associated with the lower dry densities. High unit weights are associated with well-graded granular soils containing just enough

Soil texture and plasticity data

No.	Description	Sand	Silt	Clay	LL	PI
1	Well-graded loamy sand	88	10	2	16	NP
2	Well-graded sandy loam	78	15	13	16	NP
3	Med.-graded sandy loam	73	9	18	22	4
4	Lean sandy silty clay	32	33	35	28	9
5	Lean silty clay	5	64	31	36	15
6	Loessial silt	5	85	10	26	2
7	Heavy clay	6	22	72	67	40
8	Poorly graded sand	94	– 6 –		NP	–

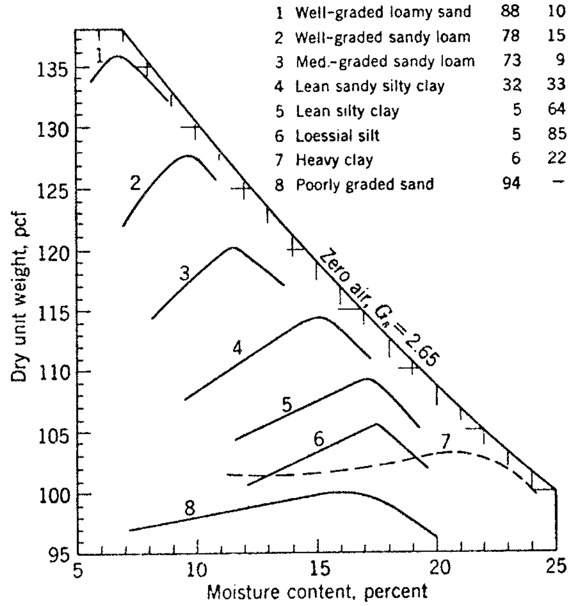


Fig. 7-3 Moisture-content–dry-unit-weight relationships for eight soils compacted according to AASHO T 99. (*From Johnson and Sallberg* [7].)

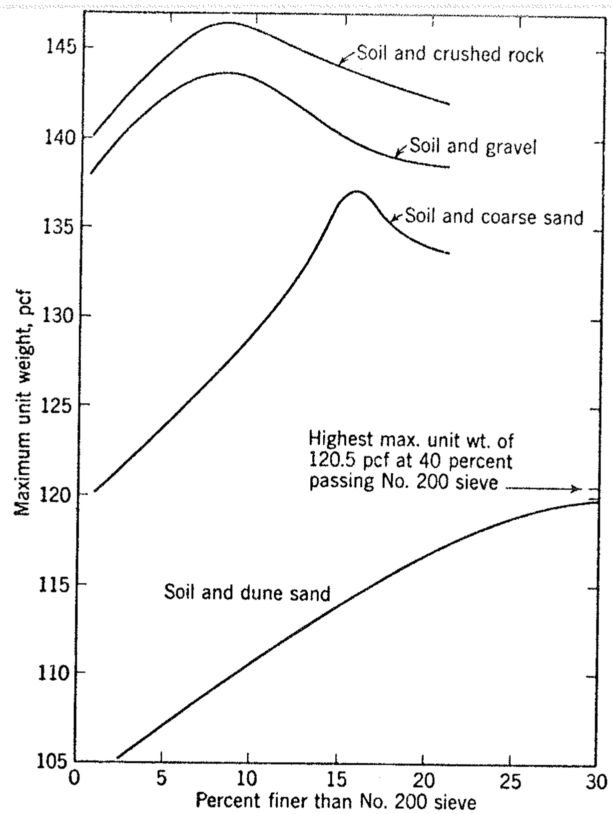
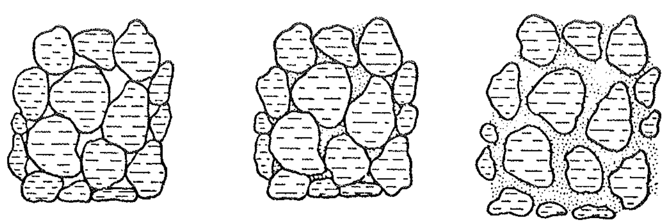


Fig. 7-4 Variation in maximum unit weight of aggregate-clayey soil mixtures with percent passing the No. 200 sieve. The silty clay soil admixed to aggregate had LL of 27 and PI of 6. (*After Yoder* [26].)

fines as "binder" to fill small voids. Uniformly graded sand, clays of high plasticity, and organic silts and clays typically respond poorly to compaction. Moisture-density curves for eight different soils compacted in the laboratory by standard methods are shown in Fig. 7-3. Note that not only does optimum moisture content decrease with increasing maximum dry density, but the sensitivity of the soil to increase in moisture content, as indicated by the shape of the compaction curve, changes also. Thus, the heavy clay and poorly graded sand, No. 7 and No. 8, are relatively insensitive, whereas the soils yielding higher densities show a greater response to changes in moisture content.

Gradation is critical to the compactibility of granular materials. As grain-size distribution becomes increasingly well graded, the dry density of the material will increase. Ideally, a soil in which gradation allows for a minimum of void space is best for compaction. This suggests that in order to achieve a high dry density, a granular soil such as a base-course aggregate should contain some fines. Research results given in Fig. 7-4 show this to be the case. The concept of improving density and stability of aggregate by providing fines to act as binder is important to the design of soil-aggregate blend used for pavement base. The effect of binder on the properties of compacted aggregate is summarized in Fig. 7-5.

The dry unit weight of fine-grained soils may be increased by additions of aggregate as shown in Fig. 7-6. This effect continues for increasing amounts of coarse aggregate as long as the aggregate particles do not interfere with the compaction of the soil mortar, the matrix of fine-grained soil. Individual aggregate particles, which are more dense than the compacted mortar, simply displace fine-grained soil. A point is reached, however, at which the percentage of coarse aggregate is sufficiently high for individual aggregate particles to be in contact and prevent good compaction of the smaller-sized material. The result is a drop in dry unit



(*a*) Aggregate with no fines	(*b*) Aggregate with sufficient fines for maximum density	(*c*) Aggregate with great amount of fines
Grain-to-grain contact	Grain-to-grain contact with increased resistance against deformation	Grain-to-grain contact destroyed, aggregate "floating" in soil
Variable density	Increased density	Decreased density
Pervious	Practically impervious	Practically impervious
Non-frost-susceptible	Frost-susceptible	Frost-susceptible
High stability if confined, low if unconfined	High stability in confined or unconfined conditions	Low stability
Not affected by adverse water condition	Not affected by adverse water condition	Greatly affected by adverse water condition
Very difficult to compact	Moderately difficult to compact	Not difficult to compact

Fig. 7-5 Physical states of soil-aggregate mixtures. (*After Yoder* [26].)

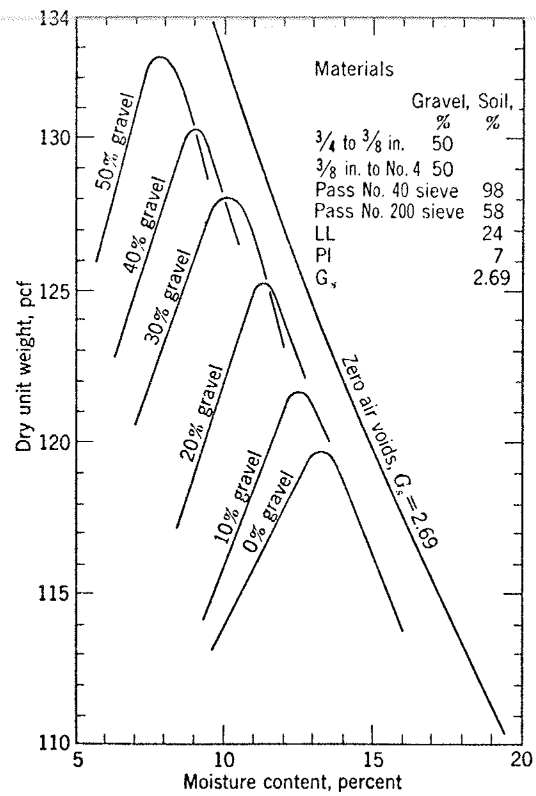


Fig. 7-6 Effect of gravel content on moisture-content–unit-weight relationship. Compaction according to AASHO T 99. (*After Zeigler* [29].)

weight at high aggregate contents. The point at which this drop occurs depends on the size and gradation of the aggregate and the gradation and plasticity properties of the soil. As the fine fraction becomes finer and more plastic, the drop occurs at higher aggregate contents.

The effect of gravel content on compaction density is important to applying the results of laboratory compaction to the field. For laboratory work, gravel is generally removed with a No. 4 sieve before determining the compaction characteristics of the soil. When field specifications are based on the laboratory results, the influence of coarse particles on unit weight must be recognized. This may be done by adjusting the laboratory apparatus or procedure so that the gravel is included in the soil tested, by computing the effect of gravel on the basis of theory and experimental work using one of the many methods available [8], by a combination of these methods, or by establishing the compaction requirement in the field during construction, which is usually the most reliable procedure.

Relatively free-draining, poorly graded granular materials having virtually no fines may exhibit no normal relationship between optimum moisture content and dry unit weight or may show an increase in dry density with increase in water content to a saturated condition on compaction. In addition, they may respond poorly to the impact type of compaction employed in laboratory tests. This is illustrated by the poorly graded sand in Fig. 7-3. If such material is used as a pavement component or pavement foundation, there is the danger of postconstruction compaction from repetitive wheel loads early in the life of the pavement, resulting in excessive pavement deformation. Indeed, with all granular materials, the primary objective of compaction during placement is to prevent such wheel-load compaction. Clean sands respond well to compaction under vibratory loads in the presence of large amounts of water; thus, where they have been placed simply with compaction by rolling at a moderate water content, in-service densification may be a serious problem. Vibratory compaction during placement, with a vibratory roller or sled, has been found to be effective. In some situations, such as with fills of substantial thickness, a combination of vibration and flooding has been used in a process called *vibroflotation*. In any case,

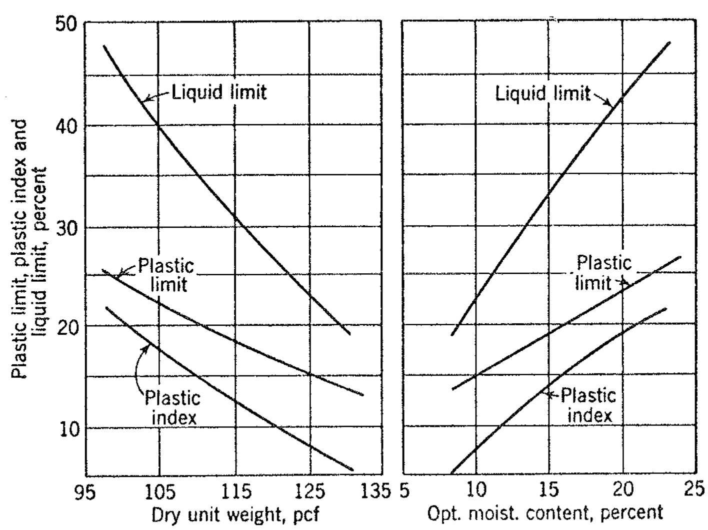


Fig. 7-7 Average relationships between plastic properties and maximum dry unit weight and optimum moisture content for 1,367 Ohio soils. (*After Woods and Litehiser* [25].)

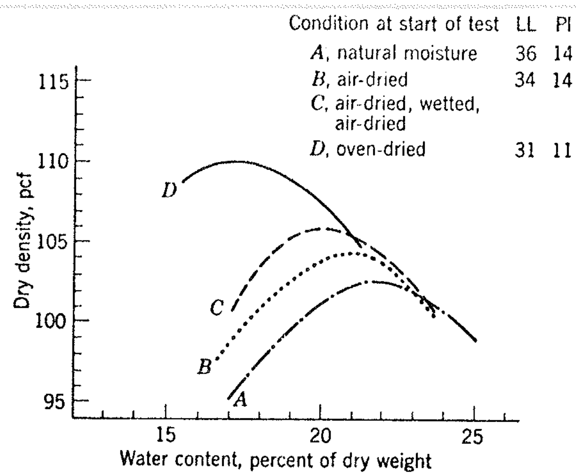


Fig. 7-8 Effect of drying and wetting sequence on compaction test data. (*After Holtz* [6].)

the degree of compaction of such materials is best thought of in terms of relative density, as defined in Chap. 2, rather than in terms of achieving
32 in the field the same level of density obtained by standard laboratory-compaction testing.

The compaction properties of fine-grained soils are largely governed by plasticity characteristics. As shown in Fig. 7-7, an increase in liquid limit and plasticity index is generally accompanied by an increase in optimum moisture content and a decrease in dry unit weight. Such a correlation is to be expected, since both liquid limit and compactibility depend on properties of the water adjacent to the soil particles. The less tightly adsorbed or highly oriented the water, the more the freedom of movement of water and solids. Important changes in plasticity properties by virtue of drying lead to changes in compaction properties. This is illustrated by compaction test data in Fig. 7-8.

Some of the general compaction characteristics of soils according to their unified soil classification system class and their value as base, subgrade, and embankment material are given in Table 7-1. Table 7-2 relates the anticipated embankment performance of soils to their classification according to the AASHO system.

7-3 COMPACTIVE EFFORT

As soil compaction proceeds, dry unit weight increases. The increase is rapid at first, but it becomes less with each additional application of

Table 7-1 Compaction characteristics and ratings of unified soil classification classes for soil construction*

Class	*Compaction characteristics*	*Maximum dry density standard AASHO, pcf*	*Compressibility and expansion*	*Value as embankment material*	*Value as subgrade material*	*Value as base course*
GW	Good: tractor, rubber-tired, steel wheel, or vibratory roller	125–135	Almost none	Very stable	Excellent	Good
GP	Good: tractor, rubber-tired, steel wheel, or vibratory roller	115–125	Almost none	Reasonably stable	Excellent to good	Poor to fair
GM	Good: rubber-tired or light sheepsfoot roller	120–135	Slight	Reasonably stable	Excellent to good	Fair to poor
GC	Good to fair: rubber-tired or sheepsfoot roller	115–130	Slight	Reasonably stable	Good	Good to fair
SW	Good: tractor, rubber-tired, or vibratory roller	110–130	Almost none	Very stable	Good	Fair to poor
SP	Good: tractor, rubber-tired, or vibratory roller	100–120	Almost none	Reasonably stable when dense	Good to fair	Poor
SM	Good: rubber-tired or sheepsfoot roller	110–125	Slight	Reasonably stable when dense	Good to fair	Poor
SC	Good to fair: rubber-tired or sheepsfoot roller	105–125	Slight to medium	Reasonably stable	Good to fair	Fair to poor
ML	Good to poor: rubber-tired or sheepsfoot roller	95–120	Slight to medium	Poor stability, high density required	Fair to poor	Not suitable
CL	Good to fair: sheepsfoot or rubber-tired roller	95–120	Medium	Good stability	Fair to poor	Not suitable
OL	Fair to poor: sheepsfoot or rubber-tired roller	80–100	Medium to high	Unstable, should not be used	Poor	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70–95	High	Poor stability, should not be used	Poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80–105	Very high	Fair stability, may soften on expansion	Poor to very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65–100	High	Unstable, should not be used	Very poor	Not suitable
PT	Not suitable		Very high	Should not be used	Not suitable	Not suitable

* Adapted from U.S. Army Corps of Engineers [23].

Table 7-2 General guide to selection of soils on basis of anticipated embankment performance*

HRB classification	*Visual description*	*Maximum dry weight range, pcf*	*Optimum moisture range, %*	*Anticipated embankment performance*
A-1-a A-1-b	Granular material	115–142	7–15	Good to excellent
A-2-4 A-2-5 A-2-6 A-2-7	Granular material with soil	110–135	9–18	Fair to excellent
A-3	Fine sand and sand	110–115	9–15	Fair to good
A-4	Sandy silts and silts	95–130	10–20	Poor to good
A-5	Elastic silts and clays	85–100	20–35	Unsatisfactory
A-6	Silt-clay	95–120	10–30	Poor to good
A-7-5	Elastic silty clay	85–100	20–35	Unsatisfactory
A-7-6	Clay	90–115	15–30	Poor to fair

* After Gregg [2].

energy until finally it is negligible. The curves in Fig. 7-9 express, for field compaction with pneumatic-tired rollers, dry unit weight vs. number of roller passes. Such curves are known as *growth curves*. Note that there is a point in a growth curve beyond which additional rolling does little to increase compaction. This point is reached earlier with the heavy roller than with the light roller. Also, the heavy roller results in a greater dry unit weight throughout. In all cases, the soils were compacted at near-optimum moisture content, but this optimum was somewhat less for the heavy rolling than for the light rolling. The conclusion to be drawn, and it has been verified with laboratory work, is that an important factor in the effectiveness of compaction is the amount of energy applied with each application of pressure. The greater the number of applications required to apply a certain amount of compaction energy, the smaller the resulting dry density is likely to be. With the total applied energy held constant, dry density decreases with increases in the number of work applications used to achieve that level of energy input.

The effectiveness of field compaction depends on the type of compaction equipment employed, which governs the manner in which compactive effort is applied and the magnitude of compactive effort. Numerous studies have been made on the results obtained using various types

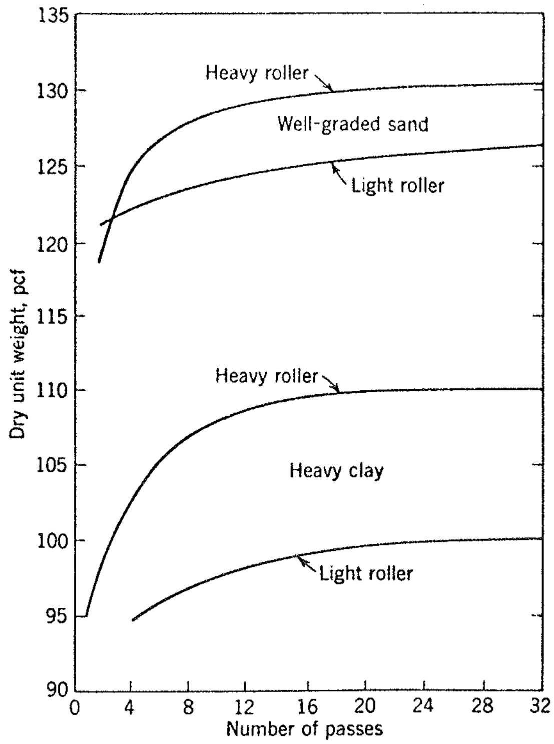
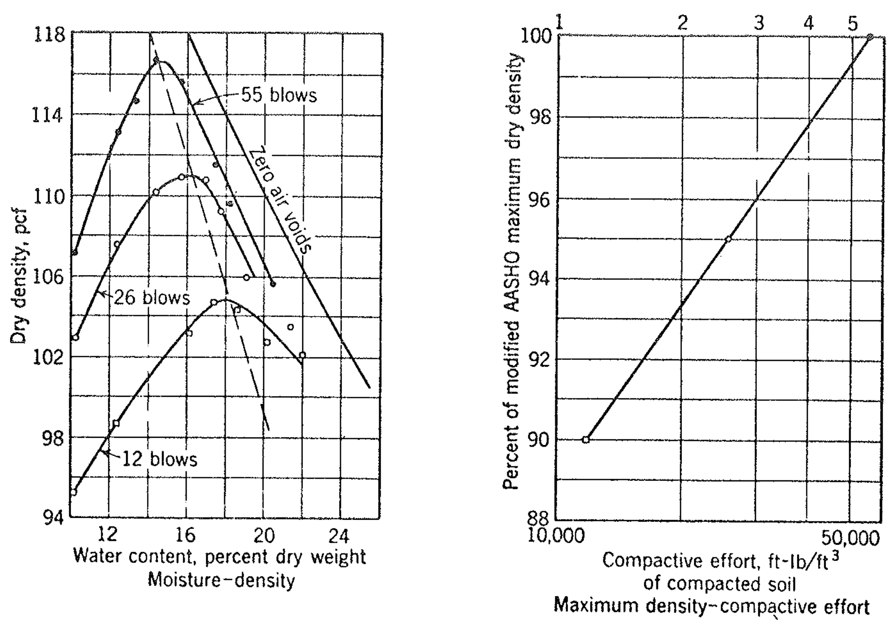


Fig. 7-9 Typical growth curves. These growth curves were obtained for two soils compacted with heavy and light rollers. The heavy roller compacted a 12-in. loose lift with a wheel load of 22,400 lb and tire pressure of 140 psi. The light roller compacted a 9-in. loose lift with a wheel load of 2,985 lb and tire pressure of 36 psi. The dry unit weight is for the upper 6 in. of compacted soil. (*Redrawn after Lewis* [12].)

of compactors, including smooth-wheeled, rubber-tired, sheepsfoot, and vibratory rollers of various weights. It can be concluded that for granular soils, vibratory compaction with a vibrating sled, baseplate, or roller is superior in attaining high unit weights and in compacting to relatively great depths. This is especially true of cohesionless materials and becomes less so as the soil becomes more cohesive. Beyond this conclusion, it is difficult to generalize. There is no single compactor type best suited for all soil types in all cases. For any particular job, the most suitable compactor is determined by such factors as desired density, lift thickness, and uniformity of compaction.

The unit weight of a compacted lift will decrease from the surface downward in proportion to the decrease in pressure intensity. Pressure intensity from a surface load on soil decreases rapidly downward, especially in the more plastic materials. Thus, to obtain appropriate compaction of a thick lift, a compactor that yields a high-intensity surface load must be employed. This, however, has the disadvantage of causing excessive sinkage, rutting, and pushing of soil ahead of the wheels. In some cases, considerable added power may be required to complete the first pass or it may be necessary to employ precompaction with lighter equipment. Obviously, such procedures greatly detract from the potential economy of compacting in few thick lifts with heavy equipment rather than a number of thin lifts with lighter equipment.

In the laboratory, one may easily study the influence of compactive



	Layers	Blows per layer	Weight of hammer, lb	Drop, in.	Compaction effort,* ft-lb/ft³
•	5	55	10	18	56,000 (mod. AASHO)
∘	5	26	10	18	26,400
▫	5	12	10	18	12,200 (std. AASHO)

* Tests made on lean clay from Waterways Experiment Station field-compaction studies. Liquid limit 36 percent, plastic limit 13 percent, class ML-CL.

Fig. 7-10 Typical plots of compaction results. (*After McRae* [13].)

effort on the moisture-density curve. It is customary to compact soil in layers in a cylindrical mold using a drop hammer. If the number of hammer blows applied per layer is varied to establish compaction curves for differing levels of compactive effort, results similar to those in Fig. 7-10 will be obtained. With increasing compaction energy, not only does maximum dry density increase, but optimum moisture content decreases. When dry unit weight is plotted against logarithm of compactive effort, a straight-line relationship is given. The slope of the line is generally steeper for the more plastic, less easily compacted soils.

It is important to note that a certain level of compaction in terms of dry density may be achieved at any of several compaction water contents by adjusting compactive effort. Thus, for the soil shown in Fig. 7-10, a dry density of 104 pcf may be achieved with a relatively low compactive effort and a water content of about 17 percent. If the compactive effort is doubled, this density might be achieved with a water content as low as 11 percent. This suggests that one might compensate for a moisture deficiency in the field by increasing roller weight. Unfortunately, for fine-grained soils the levels of both compactive effort and moisture content have an influence on soil structure. As a result, such important properties as strength and swell potential will not be the same simply because dry unit weight is the same. It may well be that the soil compacted with the heavy roller and moisture deficiency will show considerable instability during the life of the pavement structure. 37

PROPERTIES OF COMPACTED FINE-GRAINED SOILS

The purpose of compaction is to provide soil with stability. The stability required of compacted subgrade soils differs in its nature for granular and fine-grained soils. For granular soils, resistance to densification or "shifting" under imposed wheel loads is of primary importance. Such resistance can be achieved with high relative density, which in turn depends on good compaction of properly graded materials. For fine-grained, cohesive soils, resistance to deformation under imposed loads and volume change attending change in environmental conditions are both important. Moreover, both are more difficult to attain with certainty than in the case of sands and gravels. The cohesive soil must not only be compacted to a required degree of stiffness, but this deformation resistance must be maintained at an acceptable level during the life of the pavement. Thus, weakening by saturation with water during wet periods or by capillarity and frost action, and the volume change that this may cause, must be understood and accounted for. To this end, the stability and structure of compacted clays is discussed below. A discussion of fundamental aspects of soil structure and swelling potential is found in Chaps. 2 and 3 and information on water adsorption and frost action and their influence on soil stability is found in Chap. 5.

7-4 STABILITY INDICES

For fine-grained subgrade soils, an index of stability is resistance to excessive deformation under load or pressure. The definition of "excessive deformation" varies widely according to the stability problem at hand and the approach selected for its solution. For compacted highway subgrade, a variety of methods have been developed for measuring relative stability. Most of these involve molding laboratory samples of the field soil in a manner and under conditions meant to simulate to some extent field conditions, testing for deformation or shearing resistance in a simple, direct manner, and interpreting the test results in the light of past correlations with the performance of soils underlying pavements. It is not presumed that any of the simple stability tests is a test for such fundamental parameters of soil strength as angle of shearing resistance or cohesion. The proper and complete characterization of soil strength as such is a difficult and complex art, both in execution and in interpretation of results. However, the simple laboratory stability tests have as advantages ease of performance, applicability to a wide variety of soil materials, and extensive correlation with research on soil stability and soil behavior beneath pavements. Thus, they have special value for the evaluation of highway subgrade soils.

Of the large number of stability tests that have been applied to highway subgrade, two are selected for this discussion, California bearing ratio (CBR) and unconfined compressive strength (q_u). The selection is based more on the popularity of the tests than on their ability to properly characterize stability. CBR is probably the most widely used index of stability. It is popular among highway engineers for use in pavement design, is used in research on subgrade stability, and is referenced in a large volume of literature. The unconfined compression test is very different in its nature and use. It was not designed to evaluate the stability of subgrade soils, but is nonetheless commonly employed to compare them. It is often used to evaluate the adequacy of soils stabilized with admixtures of such cementing agents as portland cement and lime.

In referring to the results of stability tests on laboratory-compacted soils, it should be borne in mind that they are limited in their significance by differences between laboratory and field compaction. This is particularly true for cohesive soils. For granular soils, when laboratory samples are prepared at relative densities close to the anticipated field densities, laboratory strength correlates well with field strength under comparable conditions. However, for fine-grained soils, strength is influenced not only by moisture content and relative density, but as well by soil structure, particularly at low strain levels and when the soil is at a moisture content somewhat wet of optimum. Structure, in turn, is greatly influenced by compaction procedure, and although attempts have been made to duplicate the nature of field compaction in the laboratory, results show that field behavior is simulated in the laboratory with only limited success.

California bearing ratio Several decades ago, as a result of an intensive study on failures in flexible pavements, the California Division of Highways devised a new method of flexible pavement design. Basic to the method is a test known as the California bearing ratio test, usually shortened to CBR. The test is designed to indicate the relative stability of soil that has been constructed with a particular density and water content and that has adjusted to its environment beneath the pavement. It provides for compacting soil in a cylindrical mold and soaking the sample for 4 days with an imposed load roughly equivalent to that which would be given by a prototype pavement (Fig. 7-11). The compaction simulates construction and the soaking simulates a water-content adjustment roughly equivalent to that which would occur if the water table were 2 ft below the base of the pavement. The amount of volume change is recorded during the soaking period, and soils with swell exceeding 3 percent were rated as poor for subgrades. The "strength" test is a penetration test, whereby a circular piston is forced into the soaked soil at a constant rate. A load-penetration curve is developed and this curve is compared to a standard curve obtained for crushed 39

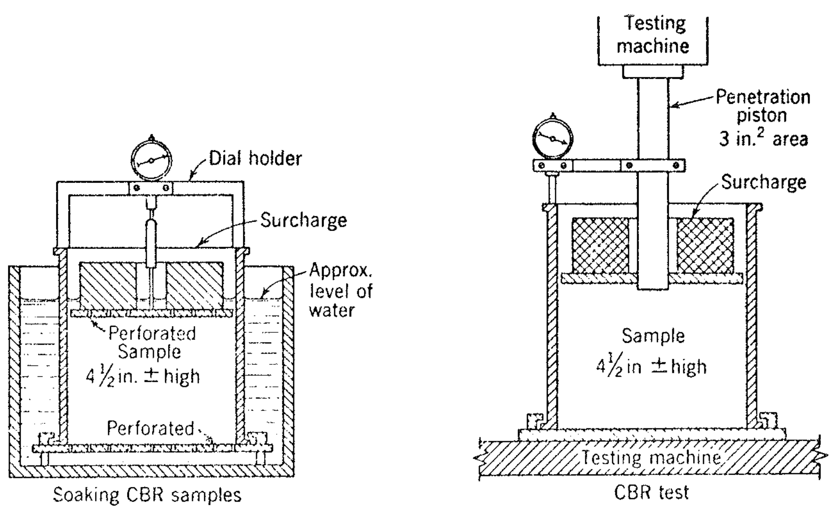


Fig. 7-11 The California bearing ratio test. (*After Walker, Yoder, Foster, and Johnson* [22].)

stone. For most cases, CBR is defined as the load at 0.1-in. penetration for the soil compared to the crushed rock and expressed as a percentage. Values range from 3 percent for poor subgrades to 80 percent or higher for good base courses.

The California Division of Highways now uses an alternate method of pavement design, but the Army Corps of Engineers modified and adopted the test and has conducted several valuable studies on its use for airfield construction. In addition, it is used by nearly a score of state highway departments. The test is now standardized in all particulars and carries the ASTM designation D 1883-67.

Comparison of CBR values with the results of more elaborate and exacting strength tests indicates that CBR stability values correlate well with the soil strength values at low to intermediate levels of strain. Thus, CBR emphasizes the low strength of soft soils and gives high stability readings for stiff materials, especially those stabilized with the aid of portland cement or lime. It is not uncommon to obtain erratic results with granular materials.

40 **Unconfined compressive strength** The resistance of soils to failure in shear under a compressive load is derived from two sources, cohesion and intergranular friction. Since strength from friction is a function of the pressure applied to the plane of shear, soil shear strength increases with increases in effective stress. It follows that placing a confining pressure on a sample will increase its strength to the degree that this confining pressure is effective stress. Values of cohesion change with void ratio, being higher for smaller void ratios. Hence, if a confining pressure is applied to a sample and this causes the soil to consolidate, the cohesion component of strength can be expected to increase as well. The unconfined compression test, abbreviated q_u (quick, unconfined), does not evaluate these changes in strength with applied stress. However, for materials in which the major component of strength is cohesion, and for which one might expect limited volume change owing to confinement under field conditions, the test is a valuable indication of minimum strength for the material. Moreover, for many soils there is a correlation between compressive strength and modulus of elasticity, stiffness, so that q_u evaluates as well the relative stiffness of the sample.

The unconfined compressive strength test is conducted not unlike the usual compression test on cylinders of concrete. A small cylindrical sample, 4 to 6 in. high and less than half as wide, is loaded from the top until failure occurs as shown by a drop in load resistance or, for plastic materials, by excessive strain. Readings taken at intervals during the application of load allow the construction of a stress-strain curve and definition of ultimate strength. For laboratory-compacted soils, samples of suitable size may be prepared using the "Harvard Miniature" compaction apparatus, which consists of a small cylindrical mold and a tamper with a spring-loaded plunger. Alternately, and often with better results, soil extruded from the standard laboratory-compaction mold may be quartered with a band saw and trimmed to size with a soil lathe. The soil may be soaked, cured, frozen and thawed, or subjected to any other treatment that might be helpful in simulating field conditions. For storage, samples can be protected from drying using alternate layers of paraffin and a wrap of impervious material.

The test has been standardized, ASTM D 1633-63, for use with soil-cement mixtures and is also effectively used to evaluate lime-stabilized clays as well as nonstabilized soils. The adequacy of the stabilized soil can be evaluated easily after appropriate curing, with or without soaking. Some highway departments require a particular level of q_u for such stabilized soils. Virginia, for example, seeks a minimum of 150 psi for lime-stabilized soils to be used as subbase.

7-5 STABILITY AND STRUCTURE OF COMPACTED CLAY

The stability of compacted clay depends not only on such factors as soil type and achieved density but is also highly dependent on the structure developed in the clay fabric. Thus, most of the following discussion concerns claylike soils rather than all fine-grained types. 41

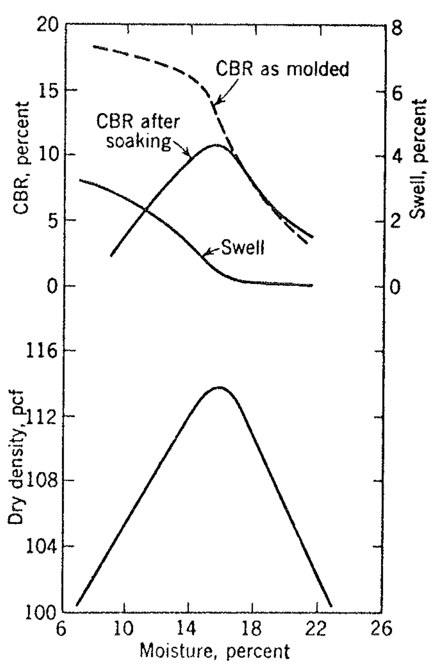


Fig. 7-12 Density and CBR for a typical silty clay (CL). (*After Yoder* [27].)

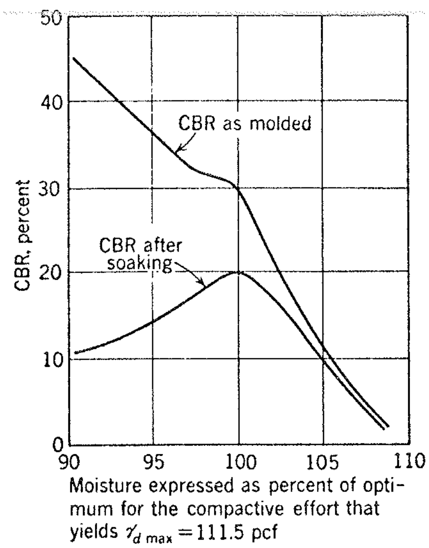


Fig. 7-13 Effect of soaking and compaction moisture content on CBR. (*Drawn from data by Turnbull and Foster* [19].)

A typical silty clay soil compacted with a constant compactive effort will yield a compaction curve such as that shown in Fig. 7-12. If stability, in terms of CBR and percent swell, is determined for samples molded at various points along the curve, it is found that "as-molded" CBR decreases with compaction moisture content. However, if CBR is determined for the same specimens after soaking for a 4-day period, a peak value roughly corresponding to optimum moisture content is noted. The samples compacted at low moisture contents show large amounts of swell during the soaking period, with percent swell decreasing with increasing molding moisture content until it becomes relatively constant for moisture contents greater than optimum. The data illustrate that clay compacted dry of optimum moisture content assumes a structure conducive to high strength and stiffness. However, this structure allows appreciable swell with exposure to water. The swell is disruptive and remolds the soil or separates particles to the extent that a large fraction of the initial strength is lost. Soils compacted wet of optimum, on the other hand, possess a structure more stable in the presence of water but less able to yield stability under load. Presumably, compaction near the optimum moisture content results in an intermediate form of structure that gives the best features of both conditions, strength and stiffness as molded, and resistance to stability loss and swell with soaking.

Similar data are given by soil compacted at different compactive efforts and moisture contents, but to a constant density. The curves in Fig. 7-13 are for the soil used to develop the compaction curves in Fig. 7-10. The 111-pcf density may be achieved at optimum moisture content, 16 percent, using the 26-blow compactive effort as determined by the curve in Fig. 7-10. This yields a soil with a soaked CBR of approximately 20 percent. However, it cannot be assumed that the 111-pcf density always results in this level of CBR, regardless of compaction moisture content. If the soil to be molded has a water content of 11 or 18 percent, it may be more convenient to achieve the 111-pcf density with the 55-blow effort than to change the moisture content prior to compaction. But the product of compaction will be the same in density only. Important differences in the structure produced in the soil will cause the soaked CBR to drop to a small fraction of the value for compaction at optimum moisture content. This is an important practical consideration. If pavement design is based on the soaked CBR obtained for compaction at optimum moisture content and maximum dry density, preparation of the pavement subgrade must be not only by compaction to the proper density, but by compaction at a proper moisture content as well.

An even more dramatic indication of the structure differences in clays compacted under different conditions is given by data on permeability. Structure is the most important single variable affecting the permeability of compacted clay. Clay compacted dry of optimum moisture content may have a permeability value two or three orders of magnitude more than when compacted wet of optimum. If the compactive effort is held constant, a sharp change in permeability usually occurs in the vicinity of optimum moisture content. If compaction density is held constant by varying compactive effort, the marked permeability decrease may occur at moisture contents considerably higher than optimum for the density achieved [14]. 43

If the structure of compacted clay is viewed in terms of particle orientation, which may be either random or oriented as discussed in Chap. 2, then degree of orientation is found to increase with compaction moisture content as shown in Fig. 7-14. Lambe [10, 11] has illustrated this important change in structure with the schematic diagram of Fig. 7-15 and described its effects on soil properties as given in Table 7-3. The orientation of particles at the beginning of the compaction process will generally be of the "flocculent" or randomly oriented structure common to nearly all clays found in natural soils. The process of compaction, a combination of compression and shear, works toward particle orientation. The degree to which particle alignment is achieved depends on the nature, effort, and water content of the compaction. Compaction moisture content is of great importance because it determines the ease with which the particles may shift or move relative to each other. Water is adsorbed on the particle surfaces and tends to separate them and decrease net interparticle attractive force. Accordingly, density is achieved more through particle realignment than through forcing particles into positions of closer proximity to each other. When particles are forced together in the presence of a water deficiency, a strong potential for swell is produced, as noted in Table 3-7. Adsorbed water cannot be accepted by the particles without their separation, and the high suction potential of the compacted clay induces considerable adsorption. Particle separation on swelling greatly weakens the structural framework of randomly oriented particles.

Complementing the particle-orientation view of structure in compacted clay is the concept of cluster or packet structure. For many natural clays, especially those that have undergone extensive drying and reworking or weathering in soil zones subjected to seasonal changes in water content, there is an aggregation of individual clay particles into visible clusters, as illustrated in Fig. 2-9. The compaction of such clay is governed by resistance to deformation and packing of the clusters, as is the final structure of the fabric. The resistance to cluster deformation can be expected to decrease with increasing molding moisture content, allowing a given cluster to adjust to the contours of adjacent clusters. Accordingly, increasing with molding moisture content are density and particle orientation within clusters, and decreasing is permeability, which strongly depends on the amount and continuity of intercluster pore space.

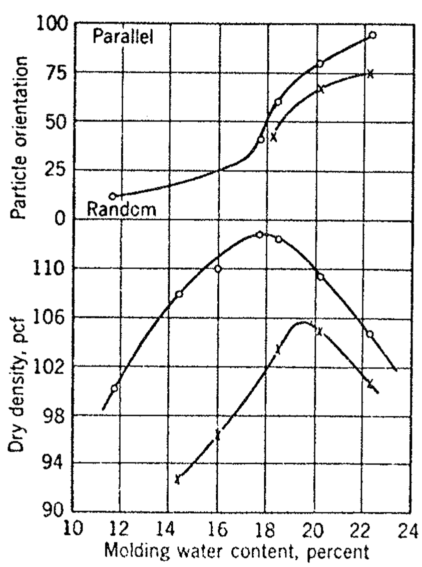


Fig. 7-14 Effect of molding water content on particle orientation for compacted Boston Blue Clay. (*From Lambe* [11].)

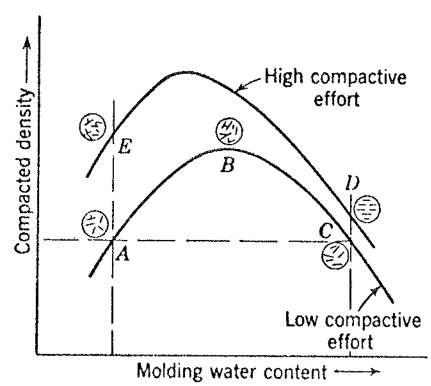


Fig. 7-15 Effects of compaction on structure. (*From Lambe* [11].)

Table 7-3 Comparison of dry-of-optimum with wet-of-optimum compaction*

Property	*Comparison*
1. Structure	
(*a*) Particle arrangement	Dry side more random
(*b*) Water deficiency	Dry side more deficient, therefore imbibe more water, swell more, have lower pore pressure
(*c*) Permanence	Dry-side structure sensitive to change
2. Permeability	
(*a*) Magnitude	Dry side more permeable
(*b*) Permanence	Dry side permeability reduced much more by permeation
3. Compressibility	
(*a*) Magnitude	Wet side more compressible in low pressure range, dry side in high pressure range
(*b*) Rate	Dry side consolidates more rapidly
(*c*) Rebound	Wet-side rebound per compression greater
4. Strength	
(*a*) As molded	
(1) Undrained	Dry side much higher
(2) Drained	Dry side somewhat higher
(*b*) After saturation	
(1) Undrained	Dry side somewhat higher if swelling prevented; wet side can be higher if swelling permitted
(2) Drained	Dry side about the same or slightly greater
(*c*) Pore-water pressure at failure	Wet side higher
(*d*) Stress-strain modulus	Dry side much greater
(*e*) Sensitivity	Dry side more apt to be sensitive

* From Lambe [10].

As optimum moisture content is approached and exceeded, clusters soften to the point of becoming largely destroyed by the compaction energy so that density becomes controlled more by the separation and orientation of individual particles than by cluster packing. The dramatic decrease in permeability as optimum moisture content is exceeded probably reflects this process. The cluster-structure concept is not only especially applicable to explaining the hydraulic properties of compacted clays, but applies as well to a mechanistic picture of the behavior of clays treated with chemical stabilizing agents that cause strong flocculation and cementation.

Method of compaction influences clay structure in that it governs particle orientation by shear deformation. Shear strains tend to produce a parallel arrangement of soil particles. Thus, for soils compacted at or above optimum moisture content, for which interparticle attractive forces are not so great as to ensure flocculation under all compaction con-

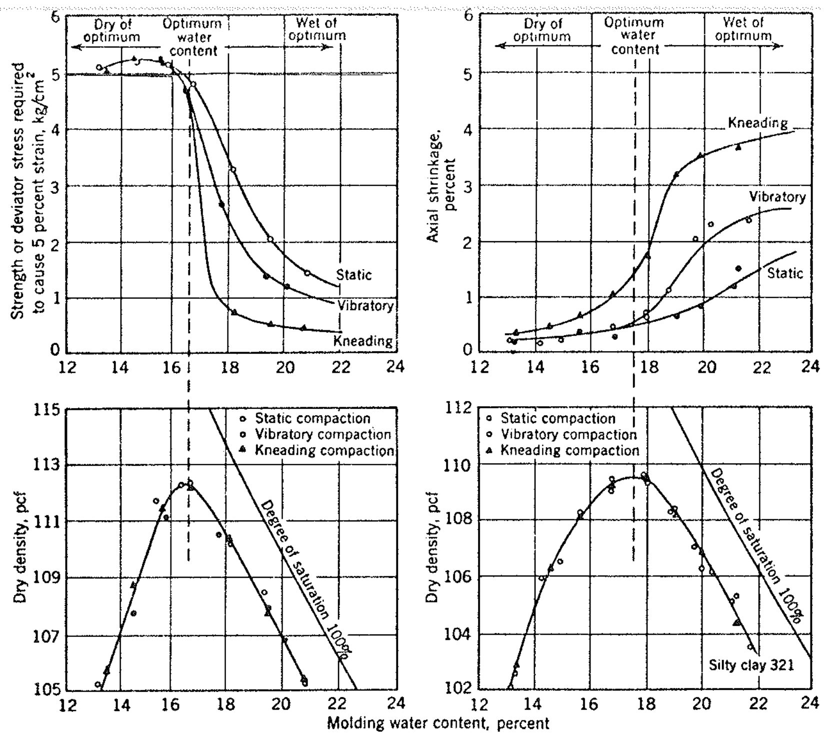


Fig. 7-16 Influence of method of compaction on strength and shrinkage of silty clay. (*After Seed and Chan* [18].)

ditions, the methods of compaction inducing the greater shear strains will produce the greater degree of particle orientation. The result is lower strengths and less swelling than for compaction yielding little shear strains. This effect is illustrated in Figs. 3-11 and 7-16. Seed and Chan [18] found that for laboratory samples compacted wet of optimum to any given water content and density, particle orientation tends to increase and strength to decrease in the following order of laboratory-compaction methods: static, vibratory, impact, and kneading. The implications for field-compacted subgrades are clear; one may expect somewhat different behavior from clays compacted in the field by different types of equipment, even though moisture content, dry density, and soil type remain the same.

7-6 THE FAMILY-OF-CURVES METHOD FOR STUDY OF STABILITY

Because of the importance to the behavior of compacted clay of both molding moisture content and dry density, it is of considerable advan-

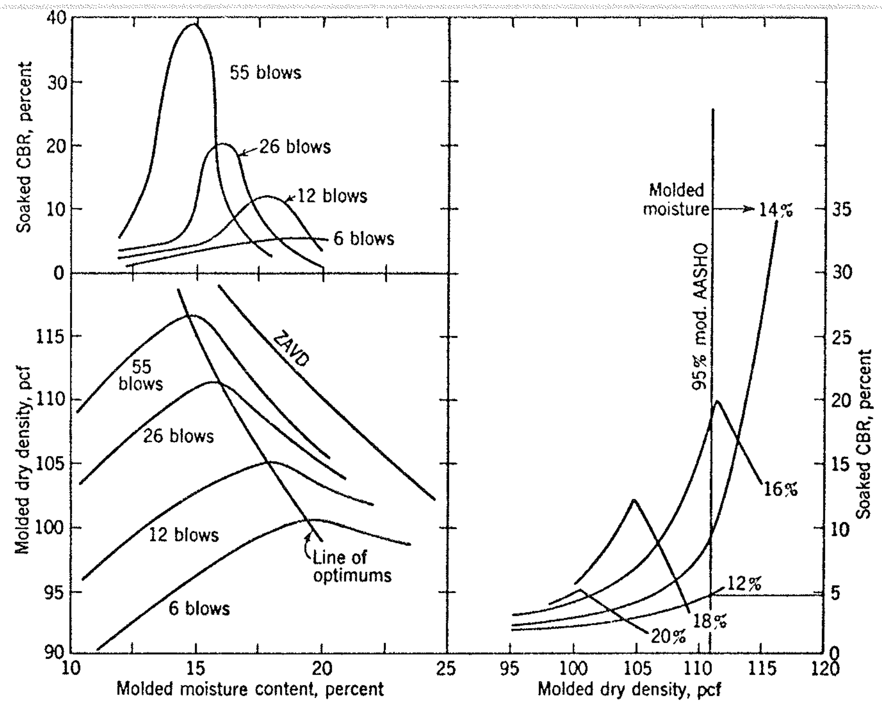


Fig. 7-17 Molding moisture content, dry density, and soaked CBR relationships for a lean clay (ML-CL) compacted at various compactive efforts. Blows refer to number of blows per compacted layer, as in Fig. 7-8. (*From Turnbull and Foster* [19].)

tage to be able to study the influence on stability of these factors as separate variables. This can be done utilizing the *family-of-curves method* of compaction-data analysis developed by Turnbull and McRae [20]. By this method the usual moisture-density curves are developed for the soil, but for three levels of compactive effort as shown in Fig. 7-10. It is important that the efforts used cover the practical working range of water content and density to be expected in the field. From tests on the stability of soil specimens from several points on each compaction curve, stability vs. water content of compaction for each compactive effort is obtained as shown in Fig. 7-17. Then, CBR value vs. dry density may be plotted for each moisture content of interest.

The curves in Fig. 7-17 show the effect of molding moisture content and compacted density on the stability of the soil. Thus, if a CBR value of 10 is sought as a minimum and if the soil to be compacted has a moisture content of 16 percent, a compacted dry density of at least 108 pcf is called for. Conversely, if it is determined that a density of 105 pcf was achieved in the field at a moisture content of 18 percent, a CBR value of 12 might be expected. The family of curves permits an evaluation of the extent to which control of water content or dry density or both is necessary in compacting a given soil in the field to achieve a given degree of stability.

A common practical use of such curves is easily given. In Fig. 7-17, the 55-blow curve represents the "modified AASHO" compactive effort; thus, maximum modified density is about 117 pcf. It is common in airfield construction to specify 95 percent "modified AASHO" density for the subgrade compaction requirement, in this case 111 pcf. This density can then be plotted on Fig. 7-17 as shown and at a glance the engineer can make recommendations as to allowable range of compaction moisture contents in order to achieve a certain minimum CBR. For example, if moisture were controlled between 12 and 16 percent (4 percent range in moisture being about a practical minimum in the field), then the engineer would be assured of having a minimum CBR of about 4 when the subgrade became saturated. Because of the nature of this soil, at these high densities, control of moisture is critical because "soaked" stability is largely lost if compaction moisture goes to 18 percent.

The family of curves in Fig. 7-17 suggests that it is possible to "overcompact" or "overstress" a soil. Note that for moisture contents of 16, 18, and 20 percent, there is a density beyond which stability decreases. Thus, if the moisture content of the soil is 18 percent and a compacted density of 105 pcf is anticipated for pavement-design purposes, compaction to 110 pcf by using a high compactive effort may lead to serious problems. This phenomenon results from inducing high pore pressures and swelling potential in the soil, which ultimately lead to decreased stability.

In establishing field-compaction requirements on the basis of a single laboratory moisture-density curve, it is tacitly assumed that the field-compaction curve duplicates the laboratory curve at least in the region of optimum moisture content. Using the family of curves method for compaction specification, greater or lesser compactive effort is allowed as long as a satisfactory product is obtained. However, there is still an important hidden assumption: that the "line of optimums," as shown in Fig. 7-17, coincides for both the laboratory- and the field-compacted soil. This often is not the case. The relative position of the lines in relation to the ZAVD curve varies for various field- and laboratory-compaction methods and for various soil types. Thus, the success of predicting field properties of compacted clays from laboratory-compaction studies is limited.

FIELD COMPACTION PROCEDURES AND REQUIREMENTS

Field engineering for sound earthwork construction during the normal cut-and-fill operations of highway construction is one of the most important and difficult tasks of the engineer. Variability of material and environmental conditions on the job, and of equipment from job to job, requires the continuous exercise of engineering judgment at the construction site. Special problems in working conditions and materials are almost as much the rule as the exception. At times, the solution to such problems must be based on visual inspection alone. At most other times, the available test results on quality require considerable interpretation. It is not a discredit to state highway departments that they differ in their specifications and procedures for earthwork construction. These differences are required to properly handle different materials under varying environmental conditions. Moreover, local experience has been applied to evolve many local practices for handling specific problems. Hence, national standards are neither extant nor desirable.

The compaction procedures and requirements of the various state highway departments have been reviewed by Wahls, Fisher, and Langfelder in a report to the Bureau of Public Roads [21]. Their thorough discussion concludes with valuable recommendations for current practice. Many of these and some of their observations have been incorporated into the material that follows. However, students with special interest should not neglect study of the report proper to supplement this brief discussion with the many significant details that could not be included here.

7-7 COMPACTION METHODS AND EQUIPMENT

Material to be compacted may be the subgrade soil in cut sections, embankment soil and subgrade in fill sections obtained either from cuts or borrow pits, and pavement undercourses such as base and subbase. Natural subgrade is generally compacted to a depth of at least 6 in. Fill is usually distributed into uniform layers no more than 8-in. thick with a bulldozer, road grader, or scraper. When lift thickness is specified, it is usually in terms of the uncompacted material. It is found that for heavy clays, thicknesses greater than 6 in. may be difficult to work. Compaction usually results in a layer depth of from two-thirds to three-fourths the depth of the loose spread. For base-course materials, compacted thickness is usually specified, with a maximum of 6 in. commonly allowed.

When the soil to be compacted is too wet, drying is enhanced by aeration through manipulation with discing, harrows, or cultivators. For heavy clays, satisfactory drying may be impractical and the soil must either be compacted wet of optimum moisture content, placed in positions where compaction is of little consequence, treated with additives such as lime, or wasted.

When additional water is required, moisture is usually added after the soil has been spread loosely in place using a tank truck and sprinkling system and auxiliary equipment such as discs, harrows, or mixers. Distribution is carried out according to the characteristics of the soil, as indicated in Table 7-4. If the soil is obtained from a borrow pit, better moisture control is obtained when the water is added at the pit. In either case, close attention should be given the procedures and equipment used; proper mixing of soil and water is critical.

Compaction is usually accomplished with a maximum of 6 to 10 complete coverages of the compaction equipment. An increasing num-

Table 7-4 Generalized correlation of soil classification and equipment and methods for incorporating water prior to compaction*

Type of soil	*Equipment and methods*
Heavy clays	Difficult to work and to incorporate water uniformly. Best results usually obtained by sprinkling followed by mixing on grade. Heavy disc harrows are needed to break dry clods and to aid in cutting in water, followed by heavy-duty cultivators and rotary speed mixers. Lift thickness in excess of 6-in. loose measure is difficult to work. Time is needed to obtain uniform moisture distribution
Medium clayey soils	Can be worked in pit or on grade as convenience and water-hauling conditions dictate. Best results are obtained by sprinkling followed by mixing with cultivators and rotary speed mixers. Can be mixed in lifts up to 8 in. or more loose depth
Friable silty and sandy soils	These soils take water readily. They can often be handled economically by diking and ponding or cutting contour furrows in pit and flooding until the desired depth of moisture penetration has taken place. That method requires watering a few days to 2 or 3 weeks in advance of rolling (depending on the texture and compactness of the soils) to obtain uniform moisture distribution. These soils can also be handled by sprinkling and mixing, either in-put or on-grade, and require relatively little mixing. Mixing can be done with cultivators and rotary speed mixers to depths of 8 to 10 in. or more without difficulty

* From *Highway Res. Bd. Bull.* [3].

ber of passes beyond this amount usually proves uneconomical. As illustrated in Fig. 7-9, the required number of passes to achieve a particular density is strongly a function of the weight of the roller. Heavy rollers achieve adequate compaction with fewer number of coverages than do light rollers. However, as mentioned in Sec. 7-3, sinkage, rutting, and shoving of material become a problem if roller weight is too large.

Because it is in the contractor's best interests to compact efficiently, enforcing minimum state standards on compaction equipment is seldom a problem and overstressing material during routine compaction seldom occurs. However, loads from very heavy hauling and paving equipment may overstress and fail some embankments, especially those constructed of silty materials. Accordingly, maximum allowable wheel loads for such equipment should be specified.

The effectiveness of compaction varies with type of compaction equipment and such equipment parameters as size and weight of roller. The major equipment types are smooth-wheeled rollers, pneumatic-tired and sheepsfoot rollers, and vibratory compactors, which may consist of a vibrating plate or roller. For cohesive soils the kneading action provided by sheepsfoot and pneumatic-tired rollers works well. Silty soils may also be compacted efficiently with sheepsfoot and pneumatic-wheeled rollers or smooth-wheeled rollers may be used. Generally, smooth-wheeled rollers, which compact primarily through the action of static weight, are most used for finishing a compaction surface at the end of a day's operations. For compacting granular soils, vibratory compactors give excellent results. The smooth-wheeled paving roller is sometimes used also, and pneumatic rollers will give satisfactory results if the granular material contains some fines. Most states specify some minimum equipment standards for at least one type of equipment, usually pneumatic-tired or smooth-wheeled rollers, and for construction of at least one component of the pavement section, commonly the base course. Further, most specifications provide for compaction equipment to be approved by the engineer.

Rolling pressures are applied for a relatively short time, depending on roller speed, usually 2 to 5 mph. The sheepsfoot roller is a steel drum with projecting lugs or feet that apply a high pressure, normally more than 200 psi. Efficient compaction occurs when there is a gradual "walkout" of the roller lugs with successive coverages. This has led to the concept that the sheepsfoot compacts from the bottom up, but actual field measurements suggest that this common idea is erroneous. The pneumatic tire is an excellent compactor for a variety of soil and construction conditions. These rollers may be either single-wheeled or multiple-wheeled types with tire inflation pressures of 70 to 90 psi and gross weights in excess of 10 tons. They apply moderate pressure to a relatively wide area so that the pressure can be supported by the subgrade without failure. Smooth-wheeled rollers normally weigh about 10 tons and apply a pressure of about 300 to 325 psi. They are not as generally satisfactory for earthwork as other rollers unless the layers to be compacted are thin and well leveled; however, they do tend to bridge low spots and produce a smoother finished grade.

For vibrating compactors, vibration is produced by offset cam arrangements that supply frequencies between 1,500 and 2,000 cpm. For granular materials, thicker lift can be compacted with vibratory equipment than with conventional rollers. Also, there is indication that less degradation occurs in the softer types of aggregate such as limestone. Degradation is a process whereby excessive amounts of fines are produced by the breakdown of aggregate under repeated load. For aggregates of relatively soft stone, it can be a serious problem during base-course construction.

An excellent general discussion of compaction equipment is contained in Refs. 7 and 21. In addition, data dealing with performance characteristics of the wide variety of available equipment may be obtained from equipment manufacturers.

7-8 COMPACTION REQUIREMENTS

A compaction requirement may specify the procedure by which compaction is to be accomplished, the quality of the compacted material, or a combination of procedure and end result. "Procedural specifications" may include moisture control, lift thickness, type and size of equipment, and number of coverages or some visual criterion such as the walkout of a sheepsfoot roller. "End-result specifications" are in terms of dry density achieved, usually expressed as a percentage of laboratory maximum dry density, and they may include some procedural requirements such as compaction moisture content and lift thickness.

Procedural requirements have the advantage of not requiring control testing of the finished product. However, the variability of soils makes difficult the establishment of general procedures that will be satisfactory throughout the job. Thus, reliance on equipment and procedural specifications is most often limited to closely controlled raw materials such as in base courses, which normally meet certain gradation and quality requirements. Procedural requirements may be established on the basis of a test section in the field when sufficient quantities of a particular soil exist to justify its construction. End-result specifications take into account material variability by expressing requirements in terms of those that have been achieved for the material with laboratory or field testing. Still, a number of problems are introduced by the required control testing.

Density requirements Density control specifications are invariably applied to embankments, almost always relied on for subgrades, and usually applied to base and subbase construction. For subgrades and embankments, 95 percent of the maximum dry density achieved in the laboratory with standard AASHO compaction is the usual field requirement. Subgrade requirements are always at least equal to and are sometimes greater than requirements for embankments. Base-course requirements are normally more stringent. A greater percentage of the standard AASHO density, often 100 percent, or a percentage of the modified AASHO density, usually 95 percent, may be required. However, the AASHO impact tests can be applied to the coarse materials of bases only with considerable difficulty, and so it is not uncommon for alternate tests such as laboratory vibration or procedural requirements to be used.

Occasionally, variations in compaction requirements are introduced with changes in soil type or maximum dry density. Such changes are not justified on a rational basis, but have arisen primarily from local experiences and from the need to deal with such problem soils as elastic silts or swelling clays.

Moisture-content requirements It is less common to apply quantitative moisture requirements to compaction than to specify density. Often, moisture control is left to the judgment of the engineer on the job. Experienced engineers and technicians can judge the relative moisture conditions of soil by feel and by visual examination. Also, moisture control is not critical for some soils such as granular materials. However, it is best to have rigid control procedures in instructions to field personnel and the contractor. These are of value in any legal controversy and lift a large burden of responsibility from the inspector or project engineer. In exceptional cases, the specific control procedures may be waived or compromises may be indicated in the specifications.

The important influence of molding moisture content on the physical properties of compacted fine-grained soils has been emphasized in previous discussion. Specifications should require compaction in the vicinity of optimum for the field-compaction conditions. Unfortunately, this will in general differ from the laboratory optimum, so that when standard laboratory tests are used to obtain quantitative specifications, a large allowance must be made for the fact that with moisture-sensitive soils such as silts, contractors may have difficulty achieving the proper density unless compaction is near optimum for the field working conditions. Thus, it is common to allow as much as 3 to 5 percent moisture either side of the laboratory optimum in quantitative specifications. Since the more moisture-sensitive soils generally have a lower optimum moisture content, specifications expressed in terms of percentage of optimum rather than percent moisture may be used. Thus, for subgrades and embankments, Virginia allows plus or minus 20 percent of optimum, not to exceed 5 percent moisture content from optimum. This means that a soil with an optimum moisture content of 28 percent must be compacted within the range of 23 to 33 percent, whereas a soil with an optimum moisture content of 20 percent must be compacted within the range of 16 to 24 percent.

In extreme climates such as the humid areas of Washington or Oregon or the arid regions of Arizona and New Mexico, compaction may have to proceed under very wet or very dry conditions for construction to be practical. This is not serious if embankment and pavement design is coordinated with the properties of the soils compacted at the prevailing environmental conditions.

As suggested by laboratory data, swelling clays should be compacted wet of optimum in order to minimize swelling potential. In humid regions this normally occurs because of the naturally wet condition of such soils. Indeed, workability from the standpoint of equipment mobility and spreading the lift may govern the upper limit of the compaction moisture content unless special stabilization techniques, such as the use of lime, are employed. In extremely arid regions or in embankments under heavy overburden pressures, swelling potential assumes less importance and a strength advantage may be gained by compacting dry of optimum.

The specification and control of moisture for field compaction is as difficult as it is important. Although test sections may be built and rolled to determine desirable compaction densities, it is difficult in the field to alter moisture content and study the influence of these adjustments on soil properties. The "family of curves" method is probably the best approach to the study of the influence of moisture on compaction properties, but it is impractical for routine highway construction work because of the large amount of time and sample required, the variability of subgrade materials encountered during normal construction operations, and the lack of positive correlation between laboratory- and field-compaction results.

Use of laboratory-compaction results The usual procedure for establishing compaction requirements and exercising field control is to reference moisture and density requirements to the standard impact laboratory-compaction test, AASHO T 99. The test is performed on representative primary materials obtained prior to the onset of construction. Control and acceptance procedures are then based on field density as a percent of the standard test and moisture content is referred to the optimum obtained in the laboratory. The laboratory moisture-density curves are generally available to field engineers for use as control curves. Field density and moisture content achieved by the contractor may then be measured and compared to the control curve that appears to be most representative of the material at hand. To aid in selecting the appropriate laboratory curve for the field soil, identification of the location where the initial sample was taken and a description of the soil sampled is included with the curve. A collection of jar samples of soils for which the curves were established may be kept at the job site as well.

Basing earthwork requirements and control on laboratory tests has both advantages and disadvantages. The chief advantage is that the laboratory work must be done regardless of any decision to use the data in the field in order to obtain soil parameters for pavement design, to recognize problem soils and techniques for their handling at an early stage, and to locate borrow areas and select suitable materials for components of the pavement. It is recognized that the densities and physical properties of soil samples compacted by laboratory methods may differ appreciably from densities and properties of the same soils compacted by field equipment. However, a lack of both precision and accuracy often marks field control tests and it is thought by many that the amount of variation between laboratory- and field-compaction results for the same soil, or subtle variations in the soils sampled as opposed to those compacted in the field, may be of little significance in the face of the data variations for control tests. Still, applying laboratory moisture-density curves to soil encountered in the field is a considerable problem in many places. The primary soil materials are often mixed in earthmoving operations so that none of the laboratory curves directly apply to the material being used as subgrade. Furthermore, the variability of natural soil deposits causes a constant change in the raw materials being utilized for subgrades and embankments. Finally, in many places, and especially for base and subbase materials, the field soil contains large percentages of rock fragments. Laboratory impact compaction quickly becomes less suitable as this percentage increases.

There is little doubt that the use of laboratory data should be abandoned in favor of field test sections to establish construction procedures and evaluate compaction requirements for individual projects. However, if soil variability in the field is so great as to make difficult the application of appropriate laboratory control data, it will be equally difficult to construct the necessary number of meaningful field test sections. Thus, for variable subgrades and the cut-and-fill embankment construction of highway work, some manner of referencing compaction requirements to laboratory work will probably continue to have widespread use.

FIELD-COMPACTION TEST METHODS

Before introducing the subject of establishing compaction requirements on the basis of field-compaction test results, rather than laboratory results, it is pertinent to review the common field-compaction control test procedures. These procedures are an integral part of most methods for establishing requirements in the field. Accordingly, this section considers first quality-control procedures for establishing the density of the finished product and second, the several techniques for establishing compaction requirements utilizing field test results.

It should be constantly held in mind that the primary objective of field-compaction testing is control of the contractor's treatment of the construction material during construction, not after it. The total rejection of large quantities of earthwork, or insistence through ignorance on an unrealistic level of stabilization for the material at hand, inflicts heavy damage on the contractor. The expense of this will ultimately be felt by the contracting agency. For subsequent work by that agency, contractors will bid at high levels to allow for the contingency of rejection of their finished product. Accordingly, it is in the interest of both the agency and the contractor for the quality control to be reasonable, realistic, and effective. Further, quality-control procedures should not result in large delays in construction or in pointing out deficiencies in construction too late for corrective measures to be taken. For example, if insufficient density has been attained in a compacted lift because of an inadequate number of coverages, it behooves the inspector to determine this before further compaction can be achieved only at great inconvenience to the contractor.

7-9 QUALITY-CONTROL TEST PROCEDURES

Quality-control test procedures usually include the measurement of dry density and comparison to some maximum density that is known to be obtainable for the material. The comparison is spoken of in terms of *percent compaction*, which for this purpose is defined as the ratio of the dry density obtained in the field to the established maximum, expressed as a percentage. Dry density is normally found by measuring mass unit weight and dividing by $1 + w$. Thus, moisture-content measurements are also required even when water-content requirements are not specified quantitatively. Testing time, which can cause construction delay, is an important factor in selecting the test procedures. Most of the conventional density tests become less reliable when rock-size particles are encountered.

"Proof rolling" or test rolling by means of heavy pneumatic-tired rollers is also used as a quality-control test. Tables of operational specifications and equipment for proof rolling have been prepared [21]. Proof rolling is most often used after subgrade compaction and before base-course placement, but it may be used also on natural subgrade in cut areas to determine the need for compaction. It is especially useful for naturally wet soils where areas of low density and/or excessive moisture will become apparent. For dry soils, the subgrade may show sufficient strength under proof rolling but still prove unstable when subject to wetting during service. When proof rolling is employed, it is usually used in conjunction with density testing.

Despite the numerical nature of specifications and test results on compacted density, considerable engineering judgment enters into the quality-control procedure. It has been said that on many projects an experienced inspector could pass or fail a compacted lift simply by confining his testing to the firm or soft spots that invariably appear at isolated locations in the section. It is known that when statistical procedures are used for selecting test locations, wide variability in results on density often occurs. This is a matter of considerable concern, since not only is sufficient compaction sought, but nonuniform or variable compaction should be controlled. To this end, the inspector must constantly observe areas that traditionally give trouble. Johnson and Sallberg [7] note that these areas may occur where

1. Oversized rock is contained in the fill.
2. Frozen materials were placed.
3. Material differs markedly from normal materials.
4. Improper type and rating of compactor was employed.
5. Compactor may have lost ballast.
6. Compactors have been turned at end of trip.
7. Junctions occur between tamped and rolled or vibrated soils.
8. Embankment operations are concentrated.
9. Dirt-clogged rollers (sheepsfoot type) were used.
10. An insufficient number of passes were applied.
11. Lift thickness was excessive.
12. Moisture content was insufficient or in excess.

Conventional field-density tests Most of the conventional field-density tests require digging a hole into the compacted material, determining the weight and water content of the soil removed, and finding the volume of the hole created. The various methods differ chiefly in technique for measuring hole volume and method for the rapid determination of water content.

The sand-replacement test (ASTM D 1556-64) is probably the most popular and the most accurate of the methods for measuring the volume of the hole. A dry, uniform sand calibrated as to the density it assumes when poured from a standard container is used to fill the hole. With determination of the weight of sand used, hole volume can be computed. The test takes from 30 min to an hour to perform. Other techniques, none of which is necessarily more rapid or accurate, include refilling the hole with water contained in a rubber pouch inserted to line the hole (ASTM D 2167-66), or refilling with viscous oil, plaster, or paraffin.

Because of the time required, the standard laboratory technique for determining moisture content in a thermostatically controlled oven cannot be used for field-density work. Instead, the soil may be dried over an open flame, with the aid of forced-draft heaters, or by pouring alcohol over the sample and igniting. Such techniques work best for coarse materials and may be relatively unreliable for fine-grained soils. The Speedy Moisture Tester, on the other hand, gives fairly good results for fine-grained soils but is unsuitable for coarser materials because of the small sample utilized. Fortunately, variations of 1 percent or so in water content do not greatly affect the computed dry density. Also, the speed and small sample required for these rapid methods allow economical replication of determinations.

Nuclear field-density tests As innovations in construction methods and equipment continue to increase the rate of highway construction, it becomes increasingly important to improve on rapid methods for control testing. The most significant breakthrough in this field in recent years has been the use of nuclear methods. They are not only far more rapid than conventional methods, but they offer as well a greater degree of freedom from human error and require less judgment on the part of the operator. The major disadvantages are the need for training technicians in their use, the high cost of the equipment, and the time and effort required to check out and calibrate new equipment and repair older equipment. Still, as agencies and their personnel gain experience with the method, its acceptance among inspectors and contractors alike is rapidly increasing.

For density, the nuclear method is based on the absorption of gamma rays. A source in the instrument emits gamma rays (photons) into the soil. Through a series of collisions with the electrons of the materials making up the soil, the photons may be scattered in all directions and some will be absorbed owing to their energy loss with each collision. A detector is placed a certain distance from the source and the number of photons reaching the detector is counted. With a constant source, the photon count should depend only on the geometry of the instrument and the absorption capacity of the soil. This capacity will vary with soil density so that, for a fixed instrument geometry, there is a definite relationship between soil density and detector count.

For moisture content, the nuclear method is based on the fact that neutrons emitted from a source are more effectively slowed by hydrogen atoms in a soil-water medium than by any of the other atoms normally present. For neutron speed to be greatly reduced, it must collide with a nucleus nearly equal to that of the neutron. This is the case with hydrogen. Thus, when a detector for slow neutrons is placed a fixed distance from a neutron source, the count obtained will relate to the hydrogen in the soil, which is present primarily in water. Accordingly, a direct relationship between count and the amount of water present may be obtained.

Nuclear devices may be either in the form of probes designed to be lowered into the ground or surface gages which operate by emitting rays into the soil that are scattered or reflected back after interaction with the soil. For compaction control, surface gages are usually used. The unit-weight tests can be made in about one-tenth of the time required for conventional methods. However, the operator cannot control the depth or volume of soil being tested; it will depend on the dimensions of the gage and the moisture content and unit weight of the soil. In unit-weight tests, sampling depth decreases as density increases, with maximum depth usually 4 to 6 in. Likewise, moisture sample size decreases with increasing moisture content, with maximum depth varying from 5 to 15 in.

Nuclear gages must be calibrated for the construction materials with which they are to be used, especially in the case of density measurements. Hence, they can be used most efficiently on relatively uniform material such as base and subbase. Detailed calibration can be neglected when the density measurements need only be made on a comparison basis. For example, if a field test section is used to establish required density, and if measurements on the test section are made with a nuclear device, one may proceed with confidence with comparative nuclear measurements on the subsequent construction as long as the character of the materials and density requirement is not altered. For bases and subbases, for which this procedure is most applicable, this greatly overcomes the objection that coarse particles interfere with density measurements.

The increasing use of nuclear methods in conjunction with field test sections may lead to the elimination of standard laboratory moisture-density curves for compaction control. Moreover, because nuclear measurements can be made rapidly, with perhaps 10 measurements made in the time normally required for the conventional density tests, density specifications of the future are likely to include statistical concepts.

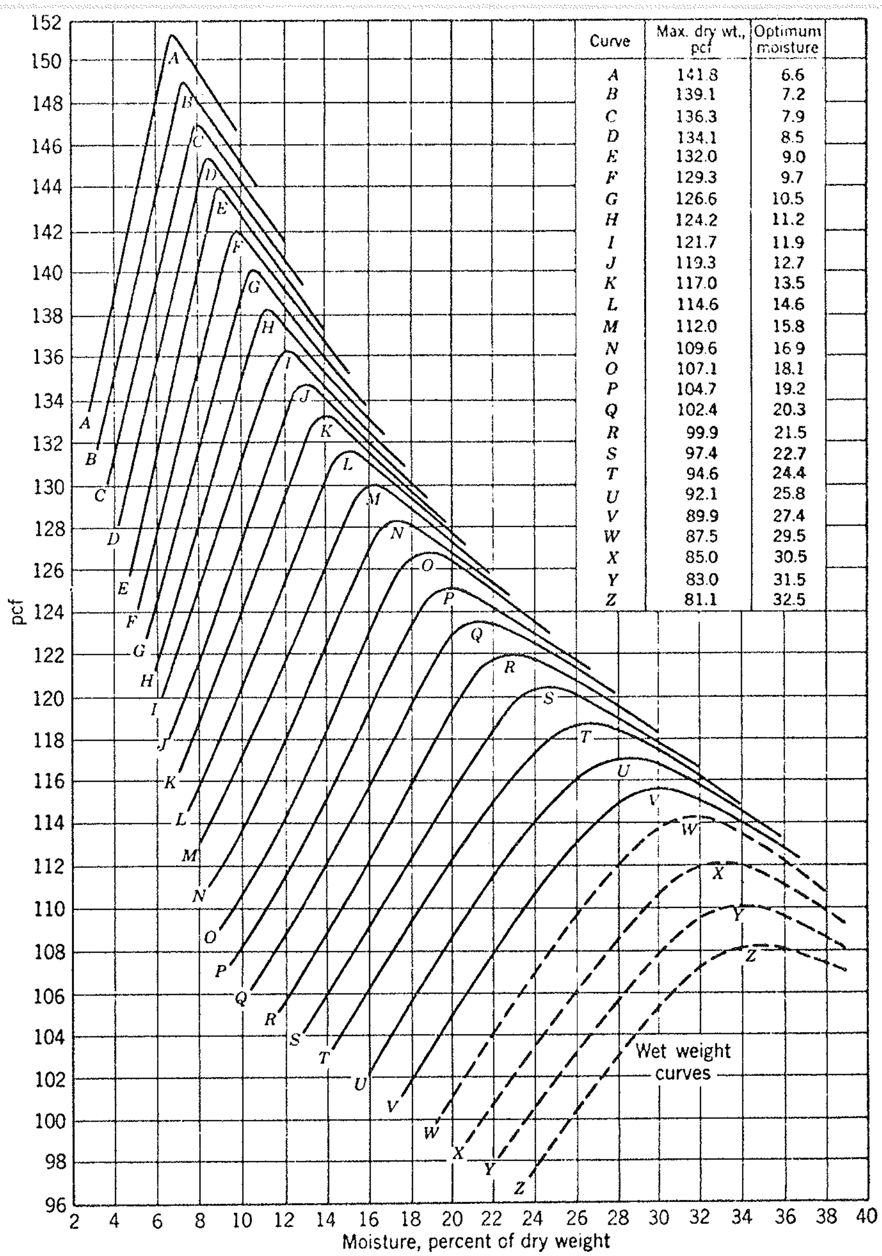
7-10 FIELD IMPACT COMPACTION TESTS

To overcome the considerable difficulty of relating laboratory moisture-density curves obtained for samples of primary materials prior to construction to the mixed and variable soils often encountered in the field, several techniques have been devised for establishing in the field and during construction operations the compaction characteristics of the soil at hand. These methods are necessarily rapid, since the delay of using standard laboratory test procedures, even when a field laboratory is available, is out of the question. However, all the methods are based on the impact technique of laboratory compaction.

The three tests discussed below are the *Ohio typical moisture-density curves method*, the *Hilf method*, and the *constant dry weight* (CDW) *compaction test*. The Ohio typical curves method is the earliest to be developed and is still in use today. It has been found in Virginia, for example, that it can easily be applied to areas other than Ohio with few modifications in the "typical" curves. The Hilf method is the most elegant and precise of the three methods and is used by the Bureau of Reclamation. The recently devised CDW test is probably the most rapid of the methods, but requires some judgment on the part of the operator as to the proper molding moisture content for the soil. The methods are only briefly outlined below to emphasize the principles on which they are based and their relative advantages. For details on their procedure, the reader is referred to Refs. 2, 7, 9, and 25 for the Ohio typical curves method, Refs. 5 and 7 for the Hilf method, and Ref. 17 for the CDW method.

Ohio typical moisture-density curves method The Ohio typical curves method was developed by K. B. Woods and R. R. Litehiser [25]. It is based on the finding that moisture-density curves have characteristic shapes. Hence, if these shapes are known, one need only determine one point on the curve to define maximum density and optimum moisture content.

On the basis of 10,149 tests, a set of 26 typical curves (Fig. 7-18) has been developed. Note that the curves are for wet density vs. moisture content instead of the customary dry density. Accompanying the curves is a table giving the maximum dry density and optimum moisture content corresponding to each wet density curve. To determine the curve that applies to a given soil, two steps must be followed. Soil must be compacted into a test mold using the standard impact method and the wet density determined. This may be done with soil removed from a test hole dug to determine field density by one of the conventional methods. The second step is to determine moisture content. For this, Ohio uses a penetration-resistance test using a Proctor Penetration Needle, a spring-loaded device with a small-diameter plunger that measures deformation resistance of the compacted soil. As with the typical moisture-density curves, typical penetration-resistance–moisture curves



Curve	Max. dry wt., pcf	Optimum moisture
A	141.8	6.6
B	139.1	7.2
C	136.3	7.9
D	134.1	8.5
E	132.0	9.0
F	129.3	9.7
G	126.6	10.5
H	124.2	11.2
I	121.7	11.9
J	119.3	12.7
K	117.0	13.5
L	114.6	14.6
M	112.0	15.8
N	109.6	16.9
O	107.1	18.1
P	104.7	19.2
Q	102.4	20.3
R	99.9	21.5
S	97.4	22.7
T	94.6	24.4
U	92.1	25.8
V	89.9	27.4
W	87.5	29.5
X	85.0	30.5
Y	83.0	31.5
Z	81.1	32.5

Fig. 7-18 Typical moisture-density curves. (*From Joslin* [9].)

are available. Finding the two curves that best fit the data obtained then defines the moisture content. Since alternate methods of rapidly finding moisture content, such as the Speedy Moisture Tester, are not difficult, it is not necessary to rely on the penetration-resistance approach. In any case, once moisture content is known in addition to wet density, the plot of typical curves and the accompanying table can be consulted to determine the compaction characteristics of the soil. If this is done in conjunction with a field-density test, percent compaction is readily calculated.

Use of the typical curves method, usually with some modification, has expanded from Ohio to many geographic areas. Wyoming adopted 17 of the curves, added 3 more, and modified them for application to their area. Experience in Virginia has been that with some minor corrections, the curves apply very well to Virginia soils except when the soil is on the wet side of optimum moisture content or dry of optimum by more than 3 percent moisture.

The curves cannot be expected to apply to unusual materials such as uniformly graded sand, highly micaceous soils, diatomaceous earth, volcanic materials, or soils for which the specific gravity of the solids differs greatly from 2.67.

Hilf method The Hilf method was developed by J. W. Hilf [5] with the Bureau of Reclamation for rapid compaction control for fine-grained soils. The method does not require water-content determinations, can be completed in an hour or less, and gives the inspector the exact percentage of standard maximum dry density in a compacted fill and a close approximation of the difference between optimum moisture content and that of the soil in place. The test is basically a three-point compaction test using the impact procedures and compactive effort of the standard AASHO test, but slightly modified equipment. Its chief disadvantage is the complicated manner in which the data obtained must be manipulated.

The data obtained consist of field density, in terms of wet unit weight, and the results of three impact compaction trials. These trials yield three points for a wet-unit-weight–moisture-increment curve. Moisture increment is in terms of water added to the soil. For the first point, no moisture is added, for the second, 2 percent of the sample wet weight, and for the third 4 percent of the sample wet weight or, if the 2 percent increment caused density to decrease, some moisture decrement obtained by drying. A parabola is then constructed through the points by carefully outlined procedures or, for more approximate work, a compaction curve is drawn freehand. Maximum wet unit weight for the soil is considered the peak point of this curve and is compared to in-place wet unit weight to obtain percentage of standard maximum dry density. This is done graphically. Finally, the difference between optimum water content and in-place water content is approximated using the moisture increment corresponding to the peak wet density with the addition of a correction obtained from a standard series of curves.

The method may give excellent results for earthwork control for highway construction, but to the author's knowledge it has not yet been applied on a routine basis. The complicated data processing is certain to slow its acceptance by inspectors. Further, it can be argued that since the degree of correlation between results given by impact compaction and those from compaction with field equipment is not great to begin with, a high level of precision in arriving at percent compaction is not justified.

Constant dry weight method The constant dry weight (CDW) method was developed by R. Schonfeld [17] with the Ontario Department of Highways. It is rapid, independent of laboratory work or any standard curves or charts except for a simple "dipstick," and requires no weighing or moisture determination. Basically, it is a volumetric test that compares the volume of a soil sample from a compacted lift with the volume achieved with the same sample after standard compaction in the standard mold. The volume of sample obtained from the subgrade is determined using one of the conventional field-density methods. After the sample has been compacted into the mold, its volume is found with the aid of a calibrated gage or dipstick. Percent compaction is then computed as the ratio of the volume of the sample in the mold to that in the test hole expressed as a percentage.

The procedure requires that the moisture content of the sample from the test hole be carefully observed and, if it is not close to optimum, adjusted. There is considerable evidence that experienced inspectors can identify the optimum moisture condition in soils with which they are familiar. Cohesive soils can be examined in terms of degree of cohesiveness. If a ⅛-in. thread can be rolled, the soil is too moist because optimum is invariably less than plastic limit. If the soil is too dry, it will not retain its shape when molded into a ball and the pressure released. For nonplastic soils, evidence of dilatancy is looked for. If vigorous shaking of the sample causes a sheen from moisture at the surface of lumps, it is too wet. Granular soils may be compacted at a water content somewhat above saturation, since drainage during the test will yield a moisture content commensurate with maximum density. Compactive effort is adjusted according to the volume of the sample by adjusting the number of hammer blows according to a set of tabulated values.

The CDW test has been only recently developed so that its use is not yet widespread. However, its accuracy and precision appear to be greater than might be expected from the empirical methods of adjusting compactive effort and moisture content. Its advantages of simplicity and speed suggest that it may become popular. As statistical procedures for compaction control become more accepted, such rapid methods will increase in importance.

7-11 THE CONTROL-STRIP TECHNIQUE

For granular base and subbase materials and uniform subgrades for which the soil is obtained from borrow pits, the most reliable means for establishing satisfactory construction procedures and acceptance criteria is by use of field test sections. They do not depend on the questionable correlation of the results of impact test methods with compaction by field equipment and are especially applicable to the granular material of bases, where impact tests are most limited. Further, field test sections establish an achievable control density; there is no doubt in the mind of either the inspector or the contractor that a density close to that attained in a test section is a practical requirement for subsequent construction with the same material. It is doubtful if test sections will ever prove practical for variable subgrades or cut-and-fill embankment construction. Also, a rapid means for testing density must be available. However, the speed of nuclear testing and the requirement that such test data be only relative, a considerable advantage for the use of nuclear gages, are resulting in the increased use of the field test section method of construction control.

One such method, the control-strip technique, provides a good example of the use of field test sections. The technique has been used by some states, notably Ohio, for many years. Its use by Virginia has recently been described in detail by Anday and Hughes [1] and will be outlined here.

For the Virginia control-strip technique, a roller pattern is obtained on a control strip, a 300-ft section of one-lane roadway, with the construction material placed on a firm subgrade or subbase at the job site. The material is placed at optimum moisture content as established in the laboratory. Compactive effort is increased by successive rolling with equipment of a specified weight. Three density tests are performed with nuclear equipment after each rolling increment, which may be two or more passes at first, but it is reduced to one coverage as maximum density is approached. The average of the three tests is then used to determine the density increment from the additional rolling. If the increase in dry density is less than 1.0 pcf per pass, two additional passes are required and if these additional passes do not add at least 2.0 pcf to the dry density, rolling is discontinued and it is considered that maximum dry density has been achieved. After the maximum density has been reached, 10 random moisture and density tests are run to provide a good value of dry density that may be used for control of subsequent construction. The completed control strip is then part of the construction and for the rest of the project, larger "test sections" of 2,000 ft of one-lane roadway are designated for compaction control on the basis of the results obtained for the control strip. These sections are tested randomly at five locations and, from statistical analysis, compliance is considered to consist of having an average density for the five tests at least 98 percent of the average on the control strip, with each individual test value at least 95 percent of the control-strip average. However, control on moisture content or equipment is not exercised for the test sections. If the section does not meet the density criteria, additional rolling and retesting is required. New control strips are requested when (1) a change in the source of the material is made, (2) a change in the material from the same source is observed, or (3) 10 test sections have been approved without the construction of additional control strips.

Nuclear equipment must be used with this method; otherwise the time required for the many tests would be prohibitive. This results in the disadvantages to the enforcing agency of bearing the cost of the equipment and having inspectors trained in its use. Also, nuclear test methods do not measure the distribution with depth of the density in the compacted lift and readings are most influenced by conditions at the surface. Thus, if "crusting" occurs, it may pass undetected. Crusting, however, should not be a problem when proper materials and equipment are used. A considerable advantage is that calibration of the nuclear devices is not critical as long as the sensitivity of the calibration curve is adequate. Finally, there is little doubt that the use of statistical quality-control procedures is an important advancement in earthwork control methods.

PROBLEMS

7-1. Using the conventional procedure, plot the dry-density–moisture-content curve for the test results given below and determine the following:

(*a*) Optimum moisture
(*b*) Maximum dry density
(*c*) Saturation moisture content at maximum dry density
(*d*) Degree of saturation at maximum dry density

Water content, %	*Wet unit weight*	*ZAVD*
12.0	96.2	124.4
16.0	113.7	115.2
20.0	122.3	107.3
24.0	121.5	100.4
28.0	115.2	94.3

7-2. List the following soils in the order of decreasing dry density after standard compaction at optimum water content:

(*a*) A-8, A-1-b, A-5, A-2-4
(*b*) GW-GM, CH, SC, CL-ML

7-3. How is it possible for a contractor to achieve 100 percent standard AASHO dry density at a water content substantially less than optimum?

7-4. Why is it important that a contractor achieve not only the specified dry density for a clay soil, but that he achieve this dry density at a compaction water content not far from "optimum"?

7-5. When soil is compacted, changes occur according to changes in conditions of compaction, other things being equal. For example, increasing compactive effort causes an increase in the dry density achieved. Also, changes in soil type, such as percent fines in a granular soil, may cause a change in the density of the final product. For the change in conditions mentioned below, indicate with the word *increase* or *decrease* their effect on the compacted soil property mentioned, assuming other things to remain the same.

(*a*) Change in dry density with increase in compactive effort
(*b*) Change in optimum water content with increase in compactive effort
(*c*) Change in dry density for increasing soil plasticity
(*d*) Change in optimum water content for increasing soil plasticity
(*e*) Change in percent swell with increasing moisture of compaction
(*f*) For an aggregate, change in dry density for increasing C_u

7-6. It is desired to lengthen an airport runway so that it may accommodate jet aircraft. The subgrade soil for the runway extension is fine grained and gave the following laboratory results when compacted using modified AASHO compaction techniques:

Maximum dry density = 118.7 pcf
Optimum water content = 16.3%

The contractor used heavy equipment during construction and preparation of the actual subgrade so that he actually achieved the following:

Dry density reached by field compaction = 118.6 pcf
Actual water content of subgrade compaction = 12.4%

(*a*) Give two good reasons why the prepared subgrade is unsuitable for use.

(*b*) If the laboratory-compacted soil was 84 percent saturated with moisture, what was the percent saturation of the field-compacted soil in the "as-compacted" condition?

(*c*) Considering "as-compacted" properties, indicate how the properties of the field-compacted soil compare with those of the laboratory-compacted soil considering the following: void ratio, stiffness, strength, and swelling potential.

(*d*) Considering "in-service" or "soaked" properties, indicate how the properties of the field-compacted soil compare with those of the laboratory-compacted soil considering the following: void ratio, strength, and stiffness.

7-7. Using the family of curves given in Fig. 7-17, determine the soaked CBR that might be expected for the soil when compacted to 95 percent "standard AASHO" maximum dry density at a water content of 16 percent.

7-8. For compaction control using the Ohio typical moisture-density curves procedure, the following test data were obtained on compacted subgrade:

Volume of field-density test hole (sand-cone procedure) = 0.055 ft³
Weight of soil removed from field-density test hole = 6.60 lb

Moisture content of soil from field-density test hole = 16.5%
Weight of soil in standard compaction mold after AASHO T 99 compaction = 4.15 lb

Determine the percent compaction achieved by the contractor.

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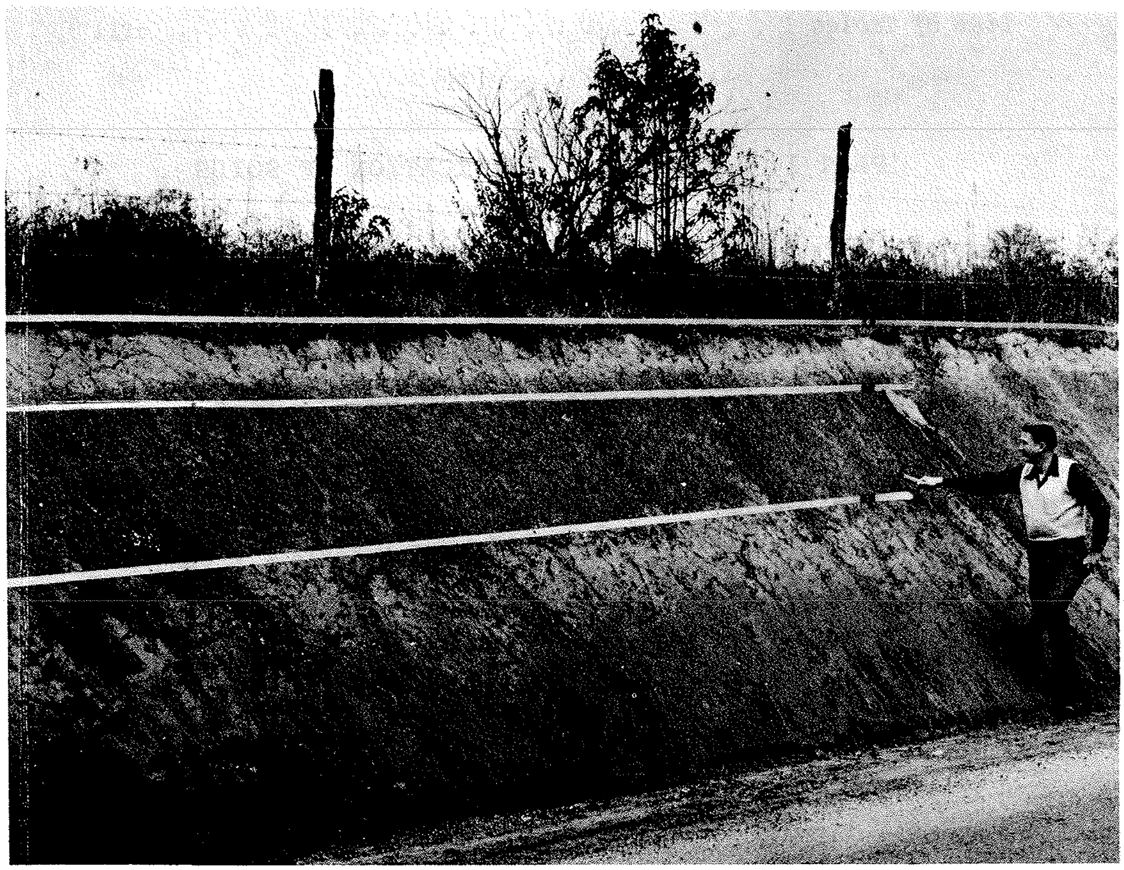
SOILS MANUAL

FOR THE DESIGN OF ASPHALT PAVEMENT STRUCTURES



Example of soil profile with well defined horizons.

THE ASPHALT INSTITUTE

MANUAL SERIES No. 10 (MS-10)

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CONTENTS

CONTENTS (CONTINUED)

PART 2: DETERMINATION OF STRENGTH VALUES FOR THE DESIGN OF ASPHALT PAVEMENT STRUCTURES

CHAPTER IV

SIGNIFICANCE OF TESTS ON SOIL MATERIALS

4.01 GENERAL.--Highway and airport engineers and soil technicians are acquainted with the basic tests performed in soils laboratories. Frequently, however, the acquaintance is superficial because of lack of experience with the tests. Many technicians are quite skillful in performing the tests but cannot interpret the results. The American Society of Civil Engineers published, in the September 1957 issue of the *Journal of the Highway Division*, a progress report on "Significance of Tests for Highway Materials--Basic Tests." This report was prepared so that those using the tests could appreciate the significance of the results.

Because it is important to know the significance of tests as well as the mechanics of performing them, three of the tests on soils described in the ASCE report are included in the following paragraphs. They are the mechanical analysis, consistency tests and indices, and the moisture-density test. These descriptions have been revised to update changes in tests. The test for specific gravity, not included in the ASCE report, also is described in this chapter.

A. PARTICLE SIZE ANALYSIS OF SOILS
(Sieve and Hydrometer Analyses)
ASTM Test Designation: D 422*
AASHTO Test Designation: T 88*

4.02 SIGNIFICANCE OF TEST.--The *mechanical analysis* of a soil is the determination of the percent of individual grain sizes present in the sample.

The results of the tests are of most value when used for classification purposes. Further use of the gradation should be discouraged unless verification by studies of performance or experience permit empirical formulae. Only rough approximations of strength or resistance properties should be attempted. Quite often it will be found that the larger the grain size, the better are the engineering properties. Also, it is a known fact that detrimental capillarity and frost damage are not a problem with the coarse (sandy) material, whereas they can be

*ASTM and AASHTO tests differ in some details.

very dangerous with the fine-grained silts and clays. Some empirical relationships have been developed such as the criterion commonly used for determining the susceptibility of soil to pumping under rigid pavements. Highway specifications for subbase and base materials also use the grain size analysis for quality measurement.

For soil stabilization, use is frequently made of grain size analyses for mix design and control. One criterion for asphalt stabilization is a requirement for a minimum percent of sand and gravel size. The percent of cement to be used in soil-cement mixtures can be estimated on the basis of the grain size. For mechanical stabilization or aggregate bases (well-graded, granular material with or without a chemical admixture) the results of the gradation tests are used to determine the size and percent of aggregates or fines that are needed for a dense, impermeable material.

On occasion, the degree of permeability (measure of the amount of water that will flow through a material) is estimated on the basis of grain size. Here again, certain generalities are possible but accurate estimates are not. The larger-grained soils will more readily permit the flow of water than finer-grained ones, i.e., sands are more permeable than silts, and silts are more permeable than clays. An example of the variation from this generality is a well-graded, granular material which can be sufficiently impermeable to serve as a core for an earth dam.

4.03 SYNOPSIS OF TEST METHODS.--The mechanical analysis consists of two parts: one, the determination of the amount of coarse material by the use of sieves or screens; and two, the analysis for the fine-grained fraction commonly employing an hydrometer analysis.

The sieve analysis is a simple test consisting of sieving a measured quantity of material through successively smaller sieves. The weight retained on each sieve is expressed as a percentage of the total sample.

The hydrometer analysis is conducted on a sample of the material that passes a 2.0 mm (No. 10) sieve.* The test is based on the principle that the soil can be dispersed uniformly through a liquid. The specific gravity of the soil-liquid mixture is then measured at various time intervals. Stoke's Law is used to compute the rate of settling of the various sizes; i.e., the larger grains settle more rapidly than the smaller grains. The computations include corrections for temperature, viscosity of the liquid and the specific gravity

*Note: AASHTO Designation T 88 contains alternative hydrometer methods; one using material passing a 2.0 mm (No. 10) sieve, the other using material passing a 0.425 mm (No. 40) sieve.

of the soil particles. The results are first expressed as a percent of the sample used in the hydrometer analysis, and then converted to percentages of the total soil sample if there is a coarse grained fraction.

4.04 TYPICAL TEST RESULTS.--The results of the mechanical analysis can be presented in either of two forms. One is a table in which there is listed the percentage of the total sample that will pass a given sieve size or is smaller than a specified grain diameter. The second form is a plot of the sieve number or grain size vs. the percentage passing the given sieve (smaller than the given diameter). For this latter form, grain size is normally plotted on a logarithmic plot due to the wide range in values, while the percentage finer is plotted on an arithmetic scale.

Nomenclature has been established for materials within certain grain-size limits. In decreasing order of size these groups are as follows:

ASTM Designation D 422

Gravel, 75 mm to 4.75 mm (3-in. sieve to No.4 sieve)
Coarse Sand, 4.75 mm to 2.00 mm (No. 4 to No. 10)
Medium Sand, 2.00 mm to 0.425 mm (No. 10 to No. 40)
Fine Sand, 0.425 mm to 0.075 mm (No. 40 to No. 200)
Silt size, 0.075 mm to 0.005 mm (No. 200 to ---)
Clay size, smaller than 0.005 mm
Colloids, smaller than 0.001 mm.

AASHTO Designation T 88

Particles larger than 2.0 mm (No. 10 sieve)
Coarse Sand, 2.0 mm to 0.425 mm (No. 10 to No. 40)
Fine Sand, 0.425 mm to 0.075 mm (No. 40 to No. 200)
Silt, 0.075 mm to 0.002 mm (No. 200 to ---)
Clay, smaller than 0.002 mm
Colloids, smaller than 0.001 mm.

Soils designated as sandy will contain more than 50 percent sand or gravel size. Silty soils will contain from 40 percent to as great as 100 percent silt size. Clays will contain as low as 30 percent or as high as 100 percent clay and colloids. Gravelly soils will normally contain at least 15 percent gravel-size material.

4.05 INFLUENCES OF THE METHODS OF TEST.--For the sieve analysis, care must be taken to remove clay and silt that may be adhering to the sand and gravel. In preparing the sample for testing, one must avoid fracturing some types of soft gravel and stone particles. For the very fine sands [0.150 mm or 0.075 mm (No. 100 or No. 200 sieves)] it will be desirable to wash the sample through the sieves.

The hydrometer analysis is particularly susceptible to poor results due to technique. The following are major sources of error:

1. Improper deflocculation (failure to separate the material into individual grains).
2. Improper mixing of soil and liquid.
3. Careless placement and removal of the hydrometer.

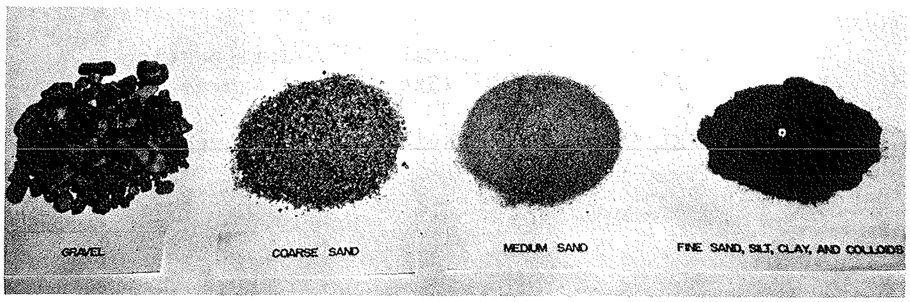

Figure IV-1. Separated soil after sieving.

B. SPECIFIC GRAVITY OF SOILS
AASHTO Designation: T 100*
ASTM Designation: D 854*

4.06 SIGNIFICANCE OF TEST.--The *specific gravity* of a soil is the ratio of the weight in air of a given volume of soil particles at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature. The specific gravity is used frequently in relating a weight of soil to its volume. The unit weight of moist soil--needed in most pressure, settlement, and stability problems--can be computed with known values for specific gravity, degree of saturation, and void ratio. The specific gravity is used in the computations of many laboratory tests on soils.

In many soils the presence of a number of minerals, each having a different specific gravity, may present difficulties. This is why the test method requires that the Method of Test for Specific Gravity and Absorption of Coarse Aggregate, ASTM Designation C 127 or AASHTO Designation T 85, be used for the coarse portion when the soil has material retained on the 4.75 mm (No. 4) sieve. The specific gravity for the soil then is

*These methods differ only in requirements given for desiccator, oven thermometer, and weighing.

determined from the weighted average of the values for the coarse and the fine portions, using the following formula:

$$\text{Combined specific gravity} = \frac{1}{\frac{pc}{gc} + \frac{pf}{gf}}$$

in which pc = percent of coarse portion expressed as a decimal
gc = specific gravity of coarse portion
pf = percent of fine portion expressed as a decimal
gf = specific gravity of fine portion

4.07 SYNOPSIS OF TEST METHOD.--The prescribed weight of the sample [all passing the 4.75 mm (No. 4) sieve or the 2.0 mm (No. 10) sieve depending upon the purpose of the test] is placed carefully in a calibrated pycnometer. Distilled water is added to fill the flask about three-fourths full. The entrapped air in the soil then is removed by partial vacuum (air pressure not exceeding 100 mm of mercury) or by boiling. The calibrated pycnometer then is filled with distilled water and weighed. The specific gravity is computed, using the determined weights and temperature corrections.

4.08 TYPICAL TEST RESULTS.--The specific gravities of soils range from below 2.0 for organic or porous particle soils to over 3.0 for soils containing heavy minerals. Most soils, however, have specific gravities in the range of 2.65 to 2.85. A soil containing different minerals can have a range of specific gravities, depending on the care used to obtain a representative sample.

4.09 INFLUENCES OF THE METHOD OF TEST.--Accurate results depend upon extreme care in obtaining weight and temperature measurements. A small error may be quite significant in the results. Calibration of the pycnometers, complete removal of entrapped air, and drying of the samples should be done with precision.

C. CONSISTENCY TESTS AND INDICES
(Atterberg Limits)

Liquid Limit	ASTM Designation: D 423*
	AASHTO Designation: T 89*
Plastic Limit and Plasticity Index	ASTM Designation: D 424†
	AASHTO Designation: T 90†
Shrinkage Limit	ASTM Designation: D 427†
	AASHTO Designation: T 92†

4.10 SIGNIFICANCE OF TEST.--The *consistency* tests or the Atterberg Limits consist of the liquid limit, the plastic limit, and the shrinkage limit. A value frequently used in conjunction with these limits is the plasticity index. The engineering properties of soil vary with the amount of water present, and results of the three consistency tests, expressed as moisture contents, are arbitrarily used to differentiate between the various states of the material. The liquid limit is the moisture content at which the soil changes from the liquid to the plastic state. The plastic limit is the border between the plastic and semi-solid, and the shrinkage limit delineates the semi-solid from the solid state. The plasticity index is the arithmetic difference between the liquid and plastic limits; i.e., it is the range of moisture content over which a material is in the plastic state.

The most common application of the test results to highway problems is in soil classification with those soils with comparable limits and indices classed together. Generally, soils with high liquid limits are clays with poor engineering properties. A low plasticity index indicates a granular soil with little or no cohesion and plasticity. Both the liquid limit and the plasticity index are used to some degree as a quality measuring device for pavement materials, in order to exclude those granular materials with too many fine-grained particles that have cohesive plastic qualities.

4.11 SYNOPSIS OF TEST METHODS.--The liquid limit test consists of molding a soil pat in a brass cup, cutting a groove in the pat with a special cutting tool and dropping the cup onto a solid base from a constant height. The liquid limit is that moisture content at which the groove closes for a length of 13 mm (1/2 in.) under 25 impacts.

The plastic limit test consists of rolling a soil sample into a thin thread. The soil thread is made by rolling a wet sample on a plate with the hand. This procedure is repeated until the sample crumbles when the diameter of the thread is

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*ASTM Designation D 423 and AASHTO Designation T 89 differ in a number of details.

†ASTM Designations D 424 and D 427 differ from AASHTO Designations T 90 and T 92 only in minor details.

equal to 3 mm (1/8 in.). The moisture content of the soil in this latter condition is the plastic limit of the soil.

The shrinkage limit is conducted by saturating a soil sample, placing the material in a small dish of known volume, and weighing. The specimen is then placed in an oven and dried to a constant weight. During the drying period, the sample shrinks and loses volume at a rate more or less proportionate to the volume of water evaporated until the shrinking stops abruptly. The shrinkage limit is the moisture content of the saturated sample at the time the shrinkage ceases.

During recent years, many laboratories have abandoned the use of the shrinkage limit. In the highway field, classification is most commonly made on the basis of the Transportation Research Board system, which requires the gradation, the liquid and plastic limits, and the plasticity index.

4.12 TYPICAL TEST RESULTS.--The liquid limit varies widely and values as high as 80 to 100 are not uncommon with values of 40 to 60 more typical for clay soils. For silty soils, values of 25 to 50 can be expected. The liquid limit test will not produce a result for a sandy soil, and the results are reported as "non-plastic."

The plastic limit of silts and clays will not vary too widely and will range from 5 to 30. Normally, the silty soils will have the lower plastic limit. Since a pure sand is non-plastic, the thin thread cannot be rolled and the material is termed "non-plastic." For the shrinkage limit, clays may range in values from 6 to 14, with silty materials most frequently showing values between 15 and 30. Pure sand will show no decrease in volume during the drying period.

The plasticity index can be as high as 70 to 80 for the very plastic clays. Commonly, clays will have P.I.'s between 20 and 40. The silty materials normally range in P.I. between 10 and 20. For quality evaluations, soils are sometimes restricted to those materials with a liquid limit of 25 or less and a maximum P.I. of 6, i.e., a predominantly granular material.

4.13 INFLUENCES OF THE METHODS OF TESTS.--For the liquid limit the most common sources of error include (1) inaccurate height of drop of the cup, (2) a worn cup due to scratching with the grooving tool, (3) too thick a soil pat, (4) the rate of dropping the cup and (5) the human element in deciding when the groove has closed 13 mm (1/2 in.).

In many laboratories where production is a major concern, the test is conducted using only one moisture content, taken when the material is considered to be at the liquid limit. In the more precise test, at least three moisture contents are determined, one below the liquid limit, one at or near the liquid limit and the third higher than the liquid limit. The results are plotted by placing the moisture content on an arithmetic plot with the number of blows on a semi-log basis. This permits a more exact establishment of the liquid limit.

D. MOISTURE-DENSITY TEST
ASTM Test Designations: D 698 and D 1557
AASHTO Test Designations: T 99 and T 180

4.14 SIGNIFICANCE OF TEST.--The *moisture-density* test is designed to aid in the field compaction of soils so as to develop the best engineering properties of the material. It is assumed that the strength or shearing resistance of the soil increases with higher densities. Thus, the test is designed to get the best results from the soil available.

The "standard" moisture-density test (ASTM D 698; AASHTO T 99) as conducted in the laboratory uses a constant laboratory compactive effort, and it is assumed that it is similar in magnitude to the weight, impact and action of the average construction equipment. As might be anticipated, a greater compactive effort will bring an increase in density, and such a procedure was followed in developing the so-called "modified" moisture-density test (ASTM D 1557; AASHTO T 180). Presumably, heavier construction equipment would be required to obtain the "modified" density than would be needed to get "standard" density.

Another important factor is that the presence of a certain amount of water is needed in order to get the densities desired. For simplicity, the water can be assumed to act as a lubricant. However, too much water tends to force the particles apart and the higher densities cannot be obtained. Therefore the laboratory test not only defines the density that should be obtained by the construction equipment, it also delineates how much water should be used during the compaction. 79

Given a density (termed maximum density) and the proper moisture content (termed optimum moisture content) the construction forces can compact the soil into the best condition practicable. As a check, field forces employ a density test to determine the density obtained by the construction equipment. If the results are lower than the values permitted by the specifications, the material should be recompacted.

4.15 SYNOPSIS OF TEST METHOD.--ASTM Designations D 698 and D 1557, and AASHTO Designations T 99 and T 180, each contain four methods (identified as Methods A, B, C, and D) for determining the relationship between the moisture contents and densities of soils. The differences in these procedures are shown in Table IV-1.

The total sample is permitted to dry in air until a damp condition is reached. From this, a sample of material is selected, the quantity depending on the method to be used. The soil is then compacted in the specified number of layers into

TABLE IV-1
ALTERNATIVE PROCEDURES FOR MOISTURE-DENSITY TESTS

Metric	ASTM D 698 AASHTO T 99				ASTM D 1557 AASHTO T 180			
Method	A	B	C	D	A	B	C	D
Rammer weight, kg	2.50	2.50	2.50	2.50	4.54	4.54	4.54	4.54
Rammer drop height, cm	30.5	30.5	30.5	30.5	45.7	45.7	45.7	45.7
Mold diameter, mm	102	152	102	152	102	152	102	152
Soil material passing sieve	4.75 mm	4.75 mm	19.0 mm	19.0 mm	4.75 mm	4.75 mm	19.0 mm	19.0 mm
No. of layers of soil in mold	3	3	3	3	5	5	5	5
No. of blows per layer	25	56	25	56	25	56	25	56
Customary	**ASTM D 698 AASHTO T 99**				**ASTM D 1557 AASHTO T 180**			
Method	A	B	C	D	A	B	C	D
Rammer weight, lb	5.5	5.5	5.5	5.5	10	10	10	10
Rammer drop height, in.	12	12	12	12	18	18	18	18
Mold diameter, in.	4	6	4	6	4	6	4	6
Soil material passing sieve	No. 4	No. 4	3/4 in.	3/4 in.	No. 4	No. 4	3/4 in.	3/4 in.
No. of layers of soil in mold	3	3	3	3	5	5	5	5
No. of blows per layer	25	56	25	56	25	56	25	56

a metal, cylindrical mold of designated volume. A metal rammer is dropped from a specified height on to the soil in the mold. The prescribed number of blows per layer is used. The weight of the soil in the mold is determined, and with the volume of the mold known, the density is computed by dividing the weight by the volume. A moisture content determination is made on the sample in the mold. The soil is then removed from the mold, pulverized, an increment of water mixed into the sample, and the compaction procedure repeated. The test continues until the weight of the compacted sample in the mold is equal to or less than that obtained in the preceding step.

4.16 TYPICAL TEST RESULTS.--The computations include a plot of the moisture content versus the density obtained with that moisture content. Calculations are then made of the density of the soil grains only, i.e., excluding the weight of the water. This density is also plotted versus the moisture content and is termed the "dry density" curve. The resulting plots are curved lines showing higher densities with increased moisture content up to some peak, and then lower densities with increased moisture content. The density at the peak of the dry-density curve is called the "Maximum Density" and the moisture content at that point is termed the "Optimum Moisture Content."

The following is a list of the range of values that might be anticipated for the standard moisture-density test:

Clays	-- Maximum density	1 440 - 1 680 kg/m^3 (90-105 lb/ft^3)
Silty Clays	-- Optimum moisture content	20 to 30%
	Maximum density	1 600 - 1 840 kg/m^3 (100-115 lb/ft^3)
Sandy Clays	-- Optimum moisture content	15 to 25%
	Maximum density	1 760 - 2 160 kg/m^3 (110-135 lb/ft^3)
	Optimum moisture content	8 to 15%

For the modified procedure using an increased compactive effort, maximum densities of 160 to 320 kg/m^3 (10 to 20 lb/ft^3) larger can be anticipated with optimum moisture contents of 3 to 10 percent lower.

For sandy or gravelly soils with no fines, there is no significant change in density with the use of water unless inundation methods are used.

Many compaction specifications require that a percent of the maximum density be achieved. This percent varies from 95 to 100 percent for the more granular materials and 90 to 95 percent for the fine-grained silts and clays. The percent of maximum density is the ratio of the density obtained to the maximum density expressed as a percentage.

4.17 INFLUENCES OF THE METHODS OF TEST.--The test is not particularly susceptible to dangers from poor laboratory technique. The degree of accuracy of the field density test, and that used in the laboratory will be similar.

However, certain laboratory precautions must be taken. In the mixing of the water into the soil, as thorough a mix as possible is necessary. In taking the moisture sample, care should be taken to obtain a representative sample. For very granular soils with a large portion retained on the 4.75 mm (No. 4) sieve, an adjustment is necessary to compensate for the removal of this material prior to testing.

There is considerable argument as to the adequacy of the laboratory compaction as compared to that obtained by construction equipment. The question arises as to the size, weight, and drop of the rammer, as well as the manner in which the soil is compacted. However, as long as the specifications require the standard test, the argument is not a factor to the construction forces in their routine operations.

The field density test to determine the density obtained by the construction equipment has been successfully conducted using any one of four different devices for measuring the volume of the hole from which a sample has been removed. The four techniques include nuclear, calibrated sand, a viscous liquid, and water encased in a light rubber membrane. A fifth method involving the removal of a sample by driving a thin-walled sampler into the soil is satisfactory for fine-grained silts and clays but not for material containing a significant amount of gravel or rock fragments.

CHAPTER VII

CALIFORNIA BEARING RATIO OF LABORATORY-COMPACTED SOILS

A. GENERAL

7.01 DEVELOPMENT AND DEFINITION.—The *California Bearing Ratio* (CBR) Method with its numerous variations is probably the most widely used method of designing asphalt pavement structures. This method was developed by the California Division of Highways around 1930 and has since been adopted and modified by numerous states, the U.S. Corps of Engineers, and many countries of the world. The Corps of Engineers adopted this method during the 1940s. Their test procedure was most generally used, with and without certain modifications until 1961 when the American Society for Testing and Materials adopted the method as ASTM Designation D 1883, *Bearing Ratio of Laboratory-Compacted Soils*. The ASTM procedure differs in some respects from the Corps procedure and from the American Association of State Highway and Transportation Officials (AASHTO) procedure, adopted in 1972 as AASHTO Designation T 193. The ASTM procedure is the easiest to use, and is the version described in this publication.

The CBR is a comparative measure of the shearing resistance of a soil. It is used in the design of asphalt pavement structures. This test consists of measuring the load required to cause a plunger of standard size to penetrate a soil specimen at a specified rate. The CBR is the load, in megapascals (pounds per square inch), required to force a piston into the soil a certain depth, expressed as a percentage of the load, in megapascals (pounds per square inch), required to force the piston the same depth into a standard sample of crushed stone. Usually depths of 2.5 or 5 mm (0.1 or 0.2 in.) are used, but depths of 7.5, 10 and 12.5 mm (0.3, 0.4, and 0.5 in.) may be used if desired. Penetration loads for the crushed stone have been standardized. The resulting bearing value is known as the California Bearing Ratio, which is generally abbreviated to CBR, with the percent omitted.

7.02 SCOPE.—This test method is intended to provide the relative bearing value, or CBR, of base, subbase, and subgrade materials. Procedures are given for laboratory-compacted swelling, nonswelling, and granular materials. These tests are performed usually to obtain information that will be used for design purposes

7.03 AUXILIARY SOIL TESTS.—There are certain routine soil tests that should be performed prior to conducting the CBR test. These tests are as follows:

	ASTM	AASHTO
Sieve Analysis of Fine and Coarse Aggregate	C 136	T 27
Liquid Limit of Soils	D 423	T 89
Plastic Limit and Plasticity Index of Soils	D 424	T 90
Particle Size Analysis of Soils (only for classification purposes)	D 422	T 88
Moisture-Density Relations of Soils using 5.5-lb (2.5-kg) Rammer	D 698	T 99
Moisture-Density Relations of Soils using 10-lb (4.5-kg) Rammer	D 1557	T 180

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B. DETERMINATION OF CBR FOR REMOLDED SPECIMENS

7.04 GENERAL.—The CBR value for a soil will depend upon its density, molding moisture content, and moisture content after soaking. Since the product of laboratory compaction should closely represent the results of field compaction, the first two of these variables must be carefully controlled during the preparation of laboratory samples for testing. Unless it can be ascertained that the soil being tested will *not* accumulate moisture and be affected by it in the field after construction, the CBR tests should be performed on soaked samples. As an example, there is considerable evidence that subgrades beneath Full-Depth asphalt pavements* do not accumulate enough moisture above the construction moisture content to affect their strengths adversely.

7.05 EQUIPMENT.—

1. *Loading Machine*—A loading machine with a capacity of at least 44.5 kN (10,000 lbf) and equipped with a movable head or base that travels at a uniform (not

*Full-Depth(R) asphalt pavement is one in which asphalt mixtures are employed for all courses above the subgrade or improved subgrade. A Full-Depth asphalt pavement is laid directly on the prepared subgrade.

pulsating) rate of 1.27 mm/min (0.05 in./min), for use in forcing the penetration piston into the specimen. The machine shall be equipped with a load indicating device that can be read to 44 N (10 lbf) or less.

2. *Mold*—A mold of metal, cylindrical shape with an inside diameter of 152.4 ± 0.13 mm (6 ± 0.005 in.). It shall be provided with a metal extension collar 51 mm (2.0 in.) in height, and a perforated metal base plate 9.5 mm (3/8 in.) in height. The perforations in the base plate shall not exceed 1.59 mm (1/16 in.) in diameter.

3. *Spacer Disc*—A circular metal spacer disc 150.8 mm (5 15/16 in.) in diameter and 61.4 mm (2.416 in.) in height.

4. *Rammer*—A metal rammer as specified in either ASTM Method D 698 or Method D 1557. Automatic rammers or the sliding weight rammer may be used, provided the compactive effort given is the same as that given by the comparable rammers described in Methods D 698 or D 1557.

5. *Expansion Measuring Apparatus*—An adjustable metal stem and perforated plate, with perforations in the plate not exceeding 1.59 mm (1/16 in.) in diameter, and a metal tripod to support the dial gauge for measuring the amount of swell during soaking. 85

6. *Weights*—One annular metal weight and several slotted metal weights weighing 2.27 kg (5.0 lb) each, 149.2 mm (5 7/8 in.) in diameter, with a center hole 54.0 mm (2 1/8 in.) in diameter.

7. *Penetration Piston*—A metal penetration piston 49.6 mm (1.95 in.) in diameter, 1935.5 mm^2 (3 in.^{2}) in area and not less than 102 mm (4 in.) long. If from an operational standpoint it is advantageous to use a piston of greater length, the longer piston may be used.

8. *Gauges*—Two dial gauges reading to 0.025 mm (0.001 in.).

9. *Miscellaneous*—Other general apparatus such as a mixing bowl, straight-edge, scales, soaking tank or pan, oven filter paper, and dishes.

7.06 SOIL PREPARATION.—

1. Prepare the soil sample [35 kg (75 lb) or more] in accordance with ASTM Method D 698 or D 1557 (AASHTO Method T 99 or T 180).

2. If the soil sample is damp when received from the field, dry it until it becomes friable under a trowel. Drying may be in air or by drying apparatus such that the temperature does not exceed 60°C (140°F).

3. Thoroughly break up aggregations in such a manner as to avoid reducing the natural size of individual particles.

4. For fine-grained soils, pass an adequate quantity of the pulverized soil through a 4.75 mm (No. 4) sieve and discard any coarse material retained on the sieve.

5. For granular soils, pass an adequate quantity of the pulverized soil through a 19 mm (3/4 in.) sieve and discard any coarse material retained on the sieve. If desirable to maintain the same percentage of coarse material [passing 50 mm (2 in.) sieve] as in the original field sample, replace the material retained on the 19 mm (3/4 in.) sieve as follows:

   Pass an adequate quantity of the pulverized soil through nested 50 mm (2 in.), 19 mm (3/4 in.), and 4.75 mm (No. 4) sieves.

   Discard the material retained on the 50 mm (2 in.) sieve.

   Replace the material passing the 50 mm (2 in.) sieve and retained on the 19 mm (3/4 in.) sieve with an equal weight of material passing the 19 mm (3/4 in.) sieve and retained on the 4.75 mm (No. 4) sieve.

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7.07 PROCEDURE FOR CONDUCTING THE COMPACTION CONTROL TEST.—

The compaction control test used with the California Bearing Ratio (CBR) test is <u>Moisture-Density Relations of Soils Using a 10 lb (4.54 kg) Rammer and an 18 in. (457 mm) Drop</u>, ASTM Designation D 1557, or <u>Moisture-Density Relations of Soils Using a 5.5 lb (2.5 kg) Rammer and 12 in. (304.8 mm) Drop</u>, ASTM Designation D 698. These tests are reproduced in Appendix A.

7.08 PREPARATION OF TEST SPECIMENS.—

1. Select a representative sample weighing at least 4.5 kg (10 lb) for fine-grained soils or 5.5 kg (12 lb) for granular soils, and mix thoroughly with water. The mixture may be cured by placing in a covered container until the moisture is uniformly distributed.

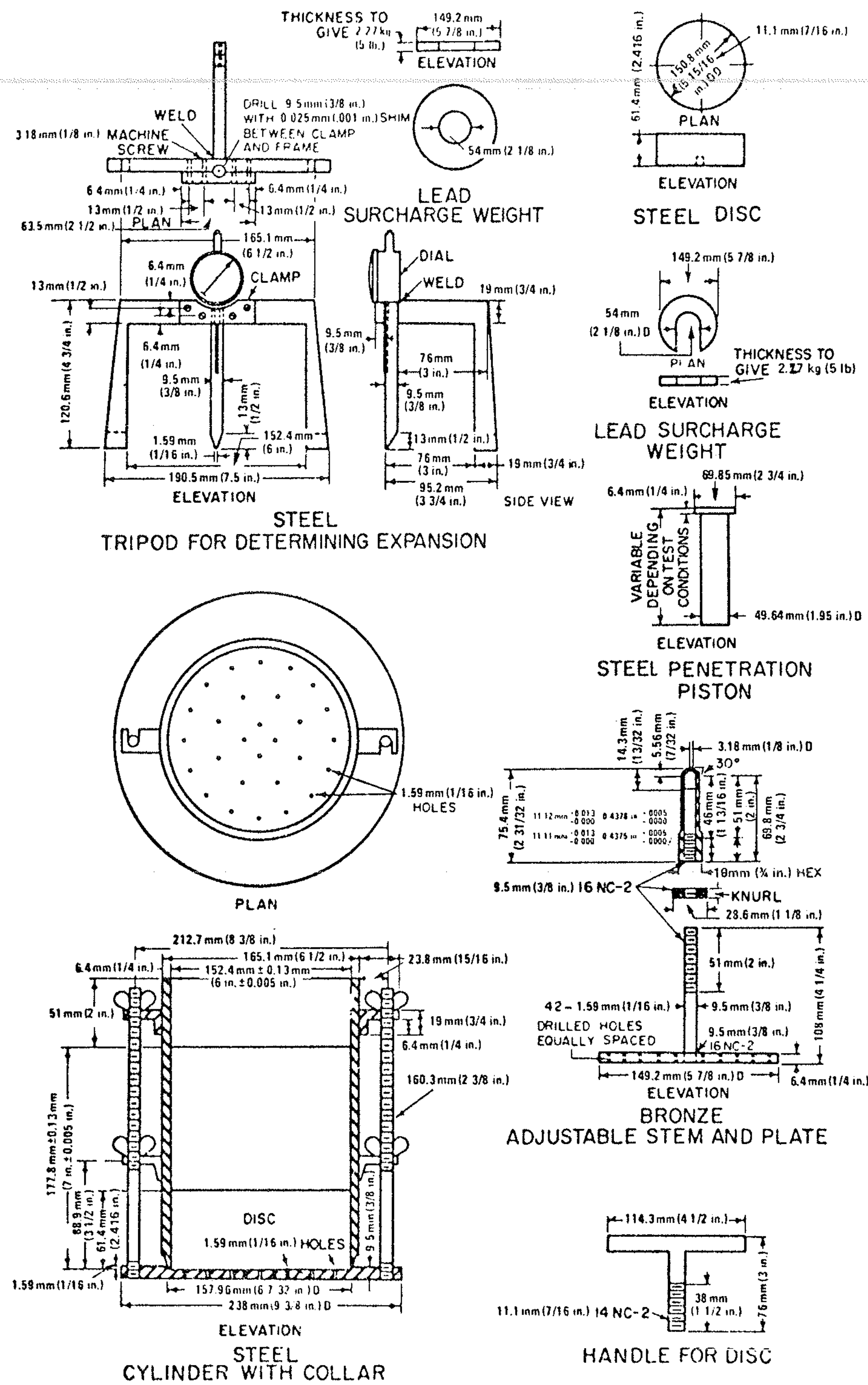
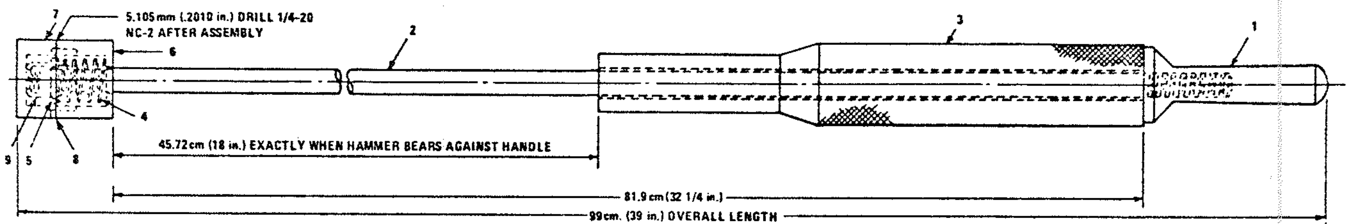
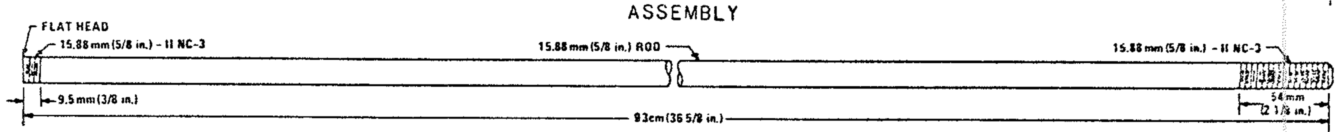


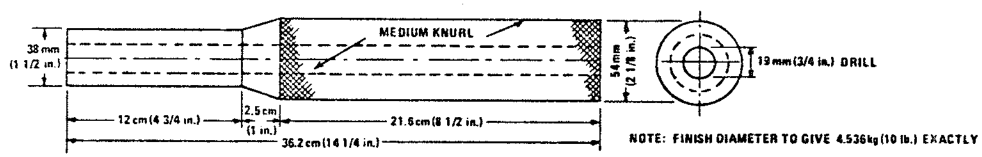
Figure VII-1. California bearing ratio test apparatus.



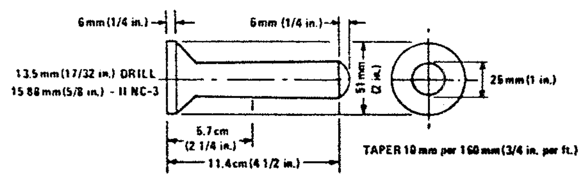
ASSEMBLY



ROD #2



DROP WEIGHT #3



HANDLE #1

NOTE: ALL PARTS TO BE FINISHED ALL OVER

LIST OF PARTS					
NO.	QT.	DESCRIPTION	MATERIAL	MASS	WEIGHT
1	1	HANDLE	STEEL	0.34kg	0.75lb
2	1	ROD	C.R.STEEL	1.45kg	3.20lb
3	1	DROP WEIGHT	STEEL	4.54kg	10.00lb
4	1	SPRING	N.D.C.STEEL	0.02kg	0.04lb
5	1	SPRING RETAINER	STEEL	0.05kg	0.12lb
6	1	UPPER PART OF HEAD	STEEL	0.27kg	0.59lb
7	1	LOWER PART OF HEAD PARTS NOT DETAILED:	STEEL	0.32kg	0.71lb
8	1	SET SCREW, SAFETY 6.35 x 3.18mm (1/4" x 1/8") LONG	STEEL	0.02kg	0.05lb
9	1	NUT, HEXAGONAL, 15.88mm (5/8") 11	STEEL	0.02kg	0.04lb
TOTAL WEIGHT				7.03kg	15.50lb

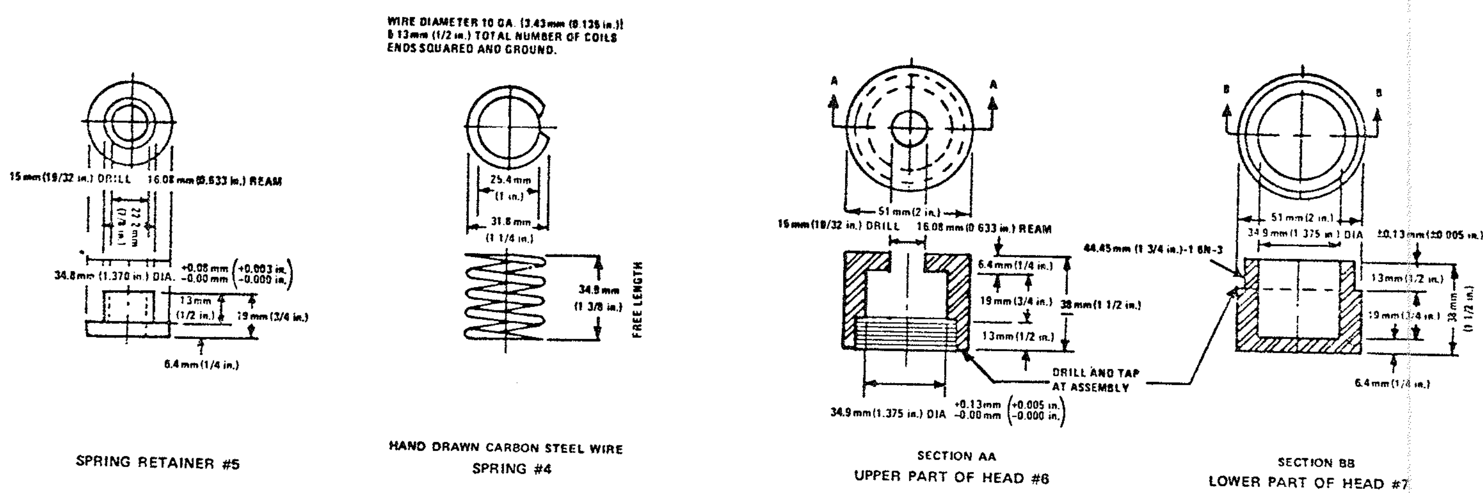


Figure VII-2. Compaction tamper; assembly and details.

2. If the specimen is to be soaked, obtain a sample of the soil—at least 100 g for fine-grained soils, 500 g for granular soils—for moisture determination at the beginning of compaction. Take another sample from the remaining material after compaction. Weigh the sample immediately, then dry it in an oven at 110 $\pm$ 5°C (230 $\pm$ 9°F) for at least 12 hours, or to constant weight.

3. If the specimen is not to be soaked, take a moisture content sample after penetration from one of the cut faces.

4. Assemble the 152.4 mm (6 in.) mold, extension collar, and perforated base plate by clamping the mold with fitted extension collar to the base plate.

5. Insert the spacer disc over the base plate, and place a 152.4 mm (6 in.) diameter coarse filter paper on top of the disc.

6. Compact sample using compacting efforts and molding water content as indicated by the moisture-density test (Method B or D of ASTM D 698 or ASTM D 1557).

7. After each sample has been compacted in the mold, remove the extension ring; strike off excess soil with a straightedge; remove the base plate; and extract the spacer disc.

8. Weigh the mold and compacted soil to determine the density of soil.

> NOTE: If the specimen is not to be soaked, omit Steps 9 through 18.

9. Place filter paper on the base plate; invert the cylinder so that the bottom during compaction is now on top; re-attach to the base plate; and place filter paper on top of soil in mold.

10. Place the perforated plate, with adjustable stem attached, on the filter paper.

11. Place surcharge weights on the perforated plate to produce an intensity of surcharge loading equal to the weight of the base material and pavement within $\pm$ 2.27 kg ($\pm$ 5 lb), but not less than 4.54 kg (10 lb).

12. Immerse the mold and weights in water to within 13 mm (1/2 in.) of the top of the mold. Place blocks under the mold to allow free access of water to the bottom of the specimen, and put water inside the mold to the same level as water on the outside of the mold.

13. After immersion, measure the height of the stem or spindle above the edge of the mold with the dial micrometer and tripod assembly. This is the initial measurement for swell.

14. Allow the specimen to soak for 96 hours (four days), maintaining constant water level outside and inside the mold.

15. Repeat step 13 to obtain the final swell measurement. Compute the swell as a percentage of the initial specimen height.

Example

Swell Data

Reading after 4 days	10.29 mm (0.405 in.)
Original reading	9.86 mm (0.388 in.)
Swell (difference)	0.43 mm (0.017 in.)

$$\text{Swell (\%)} = \frac{\text{swell}}{\text{ht. of specimen}} \times 100 = \frac{100 \times 0.43}{127} = 0.34\%$$

16. Remove the mold from the water, and pour off free water from inside the mold, being careful not to disturb the soil.

17. Remove the surcharge weights, perforated plate, and filter paper, and allow the specimen to drain for 15 minutes.

18. Weigh the specimen to determine the soil density. The specimen is then ready for the penetration test.

7.09 PROCEDURE FOR PENETRATION TESTING.—

> NOTE: This procedure is the same for all types of remolded specimens. Moreover, it is also applicable for undisturbed and field in-place tests after the testing surface has been prepared.

1. Place one 1.27 kg (5-lb) annular disc surcharge weight on the soil surface.

2. Place the mold in the loading frame or hydraulic press, and adjust its position until the piston is centered on the specimen.

3. Seat the penetration piston with a 44 N (10 lb) load, and set both the load dial and strain dials to zero. This initial load is required to ensure satisfactory seating of the piston and should be considered as the zero load when determining stress-penetration relations.

4. Add penetration surcharge weights to produce an intensity of loading equal to the weight of the base material and pavement [within $\pm$ 2.27 kg ($\pm$ 5 lb)], but not less than 4.54 kg (10 lb). If the sample has been previously soaked, the surcharge should be equal to the soaking surcharge.

5. Apply the load to the piston at a uniform rate of 1.27 mm (0.05 in.) of penetration per minute.

6. Record the total load readings at the following penetrations:

0.6 mm	(0.025 in.)
1.3 mm	(0.050 in.)
1.9 mm	(0.075 in.)
2.5 mm	(0.100 in.)
3.2 mm	(0.125 in.)
3.8 mm	(0.150 in.)
4.4 mm	(0.175 in.)
5. mm	(0.200 in.)
6.4 mm	(0.250 in.)
7.5 mm	(0.300 in.)
10. mm	(0.400 in.)
12.5 mm	(0.500 in.)

[The load readings at penetrations of 10 mm (0.400 in.) and 12.5 mm (0.500 in.) may be omitted.]

7. Release the load; remove the mold from the loading device; remove the weights; and detach the base plate.

8. Determine moisture content of the top 25 mm (1 in.) layer. Take a moisture content sample from the entire depth if average moisture content is desired.

9. From the loads obtained in 6, the CBR of the sample is determined, as illustrated in Section C.

C. CALCULATIONS

7.10 STRESS-PENETRATION CURVE.—After the test has been completed, the penetration load in megapascals (psi) is calculated and the load-penetration curve plotted on cross-section paper. In order to obtain true penetration loads from the test data, the zero point of the curve is adjusted to correct for surface irregularities and the initial concave upward shape of the curve if it is present. If the curve is uniform as in example No. 1 of Figure VII-3, the CBR value is calculated from the recorded loads. For surface irregularities as in example No. 3 of Figure VII-3 extend the straight line portion of the curve to the base to obtain a corrected origin, or zero. If the curve has a reverse bend, or concave upward shape, as in example No. 2, draw a line tangent to the steepest point of the curve (point A), and extend the line to the base to obtain a corrected origin or zero point (point B). Then read the corrected load values for 2.5 mm (0.1-in.) penetration (point C) and 5. mm (0.2-in.) penetration (point D).

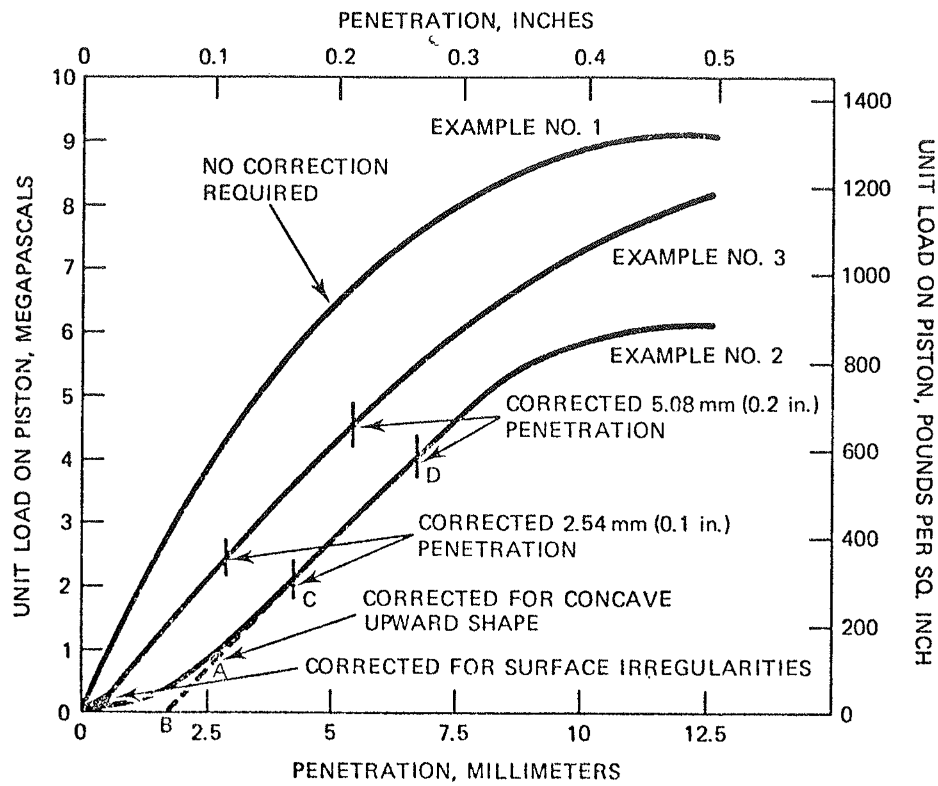


Figure VII-3. Correction of load-penetration curves.

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7.11 CALCULATION OF CALIFORNIA BEARING RATIO.—The CBR value is defined as a ratio comparing the bearing of a material with the bearing of a well-graded crushed stone. The penetration loads for crushed stone are presented in the following table:

Penetration mm	(in.)	Standard Load N	(lb)	Standard* Load MPa	(psi)
2.5	(0.1)	13345	(3000)	6.89	(1000)
5.	(0.2)	20017	(4500)	10.34	(1500)
7.5	(0.3)	25355	(5700)	13.10	(1900)
10.	(0.4)	30693	(6900)	15.86	(2300)
12.5	(0.5)	34696	(7800)	17.93	(2600)

*Plunger cross-section area = 1935.5 mm^2 (3 $in.^2$).

The CBR is determined from the corrected load values at 2.5 and 5 mm (0.1 and 0.2 in.) penetrations by dividing the loads at 2.5 and 5 mm (0.1 and 0.2 in.) by the standard loads of 6.89 and 10.34 MPa (1000 and 1500 psi), respectively. Each ratio is multiplied by 100 to obtain the CBR in percent. The CBR is usually selected at 2.5 mm (0.1 in.) penetration. If the CBR at 5 mm (0.2 in.) penetration is greater, the test should be rerun. If check tests give similar results, the CBR at 5 mm (0.2 in.) penetration should be used.

CALIFORNIA BEARING RATIO TEST DATA SHEET

Molding Date ______________ Penetration Date ______________

Project ______________________________

Sample No. ______________ Compacted at ______________

A. Weight of Compacted Sample, Mold and Base Plate; kg ______________
B. Tare Weight of Mold, and Base Plate; kg ______________
C. Weight of Sample; kg ______________
D. Height of Compacted Sample; mm ______________
E. Volume of Sample; cm^3 ______________
F. Unit Wet Weight; kg/cm^3 ______________
G. Moisture Content; % ______________
H. Unit Dry Weight; kg/cm^3 ______________

Expansion and Consolidation Data

N. Surcharge Weight ______________ kg R. % Expansion or Cons. ______________
O. Dial Reading at Start ______________ mm
P. Dial Reading at Finish ______________ mm $R = \frac{Q}{D} \times 100$
Q. Difference ______________ mm
S. Weight of Sample, Mold and Base Plate after Saturation; kg ______________

Moisture Samples

Top 25 mm		Bulk of Sample	
Pan No. ________		Pan No. ________	
Wet Wt. ________ g	Dry Wt. ________ g	Wet Wt. ________ g	Dry Wt. ________ g
Dry Wt. ________ g	Tare Wt. ________ g	Dry Wt. ________ g	Tare Wt. ________ g
Percent Moisture ________		Percent Moisture ________	

T. Sum of Net Wet Weights ________ + ________ = ________
U. Sum of Net Dry Weights ________ + ________ = ________

Penetration Data

Pen.	Load, kN (lb)	Load, MPa (psi)
0.6 mm (.025 in.):	:	
1.3 mm (.050 in.):	:	
1.9 mm (.075 in.):	:	
2.5 mm (.100 in.):	:	
3.2 mm (0.125 in.)	:	
3.8 mm (.150 in.):	:	
4.4 mm (0.175 in.):	:	
5. mm (.200 in.)	:	
6.4 mm (0.250 in.)	:	
7.5 mm (.300 in.)	:	
10. mm (.400 in.):	:	
12.5 mm (.500 in.):	:	

Corrected C.B.R.

Pen.	Corr.Load MPa(psi)	Std. MPa (psi)	C.B.R.
2.5 mm (.100 in.)	______	6.89 (1000)	______
5. mm (.200 in.)	______	10.34 (1500)	______
7.5 mm (.300 in.)	______	13.10 (1900)	______
10. mm (.400 in.)	______	15.86 (2300)	______
12.5 mm (.500 in.)	______	17.93 (2600)	______

NOTE: $C = A-B$ $F = \frac{C}{E}$

$G = \frac{C-U}{U} \times 100$ $H = \frac{F}{100 + G} \times 100$

Figure VII-4. Sample form for recording CBR test information.

96 A 10-ton diesel roller compacts hand-broken stone base for Kabupatan road in central Java (photo taken by Scott Wilson Kirkpatrick and Partners, United Kingdom, for the World Bank).

AUSTRALIAN ROAD RESEARCH

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Australian Road Research

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CONTENTS

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MECHANICAL DURABILITY OF LATERITIC GRAVELS FROM SOUTHEAST ASIA; SUGGESTED TESTS AND TEST STANDARDS FOR HIGHWAY USES

An investigation has been conducted to determine the test(s) potentially most suitable for evaluating the mechanical durability of lateritic gravels for use in road construction. The relative durability of these materials was also investigated and compared with past performance and durability of lateritic gravels in existing pavement sections in Thailand. The probable range of durability as well as durability test techniques and tentative test standards for these materials, when used in pavement sections, were established.

INTRODUCTION

1. In the last decade many countries in the tropical regions of the world have undertaken extensive road construction programmes. Laterite and lateritic gravel are commonly used construction materials in many of these countries. Little is known of the performance of these lateritic materials in modern pavement sections subjected to heavy traffic. The applicability of European and United States design and construction standards for use with lateritic aggregates has been questioned, as these standards were, by and large, developed empirically in temperate climates using non-lateritic materials.

2. One of the characteristics which govern the performance of an aggregate in a highway base course is its mechanical durability or ability to resist degradation when subjected to repeated traffic loading. Because of the petrologically heterogeneous nature of most lateritic gravels it is felt that existing tests and standards for mechanical durability should be examined to determine their suitability for use with lateritic aggregates.

3. This paper is based on an investigation which was undertaken as part of a major research project on lateritic materials. This project was jointly sponsored by the Government of Thailand and the United States Agency for International Development (Ref. 1). The purpose of this investigation was to examine the subject of durability of lateritic gravel aggregates; particularly to determine tests and test standards which may be used to insure adequate mechanical soundness of the aggregate when used in a highway surface or base course.

TEST AGGREGATES

DEFINITION AND DESCRIPTION OF MATERIALS

4. For the purpose of this paper, and to ensure consistency in the interpretation of the results presented herein, laterite and associated lateritic materials are defined as follows (Ref. 1).

Laterite — a *hardened* material formed by the primary (i.e., sesquioxides present in parent material) weathering of residual soils; or by the secondary (i.e., sesquioxide supplied by lateral ground water movement) enrichment and cementation of transported or residual soils.

Plinthite — a *non-hardened or poorly hardened* material formed by incomplete laterization of soils, similar in chemical and mineral composition to laterite.

Lateritic soil — soil of any physical composition, in which the majority, by weight, of the sample is composed of hardened laterite of any form.

All of the lateritic materials defined above occur in widely varying climatic and geomorphological conditions. Their origins and characteristics have been discussed in detail in the literature. Ref. 1 contains a complete presentation and bibliography on this subject.

5. The term 'lateritic gravel' as used in the present text refers to a lateritic soil in which the laterite occurs as unconsolidated concre-

tions in a soil matrix. The concretions may be either a result of residual weathering or secondary enrichment and cementation of transported soils. In general the latter materials are more physically and chemically heterogeneous.

6. Lateritic gravel deposits observed throughout Southeast Asia frequently varied from half to one meter in thickness and were overlain by as much as two meters of top soil. The profiles frequently became clayier with depth. The deposits were in general very erratic in both thickness and uniformity; thus leading to the widespread construction practice of stockpiling and blending prior to use.

SELECTION OF TEST MATERIALS

7. In order to make the results of this study most useful and widely applicable, careful consideration was given to the selection of materials. Since the principal uses of lateritic soils in Southeast Asia are in the construction of roads and airfields, the potential use of a material for this purpose was the major criteria for selection. Materials were selected to cover the range of uses in such construction from select fill (poor quality) to aggregate bases (best quality).

8. Before making the selection, over one hundred lateritic gravel sources were tested from throughout Thailand to locate sources of materials which provided the widest possible range of resistance to degradation by abrasion, wetting, and drying, compaction and alkaline effects. Seven sources were selected with apparent durability characteristics ranging from very good to very poor. Large samples were obtained from these sources for testing during the main test programme. A good petrologically homogeneous limestone was also obtained for simultaneous control testing. TABLE I gives general engineering data for the eight test materials. In the remainder of the discussion, these materials will be referred to by the Arabic numeral corresponding to those given for the materials in TABLE I.

TEST PROCEDURES AND TESTS

9. The test programme reported herein was composed of three separate activities.

(a) The evaluation of various accepted mechanical durability tests with respect to their suitability for use with lateritic gravels.

(b) A comparison between the effects of repeated mechanical stress, under laboratory conditions, and the results of the various durability tests on the same materials.

(c) A comparison between the observed long-term performance of over 100 base and subbase aggregates, beneath flexible pavements in Thailand, and the results of the various durability tests on the same material.

The following paragraphs will describe each of these activities in more detail.

EVALUATION OF MECHANICAL DURABILITY TEST PROCEDURES

10. The Los Angeles Rattler test (LAR) (A.S.T.M. C131-64T) and the California Coarse (D_c) and Fine (D_f) Durability tests* (Calif. 229-C) were studied to determine their suitability for evaluating the mechanical durability of lateritic gravel aggregates. For the purposes of this investigation a suitable test was defined as one which would provide results that apparently correlated well with the resistance of an aggregate to degradation under repeated mechanical stresses such as those imposed by traffic.

11. To be useful a test must not only measure the correct engineering properties of the material but it must measure them under conditions compatible with the actual field application. Therefore variables involving sample preparation and composition were believed to be important in evaluating the suitability of the above tests for use with lateritic gravels.

12. Because of the nature of their formation concretions in lateritic gravels are frequently petrologically heterogeneous. This is evident particularly from the differences in properties between the various concretion sizes within the same material, with the particles in the coarser fractions being denser and apparently more durable than the particles in the finer fractions. In some soils, ferruginous concretions may be essentially absent in fractions smaller than about the No. 30 sieve; the finer fractions be-

*The coarse and fine durability test procedure 229-C is available from the State of California Department of Public Works, Division of Highways, Sacramento, California, U.S.A.

TABLE I

GENERAL ENGINEERING PROPERTIES OF LATERITIC TEST GRAVELS

	Aggregate Numbers									
U.S. Sieve No	1	2	3	4	5	6	7	8	44	47B
¾ in.	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	—	95
⅜ in.	89.4	84.1	76.7	77.5	92.0	83.0	76.9	78.4	—	80
No. 4	59.9	63.7	57.1	61.1	64.9	62.9	56.1	55.3	—	65
No. 8	41.2	50.7	48.1	55.0	42.9	55.6	34.3	45.8	—	55
No. 16	36.1	46.6	44.2	50.8	37.8	52.7	25.4	26.8	—	49
No. 30	34.5	44.8	42.2	50.8	36.7	52.0	24.7	10.0	—	46
No. 50	33.3	43.3	38.0	47.4	35.8	50.8	24.4	7.4	—	41
No. 100	30.7	41.0	30.5	38.9	34.4	45.0	23.2	6.2	—	31
No. 200	24.1	35.3	23.2	27.8	31.1	24.9	13.5	5.4	—	22
0.010 mm	25.4	28.5	9.7	21.0	19.2	9.9	4.8	—	—	15
0.005 mm	23.1	27.0	8.1	20.1	13.3	8.7	4.4	—	—	14
0.001 mm	21.9	22.5	7.0	17.5	13.1	7.3	3.6	—	—	12
LL	55.0	77.0	24.9	39.1	43.0	20.6	18.5	NP*	42.0	27.0
PL	44.0	38.2	14.3	25.8	27.0	18.7	16.0	NP	25.0	22.0
SL	22.0	20.0	11.0	16.0	14.0	17.0	11.0	NP	19.0	14.0
PI	11.0	38.8	10.6	13.3	16.0	1.9	2.5	NP	17.0	8.0
BLS	6.3	11.0	2.4	5.5	10.0	1.6	2.4	NP	11.0	9.5
Sand Equivalents and Aggregation Index (Ref. 2)										
SE	10.0	7.0	9.1	5.8	10.9	12.2	15.1	—	—	18.7
AI	1.48	—	—	—	1.23	1.04	—	—	—	—
Apparent Specific Gravity and Absorption										
G	3.04	3.14	2.92	2.79	2.88	2.83	3.09	—	—	—
Abs. (%)	10.15	4.01	2.94	4.71	7.30	2.01	4.25	—	—	—

* NP = non-plastic

TABLE II

ARTIFICIAL SAMPLE GRADATIONS USED IN DURABILITY STUDIES

Key Word(s)	¾ in. x ⅜ in.	⅜ in. x No. 4	No. 4 x No. 8
Very coarse	100.0	0.0	0.0
Coarse	80.0	10.0	10.0
Intermediate	0.0	100.0	0.0
Well	33.3	33.3	33.3
Fine	10.0	10.0	80.0
Very fine	0.0	0.0	100.0
Standard B*	100.0	0.0	0.0
Standard C*	0.0	100.0	0.0
Standard D_c*	77.5	22.5	0.0

* The Standard B and C gradings refer to the A.S.T.M. Standards of the same letters for the LAR test. The Standard D_c refers to the unbiased California Standard Grading for the D_c tests.

ing composed predominantly of siliceous minerals. In addition to the predominance of indurated ferruginous concretions, which form the basis for calling the material lateritic gravel, sound quartzitic river gravels or other materials may be present in varying amounts in some deposits. When present in significant amounts these siliceous materials may have a marked influence on the properties of the soil.

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13. Observations of present road construction practices with lateritic gravels throughout Southeast Asia have indicated that the materials are usually used or rejected on the basis of tests on pit-run or 'naturally graded' samples. Road construction programmes may not provide sufficient funds or time to allow processing of the material to improve its quality.

14. The fact that lateritic gravels are frequently petrologically heterogeneous and used in the in situ or 'naturally graded' condition led to the decision to compare the standard test procedures and the effects of the two variables, gradation and petrologic composition, on the LAR and D_c test results.

15. Aggregates 1, 2, 6, and 8 were tested in a wide range of gradations in order to evaluate the sensitivity of the tests to gradation. These different aggregates were used in order to cover the range of results which may be associated with lateritic gravels of widely varying quality. The gradations selected represent the extreme range of possible gradations, as well as the standard test grading, and the natural (pit run) grading. The artificial gradings used are presented in TABLE II, and the natural gradings in TABLE III. All tests were run according to standard procedures with the exception of the LAR test on samples with non-standard gradings in which 10 steel balls were used regardless of grading.

16. In order to evaluate the sensitivity of the LAR and the D_c test results to variations in petrologic composition at constant grading, a very weak, soft laterite (aggregate 4) was

TABLE III

NATURAL SAMPLE GRADATIONS USED IN DURABILITY STUDIES

	Aggregate Number							
Sieve Fraction	1	2	3	4	5	6	7	8
¾ in. x ⅜ in.	18.0	32.2	44.8	50.0	14.0	38.3	35.1	39.8
⅜ in. x No. 4	50.2	41.4	37.8	36.4	47.5	45.3	31.7	42.6
No. 4 x No. 8	31.8	26.4	17.4	13.6	38.5	16.4	33.2	17.6

combined with a hard durable crushed limestone (aggregate 8) in various proportions. All of the samples were graded with the natural grading of aggregate 4 (TABLE III). The proportion of the two materials present in any given sample was constant in all fractions. The per cent of each aggregate present in each of the five samples was as follows:

Aggregate 8	100	90	50	10	0
Aggregate 4	0	10	50	90	100

17. The D_f test was run by the standard procedure, with the exception that the samples were air dried at 60 to 70°C instead of oven dried at 105°C. This was done to preclude the possible irreversible dehydration of halloysite ($4H_2O$) to halloysite ($2H_2O$), should it be present in the sample. This mineral is included in the kaolinite clay mineral group which composes most of the clay minerals in the lateritic gravels observed. The change from halloysite ($4H_2O$) to halloysite ($2H_2O$) is known to affect the engineering properties of the clay.

Coarse durability test

18. Briefly, the standard D_c test procedure requires that the aggregate larger than the No. 4 sieve be washed clean of all fines. The washed coarse sample is then mechanically shaken in a container, partially filled with distilled water, for 10 min. During this agitation period the particles abrade and fines are produced. The agitation water and fines are then put into a graduate and allowed to settle under controlled conditions for 20 min. The level of the interface between the clear fluid and the settled fines is noted after the 20 min settling period. The higher this level the lower the D_c test result.

19. For materials such as limestone, which are generally petrologically homogeneous throughout all particle sizes, the gradation of the sample apparently has little or no effect on D_c test results. However, with heterogeneous materials such as lateritic gravels, the effect of changing the grading is essentially that of changing the materials petrologic composition. Changing both variables simultaneously produces widely varying results. The D_c test results are summarized in TABLE IV. The difference between results for the four materials is indicative of the range of variability which may be expected in lateritic gravels from Thailand. *Fig. 1* indicates typical results of varying composition at constant gradation. It appears that the D_c test is very sensitive to the presence of poor materials, even in relatively small quantities. Such small quantities of material could cause the failure of an otherwise good base course if the poor material degrades to silts and clays while in service.

Los Angeles rattler tests

20. The LAR test is a well known and accepted procedure; hence a detailed discussion of the test procedure will not be presented. With the exception of the previously mentioned use of 10 steel balls for all non-standard gradings, all tests were run according to the standard LAR test procedure. The LAR test appears to be slightly more affected by grading

TABLE IV

COARSE DURABILITY (D_c) TEST RESULTS FOR VARIOUS GRADATIONS

Grading Key Word(s)	Aggregate Number 1	2	6	8
Very Coarse	24.0	24.0	56.5	80.0
Coarse	12.0	24.0	66.5	82.0
Intermediate	11.0	12.0	53.0	80.0
Well	10.5	11.0	61.5	80.0
Fine	10.0	7.0	50.0	80.0
Very Fine	6.0	7.0	43.0	80.0
Natural	9.0	11.5	64.0	81.0
Standard	12.5	22.0	69.0	82.5

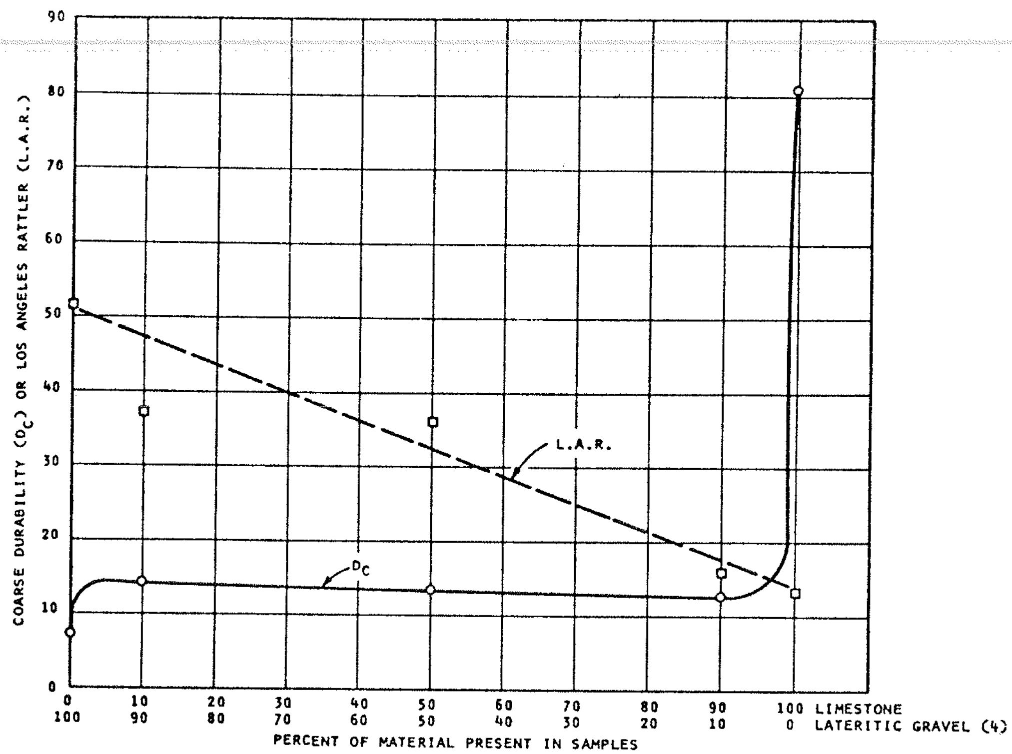

Fig. 1 — Affect of Petrologic Composition on D_c and Los Angeles Rattler Test Results

than does the D_c test. It is suggested that the cushioning effect of more fine material may result in a reduction in the percentage loss. This could account for the apparently conflicting results of the D_c and LAR tests, indicated by a comparison of the values obtained for the 'very fine' grading on aggregate number 1 (TABLE IV and TABLE V). Visual comparison of the materials tends to substantiate the evaluation of quality given by the D_c test results.

21. The data presented generally represent an average of the results of two or more tests. Normal experimental deviation could account for observed variations in both the D_c and LAR tests performed on aggregate 8 (control limestone) at all gradations. The results given in TABLE V indicate the variability which may be expected with the LAR test when run with 10 steel balls regardless of grading. *Fig. 2* illustrates the results obtained with 11 and 8 ball test charges, respectively, which tend to bracket the non-standard, natural graded, results with a 10-ball test charge.

22. The relative sensitivity of the LAR and D_c tests to petrologic composition is shown in *Fig. 1.* A major difference between the D_c and LAR tests appears to be that the LAR test does not evaluate the nature of the fines produced during disintegration of the sample. As a hypothetical example, a shale and a coarse sandstone may give similar LAR results; however, the nature of the fines produced during disintegration is very different. If incorporated in a pavement, these materials might yield considerably different performance under similar conditions. The inservice degradation of the shale to silts and clays would have a more detrimental effect than would the degradation of the coarse sandstone to sand. An aggregate's tendency to produce silt or clay when abraded in the presence of water is evaluated in the D_c test, but not in the LAR test.

TABLE V

LOS ANGELES RATTLER (LAR) TEST RESULTS FOR VARIOUS GRADATIONS

Grading Key Word(s)	Aggregate Number			
	1	2	6	8
Very Coarse	55.0	22.0	31.0	18.0
Coarse	56.0	24.0	34.0	23.0
Intermediate	47.0	27.0	28.0	20.0
Well	51.0	30.0	31.0	23.0
Fine	47.0	33.0	29.0	23.0
Very Fine	38.0	29.0	22.0	18.0
Standard B	59.0	25.0	38.0	22.0
Natural	55.0	28.0	30.0	22.0
Standard C	41.0	22.0	23.0	18.0

D_c AND D_f RESULTS VERSUS EFFECTS OF REPEATED STRESS

23. In order to evaluate the relationship between the D_c and D_f test results and the effect of cyclic dynamic loads, such as those imposed on pavements by traffic, on lateritic gravel it was felt desirable to perform a series of controlled cyclic triaxial tests on several of the test aggregates with widely varying properties. This test series was performed on the follow-

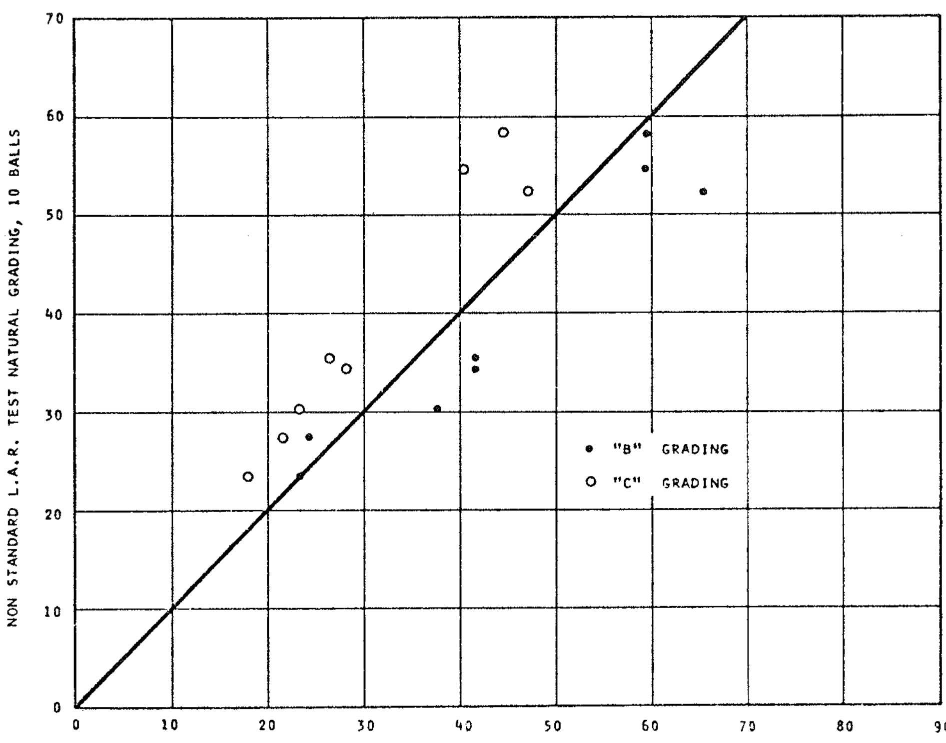

Fig. 2 — Standard vs Non Standard Los Angeles Rattler Tests on the same Material

ing four lateritic aggregates: LR47B (similar to aggregate 4), LR44, 1 and 6. For control purposes all tests were repeated on the control limestone aggregate number 8. All samples were graded identically (see TABLE VI) prior to compaction by the special non-destructive procedure described below. In addition to grading control, the same material (aggregate 6) was used in every sample for the passing No. 8 fines regardless of the coarse aggregate used in the sample. This care in preparation of the samples was intended to ensure minimum variations in the grading and plasticity of the samples, as well as eliminating the effects of the fine material as a variable in evaluating the test results.

TABLE VI

INITIAL GRADING OF ALL ARTIFICIALLY GRADED* SAMPLES

Sieve No.	Percent Passing by Dry Weight
¾ in.	100.0
⅜ in.	67.2 - 67.5
No. 4	45.7 - 46.4
No. 8	31.3 - 32.2
No. 16	22.2 - 23.4
No. 30	20.8 - 22.0
No. 50	20.5 - 21.6
No. 100	19.7 - 20.8
No. 200	16.8 - 17.4

* These samples were open graded with the sand sizes between about the No. 10 and No. 200 sieves missing. This is typical of many naturally occurring lateritic gravels.

24. In addition to the samples prepared for triaxial testing (two samples of each aggregate), other samples were prepared identically for washed sieve analysis and coarse and fine durability tests (D_c and D_f). When the tests did not require all of the sieve fractions, the material used was proportioned identically to the grading.

25. All of the samples for triaxial testing were compacted dry, in very thin lifts, by vibration under a slight vacuum. This elaborate compaction procedure was followed for three reasons.

(a) To avoid change in grading of the samples due to destruction of the aggregate during compaction.

(b) To ensure uniform samples and avoid segregation of the fines in any part of the sample.

(c) To ensure, as nearly as possible, identical relative densities. 'Relative densities' are not the same as comparative dry densities which, among other things, are a function of the absorption and specific gravity of the samples. These properties for the materials tested are highly variable as can be seen in TABLE I. The absolute value of the relative density was not obtained; however, it was estimated to be about 60 per cent.

26. After compaction, the triaxial specimens were saturated prior to testing. One of the two samples prepared for each aggregate was tested to determine M_r*. The other sample was tested to failure in consolidated, drained triaxial shear with a confining pressure of 5 p.s.i. After completion of 5000 cycles of loading, at a rate of 30 c/min, the M_r specimen was also tested to failure in a similar manner. A washed sieve analysis test was conducted on the M_r specimen after testing.

27. All of the M_r tests were conducted with a major principal stress of 12 p.s.i. and a principal stress ratio of 2.4 (confining pressure of 5 p.s.i.). The coarse porous materials were cycled in a saturated condition with drains open to avoid the build-up of excess pore water pressures. After failure, these samples were tested by washed sieve analysis to determine if any changes in grading had occurred during cycling. The M_r and strength data are presented in TABLES VII and VIII and *Fig. 3* and *4*. The modified fineness modulus referred to in TABLE VII and *Fig. 4* is the standard fineness modulus, as defined in A.S.T.M. C-125, modified to include the No. 200 sieve. A decrease in the fineness modulus indicates an overall increase in fines. It was used here in lieu of a direct comparison of sieve analysis

*The M_r is the modulus of resilience defined as the primary stress impulse divided by the recoverable strain which it induces. Reference 3 contains a complete discussion of the M_r test and its interpretation.

TABLE VII

EFFECT OF CYCLIC LOADING ON MODULUS OF RESILIENCE (M_r) AND SAMPLE DEGRADATION

Sample	Modulus of Resilience (k.s.i.) Determined After Cycles					D_f	D_c	Change in Modified Fineness Modulus, No Units
	1	10	100	1000	5000			
8	9.3	13.0	14.9	16.3	16.7	48	90	—0.005
44	8.4	10.7	11.3	11.8	12.3	35	—	—0.026
6	9.3	9.7	11.5	12.3	12.7	35	50	—0.021
47B*	6.1	—	7.7	9.2	10.6	38	15	—0.210
1	13.0	13.0	14.9	14.9	16.3	23	20	—0.262

* This was the first sample tested and several items must be considered with respect to the data reported for sample 47B. Despite the variable quartz content which complicates analysis, this material was tested because it was typical of much of the weakly cemented perched water laterite found in coarse sand or gravel alluvium.

(a) During cyclic loading, a principal stress ratio of 3.0 was used initially. This was reduced to 2.4 within 20 cycles, however, as the sample was straining badly.

(b) This material is petrographically heterogeneous. The fine sample fractions (generally smaller than the No. 4 sieve) contain considerable amounts of quartz which are not present in the coarser fractions — this results in a deceptively high fine durability result.

(c) The coarse durability reported for this material was obtained at a slightly different grading than that reported for the other samples.

results because 5000 cycles of low stress produces little disintegration of most materials which are competent enough to be called aggregates and the small consistent grading changes which occur would not be readily apparent in the gradation curves.

D_c AND D_f RESULTS VERSUS OBSERVED FIELD PERFORMANCE

28. Objective measurements relating the performance of an aggregate, beneath a flexible pavement, to mechanical durability alone are difficult to obtain and of dubious reliability. The number, type, and thickness of layers in the pavement section, its age, the level of traffic and the construction procedures used all influence the apparent condition of an aggregate at any given time. However, by selecting sections of highway for which all of the above variables except material type are relatively constant it is believed possible to determine general behavioural trends. Such a study was conducted during the present investigation.

29. Over 100 test sections were selected and excavated in highways located throughout Thailand. The history of each section was known. At the time of excavation, pavement condition surveys were made and each aggregate and soil layer in the pavement section was examined and sampled for laboratory testing. This test programme was conducted principally to develop material specifications, and check existing thickness design standards and is reported in detail in Ref. 1. Only those general comments pertinent to durability are included here. 107

30. Aggregates with D_c and D_f values in excess of about 40 did not exhibit any appreciable signs of disintegration or weakening when examined in situ beneath the pavement prior to excavation for testing. In contrast, some aggregates with D_c values in the 20's were observed to have almost completely disintegrated under similar conditions. Frequently the larger aggregate particles were badly fractured and disintegrated when disturbed.

31. Of all the test sections, those which provided the most useful data were along a 400 km section of highway constructed, in general, of 3 cm of double penetration surface course over 20 cm of lateric gravel base

TABLE VIII

STRESS-STRAIN RELATIONS BEFORE AND AFTER LOADING

			Precyclic Conditions at Ultimate Strength		Postcyclic Conditions at Ultimate Strength	
Sample	Friction Angle (ϕ)	Dry Density, (p.c.f.)	Stress (p.s.f.)	Strain (%)	Stress (p.s.f.)	Strain (%)
8	41	116	2640	10.9	2640	6.7
44	39	113	2420	14.1	2520	11.4
6	38	118	2220	8.4	2260	8.0
47B	37	98	2120	12.5	2420	4.9
1	37	94	2060	11.2	2120	7.1

founded on a dense silty quartzitic sand subgrade. The cumulative traffic over these sections during the lifetime of the pavement had been equivalent to a design traffic number (DTN) between 85 and 150.

32. The California Highway Department, the agency which developed the D_c and D_f tests, has conducted extensive carefully documented studies of the relation between durability test results and performance for a wide range of non-lateritic aggregates. Their conclusions, and resulting specifications, allow the use of materials with D_c and D_f values greater than 35 in untreated aggregate base courses. The limited field observations made during the investigation reported herein agree with the California Standards and suggest that until additional data become available these standards be accepted for use with lateritic as well as non-lateritic aggregates.

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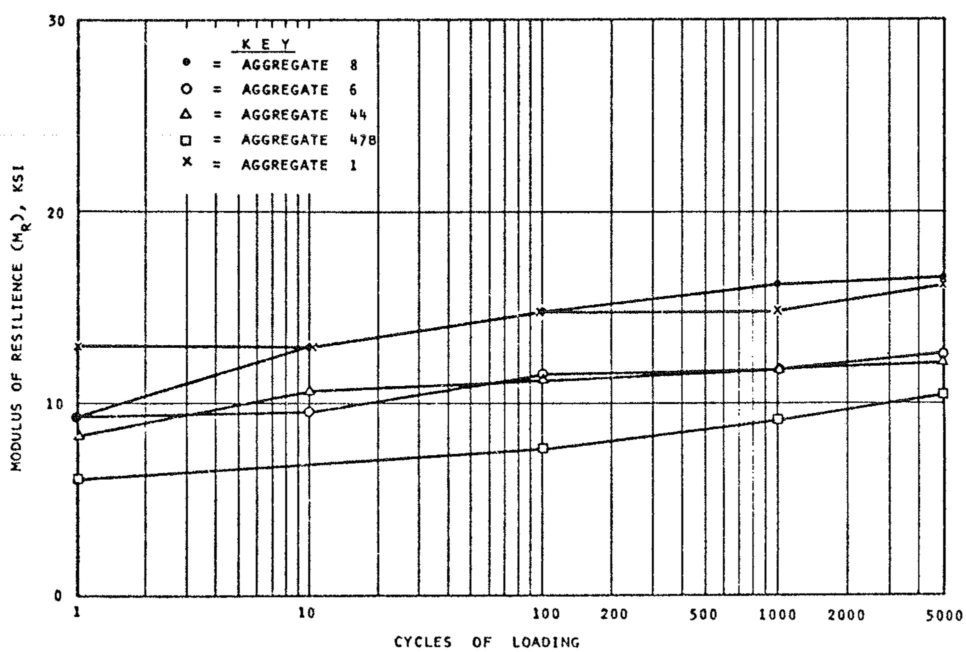

Fig. 3 — Platen to Platen Modulus of Resilience for Saturated Artificially Graded Samples

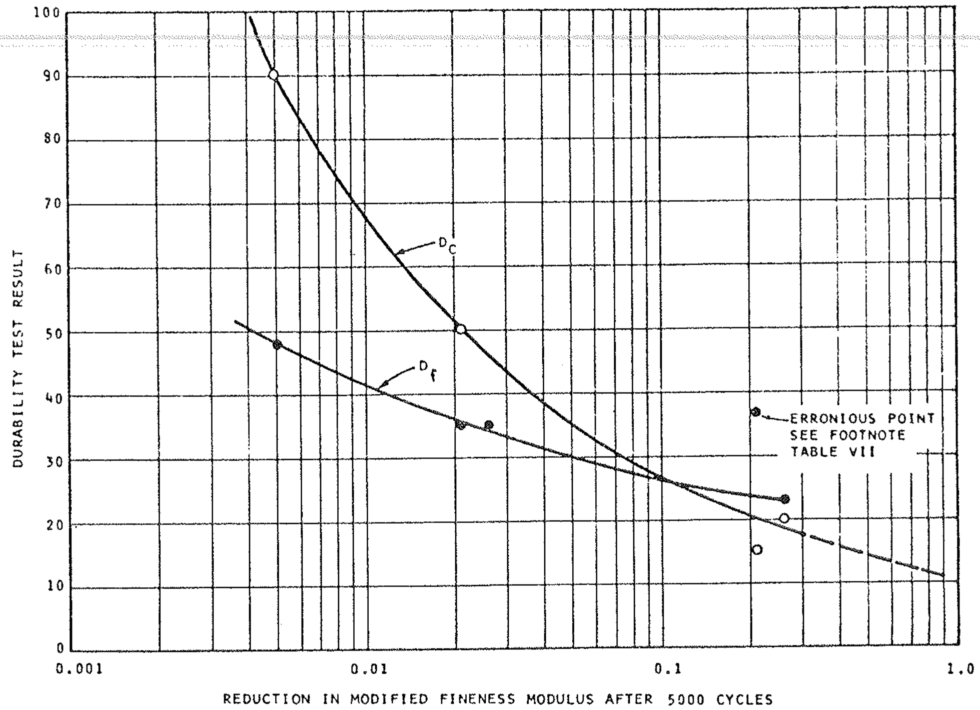

Fig. 4 — California Durability vs Change in Modified Fineness Modulus after 5000 Cycles of 12 p.s.i. Stress at a Principal Stress Ratio of 2.4

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS AND RECOMMENDATIONS REGARDING TEST METHODS

33. All test procedures investigated appear to be generally suitable for evaluating the mechanical durability of lateritic materials. However, the D_c and D_f tests appear to be better for the evaluation of aggregates, for use beneath flexible pavements, than the LAR test. The following modifications of the standard procedures are recommended in order to obtain more meaningful test results on lateritic gravel aggregates.

(a) The natural pit run or the 'as used' grading of the material coarser than the No. 10 sieve should be used in preparing samples for the D_c and LAR tests. Since lateritic gravels are petrologically heterogeneous, an artificial laboratory grading may not give results representative of the material as it will be used. The No. 10 sieve, rather than the No. 4, was selected as the minimum sieve size because most lateritic gravels observed were 'gap-graded', with the particle sizes between about the No. 10 and No. 200 sieves frequently missing. The result is a gravel 'floating' in silt or clay. Use of the No. 10 sieve will result in including the smaller nodules, frequently the weakest, in the durability test.

(b) Oven drying of samples prior to conducting the LAR, the D_c, or the D_f test is not recommended primarily because of mineralogic changes which may occur. For good quality, coarse-grained materials oven drying appears to have no significant effect on the LAR or D_c test results (Ref. 1). However, for coarse-grained lateritic materials containing a high proportion of weak laterite or plinthite (such as laterites from poorly drained tropical rain forest areas) oven drying may cause particle cementation, mineralogic alteration of clays and alteration of iron minerals.

If such changes occur, they will affect the LAR, and especially, the D_c and D_f test results. The effect will be toward indicating a higher durability than actually exists.

(c) It is recommended that the LAR test be run with 10 standard steel balls, regardless of sample grading. The effect of varying the number of balls was observed to be more severe than the effect of changing the grading. Changing both gradation and the number of balls may lead to widely differing results on the same material. In general, the LAR test run with 10 balls on the naturally graded material yielded results midway between the standard 'B' (11 ball) and 'C' (8 ball) grading results for the same material. Furthermore, it is suggested that the natural grading and 10 ball procedure be called the 'L' grading.

34. The LAR test results for lateritic gravels from Southeast Asia may range from 20 to 60, when tested with the 'L' grading. The D_c may vary from 5 to 80 and the D_f from 15 to 70. These ranges are based on results from over 500 durability tests on lateritic gravels from throughout Southeast Asia. Thus, lateritic gravel aggregates range in quality from very weak to comparable to good limestone. The data indicate that both the LAR and D_c tests correlate with the apparent mechanical durability of laterite gravels. However, the D_c and D_f tests on the air-dried natural (or as-used) grading, are believed to provide a better and more consistent measure of the relative durability of these materials in service than does the LAR. These tests measure not only the resistance of the material to rubbing abrasion in the presence of water, but also evaluate the quality and quantity of the fines produced by this abrasion. The two tests together measure the relative mechanical durability of the entire material from the ¾ in. through the No. 200 sieve.

35. Though these tests appear to provide good and valid information, there is some problem with the repeatability of the D_f test when run on some lateritic soils. This is believed to be due primarily to the small sample size used in the test, and the extreme sensitivity of the test. In order to resolve this problem it is recommended that duplicate D_f tests be run on lateritic soils and the results averaged. Although the D_c test is affected by the same factors which affect the D_f test, the test portion is over ten times as large and the coarser fractions of laterite generally are better formed and more homogeneous than the finer fractions. This accounts for the fact that the repeatability of the D_c test is much better than that of the D_f test.

36. The California Division of Highways has concluded that no direct correlation exists betwen these tests and the LAR test. An examination of the data obtained during the present investigation confirms this conclusion.

CONCLUSION REGARDING EFFECTS OF DURABILITY ON AGGREGATE BEHAVIOUR UNDER REPEATED MECHANICAL STRESS

37. Because of the extreme care in selection and preparation of the samples it is believed some specific conclusions can be drawn from the results obtained on this test series, though the number of tests is relatively small.

(a) There is apparently a direct correlation between the D_c and D_f of the sample, and the degree of disintegration incurred by cyclic loading. The changes which occurred after only 5000 cycles of relatively low stress loading are shown in *Fig. 4*. An examination of the ultimate strengths and M_r's of aggregates 1 and 8 indicate that the highly durable control limestone and non-durable lateritic gravel were of very nearly the same strength and stiffness (M_r). The slight difference is apparently due to the difference in friction angle between the two materials. However, there was over 50 times more reduction in the modified fineness modulus (MFM) of the lateritic material than of the limestone. The change in MFM shown for Aggregate 1 represents a change of about 5 per cent from the original MFM. Considering the low stress and relatively few cycles, this is felt to be excessive. A base material may be subjected to millions of cycles of a 40 to 50 p.s.i. major principal stress during its lifetime. In short, the samples had very nearly identical engineering properties and performance except for their difference in durability and the great difference in disintegration attribute to it.

(b) All of the samples had an increase in stiffness (M_r) with increasing cycles. This increase varied from 25 to 80 per cent of the original M_r value. It should be noted that for three of the samples 50 per cent of this change occurred in less than 10 cycles, and for the other samples it occurred in less than 150 cycles. The increased stiffness may be observed in the effect on the stress-strain curves of all of the materials. The following test results for sample 47B illustrate this effect for various numbers of cycles of repeated stress.

No. of Cycles	Strain at Ultimate Stress, % of Original
0	100
20	25
5000	27

38. It is significant to note that the variation in D_f given in TABLE VII is entirely due to the variation in durability of the soil fraction between the No. 4 and No. 8 sieves as the material smaller than the No. 8 was identical in every sample. The samples tested by the D_c test were composed entirely of the individual aggregate tested and predictably the results shown in TABLE VII emphasize the difference indicated by the D_f results.

GENERAL CONCLUSIONS REGARDING THE MECHANICAL DURABILITY OF LATERITIC GRAVELS

39. Lateritic gravels which are sufficiently well developed and of good enough quality to be considered for use in airfields or highways, exhibit mechanical durability properties which are comparable to those of non-lateritic materials. The D_c and D_f test procedures recommended in this paper correlate well with the observed performance of the materials tested.

40. Limited cyclic load test data and field observations indicated that the present California test standards for the D_c and D_f tests are probably suitable for lateritic gravel aggregates. These standards require a minimum value of 35 for the D_c or D_f test result to ensure satisfactory durability of materials used in a base course beneath a paved surface. Higher values would be desirable for materials used on unsurfaced roads. No requirement is specified or believed necessary for subbase materials.

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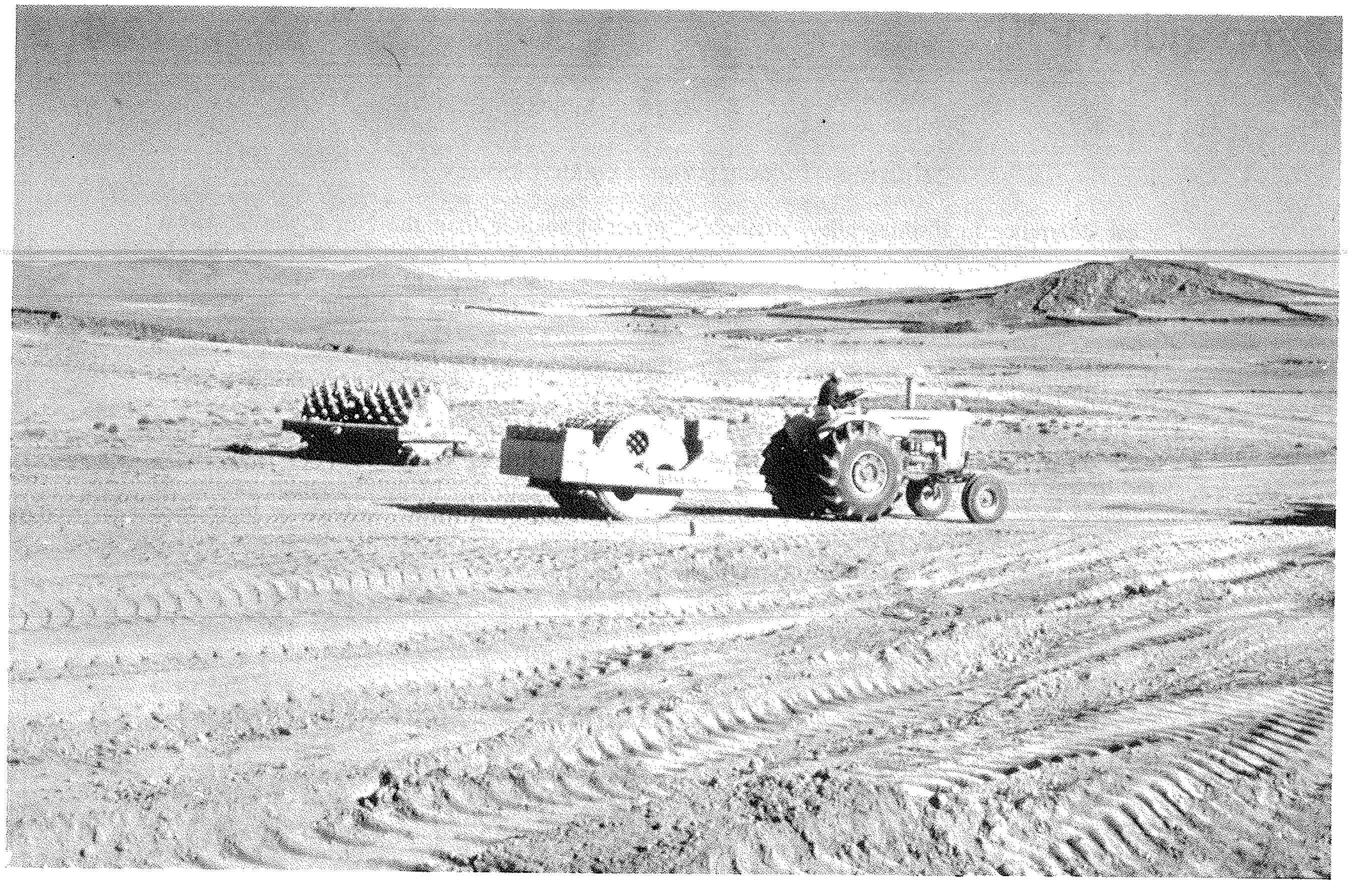

112 Compactors are towed by tractors (Bolivia).

LATERITE AND LATERITIC SOILS AND OTHER PROBLEM SOILS OF THE TROPICS

AN ENGINEERING EVALUATION AND
HIGHWAY DESIGN STUDY FOR
UNITED STATES AGENCY FOR
INTERNATIONAL DEVELOPMENT
AID/csd 3682

by W.J. MORIN
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VOLUME II

INSTRUCTION MANUAL

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CHAPTER 2

TEST PROCEDURES FOR EVALUATION OF TROPICAL SOIL PROPERTIES

INTRODUCTION

Well established testing procedures used in engineering evaluations of temperate soils are not always suitable for evaluating tropical soils. Sometimes a modification of the standard tests are necessary in order to obtain a proper evaluation. For example, experience with tropical soils has shown that the procedure of manipulating and pre-heating preparations of temperate soils will change the properties of tropical soils unless the procedure is altered.

The changes in engineering properties that occur with pre-heating prior to testing are usually irreversible. The gradation, Atterberg limits and the moisture-density relationship are all affected. An example of such changes are shown in Figure 2.1. The sample was obtained from a construction site at the Juan Santa Maria International Airport, San Jose, Costa Rica. The testing was conducted by the Departamento Laboratorio de Materiales, de Obras Públicas y Transportes, San Jose. A special study was undertaken since the contractor was unable to obtain the specified compaction which had been determined in the laboratory. The data shown on the left side of Figure 2.1 are the original test results while the data on the right are the test results of the special study. In the latter test the sample was not dried prior to testing. The compaction curve on the right represents the moisture-density relationship of the material as it exists in the field. The subgrade had a natural moisture content of 70 percent, therefore the difficulty in obtaining the specified laboratory density in the field is obvious. However, it should be noted that the problem was restricted only to the compaction since the four-day soaked CBR for both samples was the same. It is not known if the as-molded CBR would have been the same for both samples but it is known that the in-situ CBR's at the site were in the order of 16.

It is important that such moisture-sensitive soils be identified in preliminary investigations in order to avoid delays during the construction phase. An "aggregation index" is recommended to determine the propensity of a soil to change after dehydration. This index is defined as the sand equivalent value of the soil in its natural state divided by the sand equivalent of the oven-dried sample. An index of two indicates a moderately sensitive soil and an index of 12 indicates a highly sensitive soil.

The effect of drying on the Atterberg limits for several soils tested during the African study is illustrated in Table 2.1. Tests were conducted initially without drying (at the field moisture content), with air drying and with oven drying at various temperatures and drying time. The data appears to indicate that temperature causes the greatest change and the time of drying is secondary.

Variations also occur in the Atterberg limits depending upon the amount of manipulation of the sample prior to testing. Excessive manipulation prior to testing leads to breakdown of the soil structure and disaggregation. Both consequences produce fines which results in higher liquid limit values.

The hydrometer analysis is a particularly difficult test in that it is often difficult to reproduce results. The strong tendency of tropical soils to aggregate or flocculate presents a problem in dispersing the soils prior to testing.

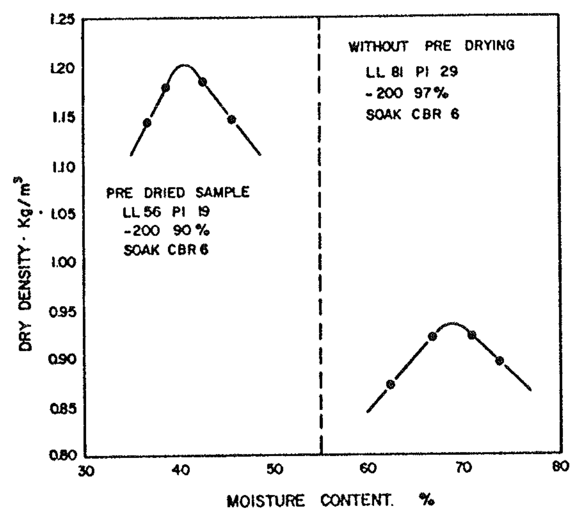

FIGURE 2.1 – COMPARISON OF PHYSICAL PROPERTIES TESTED WITH AND WITHOUT PRE-DRYING (COURTESY OF DEPARTAMENTO LABORATORIO DE MATERIALES, MINISTERIO DE OBRAS PUBLICAS Y TRANSPORTES, SAN JOSE, COSTA RICA)

TEST PROCEDURES

The following are the recommended testing procedures to be used in engineering evaluation of tropical soils. Recommendations and modifications are suggested in view of the nature of these soils mentioned above.

Test	AASHO	British Std.
1) *Dry Preparation of Soil Samples*	T 87-70	1377 Part 2 Sec. 4

Comments: It is recommended that the soil be air dried regardless of relative humidity and that oven drying be avoided.

Test	AASHO	British Std.
2) *Wet Preparation of Soil Samples*	T 146-49	1377

Comments: It is recommended that the soil be air dried and oven drying be avoided.

Test	AASHO
3) *Preparation of Soil Samples at Natural Moisture Content*	NA

Comments: This procedure has not been standardized by either AASHO or ASTM. It is a special preparation procedure and was developed to facilitate the testing of

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TABLE 2.1
Change in Atterberg Limits After Drying at Various Temperatures and Time Periods

Soil Samples		As Received	Air Dried	6 hours at 50° C	24 hours at 50° C	6 hours at 105° C	24 hours at 105° C
A	LL	63.4	62.2	60.1	60.7	57.3	55.1
	PL	39.1	31.2	27.2	28.7	27.4	28.4
B	LL	54.3	47.6	47.1	44.8	41.8	41.9
	PL	22.4	22.9	22.2	24.1	23.1	22.4
C	LL	51.8	44.6	44.7	45.5	42.8	42.9
	PL	29.8	26.8	23.9	22.9	19.8	20.3
D	LL	45.2	40.0	40.5	41.9	41.0	37.6
	PL	21.7	21.0	21.6	21.8	24.8	20.7
E	LL	36.5	34.5	36.1	36.8	36.0	35.2
	PL	38.2	30.2	31.4	29.1	26.3	28.2
F	LL	29.0	26.5	26.2	26.5	25.8	24.7
	PL	21.4	16.6	15.6	14.8	14.1	13.8
G	LL	65.0	62.9	58.3	58.5	49.4	46.0
	PL	27.2	27.8	29.4	28.5	24.4	25.9
H	LL	61.6	53.4	54.4	53.9	44.4	42.7
	PL	28.2	25.5	26.2	24.8	23.6	23.8
J	LL	45.7	44.2	44.4	44.4	43.4	44.4
	PL	23.2	23.5	25.9	25.9	26.2	26.1

tropical soils at their natural moisture content. This was necessary because some tropical soils, particularly andosols, exhibit changes in engineering properties with drying. Two factors which cause the change in properties with drying are: (1) the tendency to form aggregation on drying and (2) the loss of water in hydrated minerals. The first generally results in an increase in strength while this is not necessarily true with the second effect.

The following is the procedure for preparing a sample for the Atterberg limit tests:

Wet Method:

1) Break up the required amount of material with rubber-covered pestle or rolling pin.
2) Transfer sample to saucepan and cover with water. Let soak until all material is disintegrated. This may require 2 to 12 hours.
3) Place a No. 40 sieve in a saucepan and transfer entire soaked sample into the sieve. Wash any material still adhering to the soaking pan into the sieve by squirting water from a battery filler.
4) Pour clean water into pan containing sieve until level of water is about 1/2 inch above mesh in sieve.
5) Agitate the sieve with one hand without lifting the sieve. Concurrently, stir material with the other hand until all fine material appears to have passed through sieve.
6) Hold sieve slightly above water surface in pan and squirt water from battery filler onto sieve until retained particles and the sieve are clean. Discard material retained in sieve.
7) Place pan where it will not be disturbed and block it up on one side so water on the other side barely reaches rim of pan. Allow soil to settle for several hours.
8) Pour off liquid slowly by gradually increasing tilt of pan until cloudy layer overlaying the sediment reaches rim of pan.
9) Air-dry material to a smooth paste consistency and put in small mixing dish.

For most lateritic soils, material in suspension will settle out. If there is no indication of this after several hours, the following method may be used.

Place filter paper in a funnel and place wet soil inside the funnel in a jar or other container and allow to stand until all the excess water is filtered off.

The procedure of conducting the Atterberg test is the same with the exception that the low count (high moisture) is established first. The material is allowed to dry and the second point is established. This procedure is followed until a flow-curve is developed that will define the moisture content at 25 blows. The Plastic Limit is then determined after the Liquid Limit Test.

For many of these tropical soils that change properties the amount of + 40 material is such a minor constituent that it is often unnecessary to sieve the material prior to conducting the liquid limit test.

The following is an outline of the procedure to follow when preparing a sample at its natural moisture content for establishing the moisture-density relationship.

1) The sample at natural moisture content is passed through a 3/4 inch sieve.

2) The sample is split into five more or less equal parts each of which are sufficient for compaction in a six inch mold.

3) Two percent moisture is added to one sample and allowed to cure in a plastic bag or sealed container for a minimum of 12 hours prior to compaction; 18 to 24 hours would coincide better with normal working hours.

4) A second sample is then compacted at its natural moisture content with the appropriate compactive effort and prepared for CBR testing if required.

5) The remaining three samples are permitted to air dry for different periods of time prior to compacting the sample.

Test	AASHO	British Std.
4) *Particle Size Analysis*	T 88-70	1377 Test 6

Comments: It is not recommended to dry the soil prior to testing. It is recommended that the mechanical analysis be performed at the natural moisture content. A moisture correction must be applied prior to computations. The appropriate apparatus for performing the sieve analysis is shown in Figure 2.2.

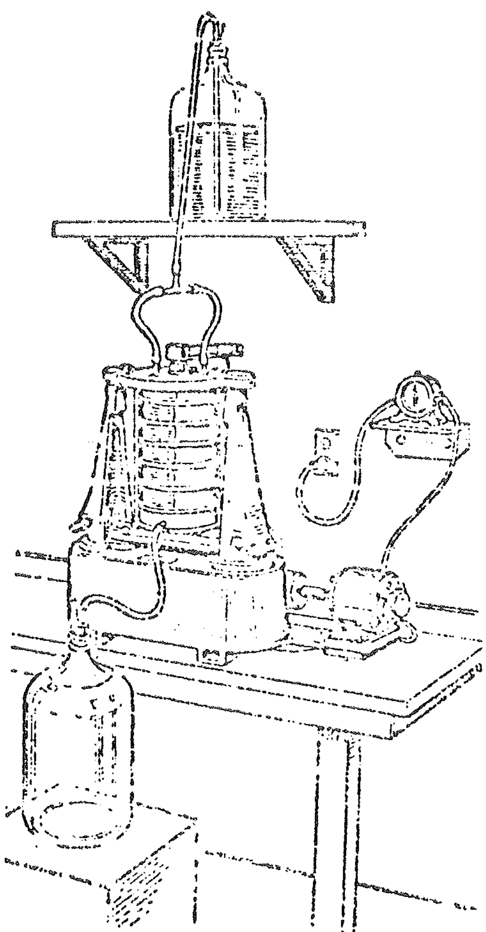

FIGURE 2.2 – WET TEST SETUP WITH MECHANICAL SIEVE SHAKER (ASTM 1969)

Sodium hexametaphosphate should be used as the dispersing agent in the hydrometer analysis. The dispersing time should be 15 minutes. Experience has shown that sedimentation tests are difficult to perform on tropical soils. It should be remembered that Stokes' law of sedimentation does not give the actual diameter of the particles but only the diameter of an equivalent sphere. The diameter of a clay plate can be five times greater than the one determined from Stokes' law. Any sedimentation test, no matter how accurately performed, gives only a general indication of the size and quantity of soil particles.

Test	AASHO	British Std.
5) *Liquid Limit*	T 89-68	1377 Test 2A

Comments: The "Single-point Method" evaluated during the African study can be used. Figure 2.3 shows the comparison of test results of the South American soils obtained in the standard laboratory procedure plotted against the results obtained with the African equation

$$LL = W \frac{(N)^{0.15}}{25}$$

where:
W = water content at N blows percent
N = number of blows.

A maximum of five minutes mixing time is recommended because tropical soils are susceptible to breakdown with manipulation.

Test	AASHO	British Std.
6) *Plastic Limit and Plasticity Index*	T 90-70	1377 Test 6

Test	AASHO	
7) *Moisture Density Relations*	T 99-70	2.
	T 180-70	2.

Comments: After mixing the samples with the various percentages of water the sample should be sealed in an air-tight container and allowed to cure for a period of 12 hours to insure a homogeneous mixture prior to compaction. When significant amounts of gravel size materials are present which are hard and impermeable large moisture samples are necessary. The following quantities are recommended for moisture determinations:

10 grams for minus No. 40 material
200 grams for minus No. 4 material
1,000 grams for minus 3/8 inch material
2,000 grams for minus 3/4 inch material

Test	AASHO	British Std.
8) *Specific Gravity*	T 100-70	1377 Test 5

Comments: When the moisture density relation is established without drying the specific gravity should also be determined without allowing the sample to dry prior to testing. A moisture correction is used prior to computations.

Test	AASHO
9) *California Bearing Ratio*	T 192-63

Comments: The recommendations given for AASHO T 99 and T 180-70 should be followed in compaction. If initial testing indicates, that the soil is moisture sensitive, the compaction should be accomplished without predrying.

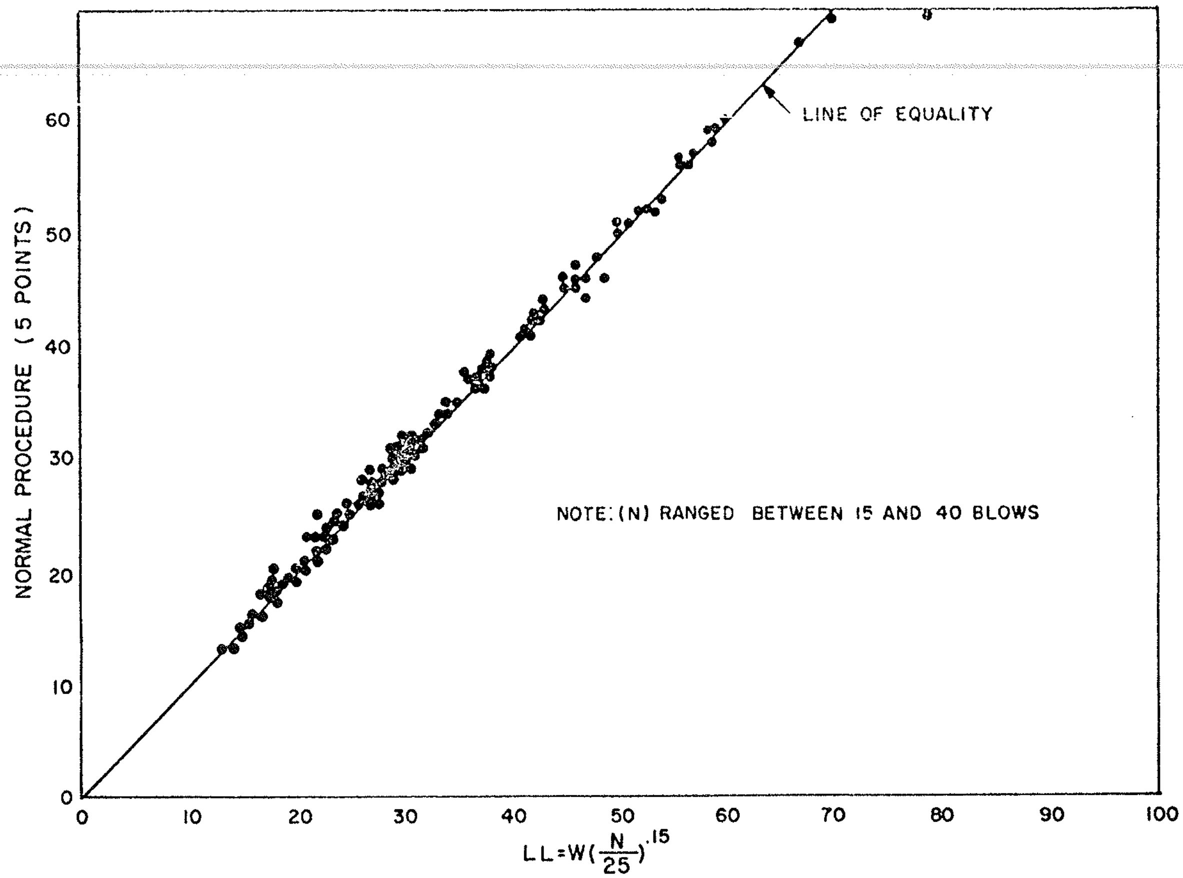

FIGURE 2.3 – COMPARISON BETWEEN 5 POINTS LIQUID LIMIT AND LIQUID LIMIT DETERMINED BY SINGLE POINT METHOD

Test	AASHO
10) Sand Equivalent Value	T 176-70

Comments: The susceptibility to change of engineering properties due to dehydration can be estimated by the aggregation index, the ratio of the oven dried sand equivalent value to the sand equivalent value at the field moisture content. The sand equivalent test is run in accordance with AASHO T 176-70 except for the field moisture sample which is not dried prior to testing. An aggregation index value greater than 2 indicates the soil may be susceptible to change in engineering properties upon drying.

The following three test procedures may not be readily available. These are reproduced in their entirety from the sources provided.

* Reproduction from test Method No. Calif. 229-C, July, 1963, Materials and Research Department – State of California, Department of Public Works, Division of Highways.

11) Method of Test for Durability of Aggregates (Mechanical Agitation in Water)*

Water)

Scope

This method describes the procedure for determining the durability factor of aggregates. The durability factor is a value indicating the relative resistance of an aggregate to producing detrimental claylike fines when subjected to the prescribed mechanical methods of degradation.

Procedure

A. APPARATUS

1. Mechanical washing vessel ("pot"): A flat bottom, straight sided cylindrical vessel conforming to the specifications and dimensions shown in Fig. 2.4a. This vessel is

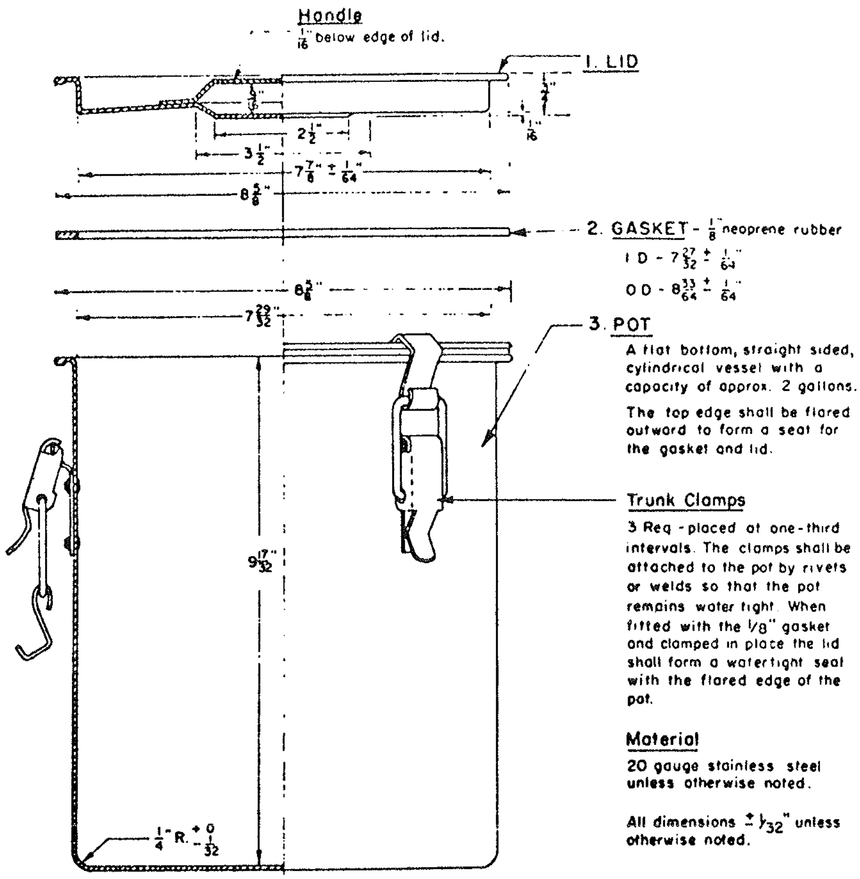

FIGURE 2.4a – MECHANICAL WASHING VESSEL

available to California State Agencies from the Service and Supply Department of the Division of Highways (Stock No. 69,577 NR).

2. Vessel: A round pan suitable to collect the wash water from the washed sample.

3. Agitator: A Tyler portable sieve shaker, modified as shown in Fig. 2.4b and set to operate at 285 ± 10 complete cycles per minute. The two agitation periods specified under E, Preparation of Sample, and F, Test Procedure are for this modified shaker. Other types of sieve shakers may be used, provided the length of time and/or other factors are adjusted so that results can be obtained which duplicate the results obtained with the modified Tyler portable sieve shaker. See Figure 2.4c for a photograph of the mechanical washing vessel secured in position in the standard mechanical agitator.

4. Graduated cylinders of 10 ml and 1,000 ml capacities.

5. A graduated plastic cylinder, rubber stopper, irrigator tube, weighted foot assembly and siphon assembly all conforming to their respective specifications and dimensions shown in Fig. 2.4d.

A sand equivalent test kit, which contains the necessary equipment, except for a 1-gal. bottle, is available to California State agencies from the Service and Supply Department of the Division of Highways (Stock No. 69,690 NR). Fit the siphon assembly to a 1-gal. bottle of working calcium chloride solution placed on a shelf 3 ft. ± 1 in. above the work surface (See Fig. 2.4e). In lieu of the specified 1-gal. bottle, a glass or plastic vat having a larger capacity may be used providing the liquid level of the working solution is maintained between 36 and 46 inches above the work surface.

6. Measuring tin: A 3-oz. tinned box approximately $2\frac{1}{4}$ in. in diameter with Gill style cover, and having a capacity of 85 ± 5 ml.

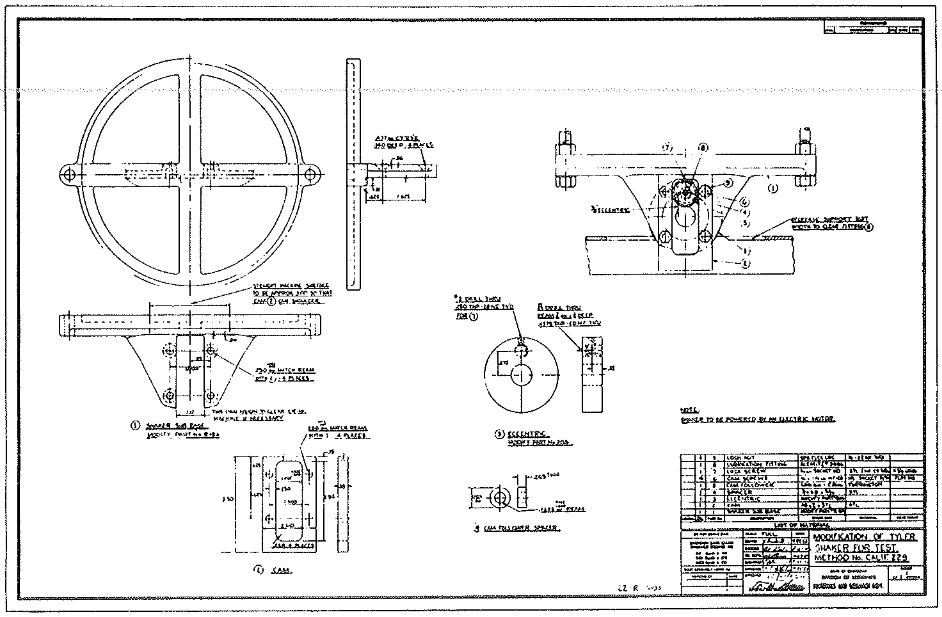

FIGURE 2.4b – MODIFICATION OF TYLER SHAKER FOR TEST

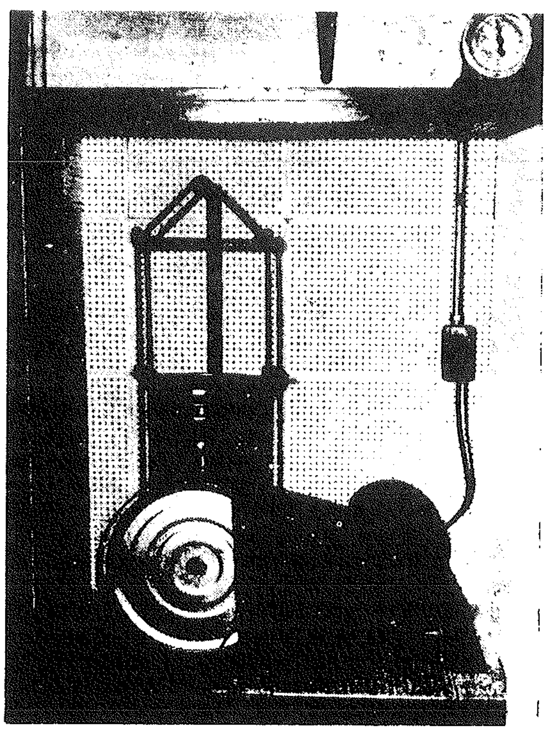

FIGURE 2.4c – MECHANICAL WASHING VESSEL IN TYLER SHAKER

7. Funnel: A wide-mouth funnel approximately 4 in. in diameter at the mouth.

8. Clock or watch: A clock or watch reading in minutes and seconds.

9. Mechanical Sand Equivalent Shaker[1]. A shaker conforming to the specifications and dimensions shown in the State of California, Division of Highways, Materials and Research Department Plans Designation D-256 (See Fig. 2.4f). Prior to use, fasten the mechanical sand equivalent shaker securely to a firm and level mount and disconnect the timer so that the sample can be agitated continuously for the prescribed 10-minute shaking time.

10. Sieves: The sieves shall be of the woven wire type with square openings and shall conform to the "Standard Specifications for Sieves for Testing Purposes," AASHO Designation M-92.

11. A balance or scale with a minimum capacity of 5,000 grams and sensitive to 1 gram.

B. MATERIALS

1. Stock calcium chloride solution (same as stock solution used in Sand Equivalent Test) consisting of:

454 g (1 lb.) tech. anhydrous calcium chloride
2,050 g (1,640 ml) U.S.P. glycerine
47 g (45 ml) formaldehyde (40 percent by volume solution).

1 This mechanical shaker is a modification of shaker designs originally developed by Henry Davis of the California Division of Highways, and by the Laboratorie Central des Ponts et Chaussees, Paris, France, under the direction of Mr. R. Peltier. The mechanical shaker is available to California Division of Highways agencies from the Service and Supply Department, Stock No. 69,849 NR.

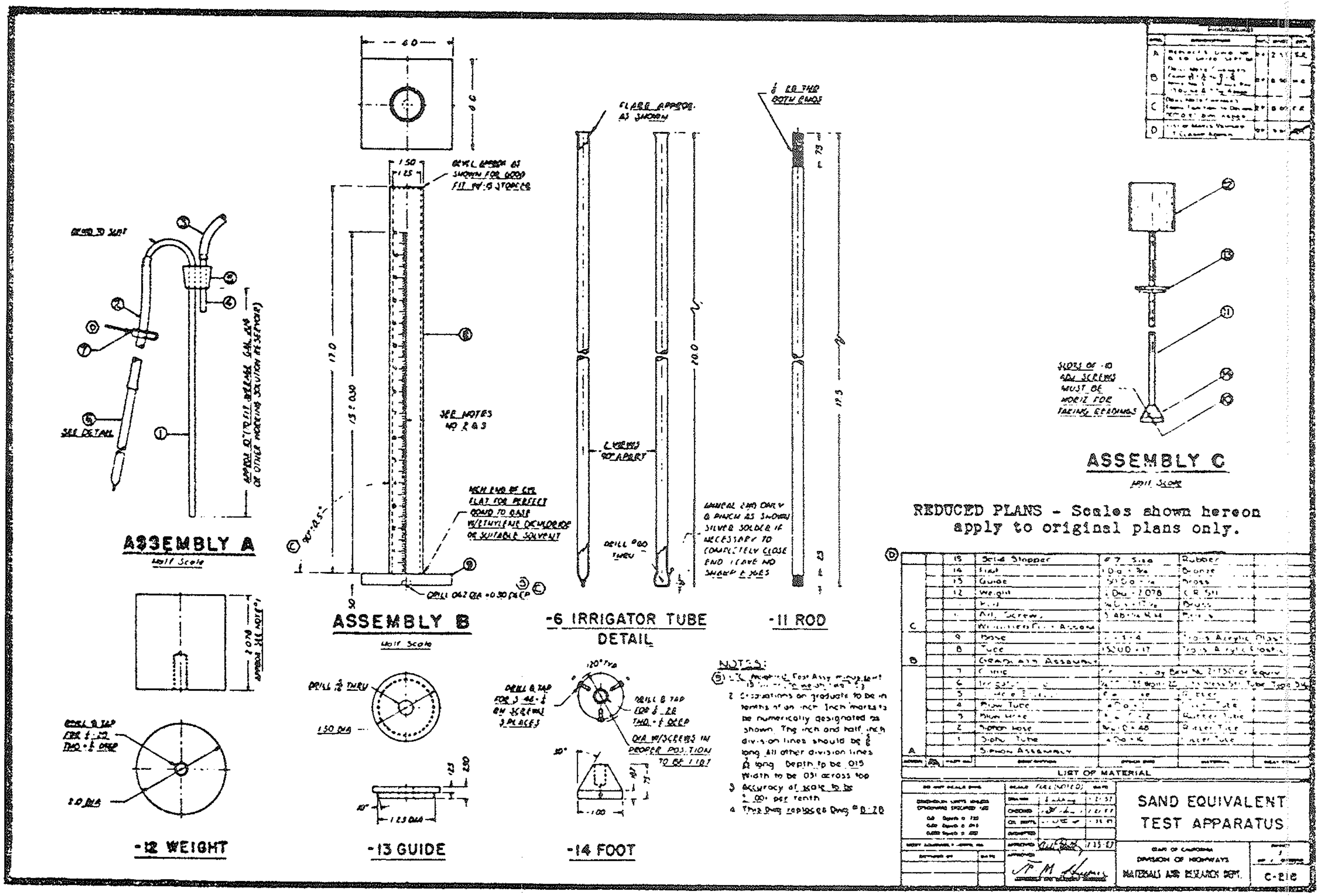

FIGURE 2.4d – SAND EQUIVALENT TEST APPARATUS

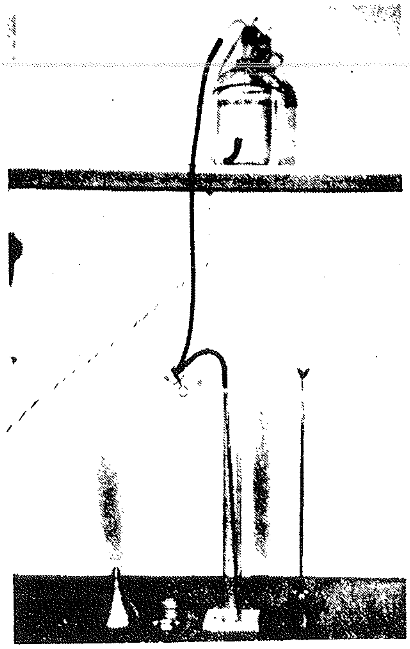

FIGURE 2.4e – SAND EQUIVALENT TEST APPARATUS EXCLUDING SHAKER

FIGURE 2.4f – MECHANICAL SAND EQUIVALENT SHAKER

Dissolve the calcium chloride in 1/2 gal of distilled or demineralized water. Cool the solution, then filter it through Whatman No. 12 or equivalent filter paper. Add the glycerine and formaldehyde to the filtered solution, mix well, and dilute to 1 gal. with distilled or demineralized water. District laboratories should secure stock calcium chloride solution from the Service and Supply Department of the Division of Highways (Stock No. 69,691).

2. Working calcium chloride solution: Prepare the working calcium chloride solution by diluting one measuring tin full (85 ± 5 ml) of the stock calcium chloride solution to 1 gal. with water. Use distilled or demineralized water for the normal preparation of the working solution. However, if it is determined that the local tap water is of such purity that it does not affect the test results, it is permissible to use it in lieu of distilled or demineralized water except in the event of dispute.

3. Water: Use distilled or demineralized water for the normal performance of this test. This is necessary because the test results are affected by certain minerals dissolved in water. However, if it is determined that the local tap water is of such purity that it does not affect the test results it is permissible to use it in lieu of distilled or demineralized water except in the event of dispute.

C. TEST RECORD FORM

Record test results on Form T-200 or T-361.

D. CONTROL

This test may be normally performed without strict temperature control; however, in the event of dispute retest the material with the temperature of the distilled or demineralized water and the working calcium chloride solution at 72 ± 5 F.

E. PREPARATION OF SAMPLE

1. Prepare the sample as described in Test Method No. Calif 201. Care should be exercised in cleaning the coarse aggregate and breaking up of clods so that the method used does not appreciably reduce the natural individual particle sizes.

2. Separate the sample on the 3/4-inch, 1/2-inch, 3/8-inch and No. 4 sieves. Set aside that portion of the material retained on the 3/4-inch sieve. Weigh and record the weights of material retained on the 1/2-inch, 3/8-inch and No. 4 sieves.

3. Preparation of Coarse Aggregate Test Sample.

a. Determine the grading to be used in preparing each preliminary test sample as follows:

(1) If each of the aggregate sizes listed below represents 10 percent or more of the 3/4"x No. 4 portion, as determined from the weights recorded in paragraph 2 above, use the oven dry weights of material specified below for preparing each preliminary test sample.

Aggregate size	Oven dry weight-grams
3/4" x 1/2"	1,050 ± 10
1/2" x 3/8"	550 ± 10
3/8" x No. 4	900 ± 5
Test Sample Weight	2,500 ± 25

(2) If any of the aggregate sizes listed in above represents less than 10 percent of the 3/4" x No. 4 portion, use the same percentage of material from the deficient aggregate size or sizes as was determined from the weights recorded in paragraph 2 above and proportionally increase the weight of the remaining size or sizes to obtain the 2,500-gram preliminary test sample weight.

Example 1 – Less than 10% of 3/4" x 1/2" aggregate size material

Aggregate size	Percent each size	Calculations	Oven dry weight-grams
3/4" x 1/2"	6	.06 x 2500	150 ± 10
1/2" x 3/8"	26	$\frac{550(2500-150)}{550+900}$	891 ± 10
3/8" x No. 4	68	$\frac{900(2500-150)}{550+900}$	1459± 5
Test Sample Weight			2500± 25

b. Prepare two 2,500-gram preliminary test samples using the prescribed grading. Dry the test samples to constant weight at a temperature of 221 to 230 F.

c. After allowing the oven dried material to cool, place one of the preliminary test samples in the mechanical washing vessel, add 1,000 ± 5 ml of distilled or demineralized water, clamp the vessel lid in place and secure the vessel in the sieve shaker.

Example 2 – Less than 10% of 3/4" x 1/2" and 1/2" x 3/8" aggregate size materials

Aggregate size	Percent each size	Calculations	Oven dry weight-grams
3/4" x 1/2"–	4	.04 x 2500	100±10
1/2" x 3/8"–	7	.07 x 2500	175±10
3/8" x No. 4	89	2500–(100+175)	2225± 5
Test Sample Weight			2500±25

d. Begin agitation after a time of *1 minute ± 10 seconds* has elapsed from the introduction of the wash water. Agitate the vessel in the sieve shaker for *two minutes ± 5 seconds.*

e. After the two-minute agitation time is completed, remove the vessel from the shaker, unclamp the lid and pour the contents into a No. 4 sieve. Rinse any remaining fines from the vessel onto the sieve and direct water (from a flexible hose attached to a faucet) onto the aggregate until the water passing through the sieve comes out clear.

f. Wash the second preliminary test sample in the same manner as prescribed above then combine all of the washed material obtained from both preliminary test samples and dry to constant weight at a temperature of 221 to 230 F.

g. After allowing the oven dried material to cool, separate the washed coarse aggregate on the 1/2-inch, 3/8-inch and No. 4 sieves. Discard the material passing the No. 4 sieve.

h. Prepare the washed test sample as follows:

(1) If the preliminary test samples were prepared using the weights specified in a.(1) above, prepare the washed test sample using the weights specified from representative portions of each size of washed material. Occasionally it may be necessary to wash a third preliminary test sample to obtain the required weight of material of a specific size.

(2) If the weights of material prescribed were adjusted as prescribed in the preparation of the preliminary test sample, use *all* of the material representing the deficient size or sizes obtained from washing the two preliminary test samples and proportionally increase the weight of the remaining size or sizes to obtain the 2,500-gram washed test sample.

4. Preparation of Fine Aggregate Test Sample

a. Split or quarter a representative portion from the material passing the No. 4 sieve of sufficient weight to obtain an oven dry weight of 500±25 grams.

b. Dry this preliminary test sample to constant weight at a temperature of 221 to 230 F. Cool to room temperature.

c. Place this preliminary test sample in the mechanical washing vessel, add 1,000±5 ml of distilled or demineralized water, clamp the vessel lid in place. Secure the vessel in the sieve shaker in sufficient time to begin agitation after ten minutes±30 seconds has elapsed from the introduction of the wash water. Agitate the vessel for a period of two minutes ±5 seconds.

d. After the two minute agitation period is completed, remove the vessel from the shaker, unclamp the lid and carefully pour the contents into a No. 200 sieve. Rinse any remaining fines from the vessel into the sieve. Direct water (from flexible hose attached to a faucet) onto the aggregate until the water passing through the sieve comes out clear. 123

e. It may be necessary to flood clayey or silty samples prior to pouring them over the sieve to prevent clogging the No. 200 sieve. Flood by adding water to the vessel following the agitation period. This dilutes the wash water and reduces its tendency to clog the sieve. Repeated flooding may be necessary in extreme cases before all of the contents of the vessel can be poured over the sieve.

f. Following the rinsing, transfer the material from the sieve to a drying pan, and dry to constant weight at a temperature of 221 to 230 F. It is necessary to wash the material from the No. 200 sieve in order to transfer the retained material to a drying pan. Leave the pan in a slanting position until the free water that drains to the lower side becomes clear, then pour off this clear water. Use large shallow pans and spread the sample as thinly as possible to speed drying.

g. After allowing the oven dried material to cool, mechanically sieve the washed test sample for 20 minutes using the following nested sieves: No. 8, No.16, No. 30, No. 50, No. 100 and No. 200. Place a pan below the No. 200 sieve to catch that portion of the material passing the No. 200 sieve. Refer to Test Method No. Calif. 202 for general instructions on sieving procedure.

h. After sieving the washed test sample recombine all of the material retained on each sieve with the material passing the No. 200 sieve that was caught in the pan.

i. Split or quarter sufficient amount of the washed and sieved material to fill the 3-ounce measuring tin to the brim or slightly rounded above the brim. While filling tin measure, tap the bottom edge of the tin on a work table or other hard surface to cause consolidation of the material and allowing the maximum amount to be placed in the measuring tin. Use extreme care in this procedure to obtain a truly representative sample. If the quartering method is used, follow the procedure as specified for "Hand quartering of samples weighing less than 25 lb." in Test Method No. Calif. 201.

F. TEST PROCEDURE

1. Test Procedure for Coarse Aggregate

a. Place the plastic cylinder on a work table which will not be subjected to vibrations during the performance of the sedimentation phase of the test. Pour 7 ml of the *stock calcium chloride solution into the cylinder.* Place a No. 8 and No. 200 sieve on the pan or vessel provided to collect the wash water with the No. 8 sieve on top. The No. 8 sieve serves only to protect the No. 200 sieve.

b. Place the prepared aggregate sample in the mechanical washing vessel. Then add 1,000±5 ml distilled or emineralized water, clamp the lid in place and secure the vessel in the sieve shaker. Begin agitation after a time of 1 min has elapsed from the introduction of the wash water. Agitate the vessel for 10 minutes ± 15 seconds.

c. Immediately following the 10 min agitation period, take the vessel from the sieve shaker and remove the lid. Then agitate the contents of the vessel by moving the upright vessel vigorously in a horizontal circular motion five or six times in order to bring the fines into suspension.

Immediately pour all of the contents of the vessel into the nested No. 8 and No. 200 sieves placed in the pan provided to collect the wash water.

d. Add enough distilled or demineralized water to bring the volume of dirty wash water to 1,000 ± 5 ml. Then transfer the wash water to a vessel suitable for stirring and pouring.

e. Place funnel in the graduated plastic cylinder. Stir the wash water with the hand to bring the fines into suspension. While the water is still turbulent pour enough of the wash water into the cylinder to bring the level of the liquid to the 15 in. mark.

f. Remove the funnel, place the stopper in the end of the cylinder, and prepare to mix the contents immediately.

g. Mix the contents of the cylinder by alternately turning the cylinder upside down and right side up, allowing the bubble to completely traverse the length of the cylinder *20 times* in approximately *35 seconds.*

h. At the completion of the mixing process, place the cylinder on the work table and remove the stopper. Allow the cylinder to stand undisturbed for *20 minutes* ± 15 seconds. Then immediately read and record the height of the sediment column to the nearest 0.1 inch.

i. There are two unusual conditions that may be encountered in this phase of the test procedure. One is that a clearly defined line of demarcation may not form between the sediment and the liquid above it in the specified 20-minute period. If this happens, and the test is being made with distilled or demineralized water, allow the cylinder to stand undisturbed until the clear demarcation line does form, then immediately read and record the height of the column of sediment and the total sedimentation time. If this should occur in a test being made with tap water, discontinue the test and retest using an untested portion of the sample with distilled or demineralized water. The second unusual condition is that the liquid immediately above the line of demarcation may still be darkly clouded at the end of 20 minutes, and the demarcation line, although distinct, may appear to be in the sediment column itself. As for the first case, rerun the test using a new sample with distilled or demineralized water if tap water was used; otherwise read and record this line of demarcation at the end of the specified 20-minute sedimentation period as usual.

2. Test Procedure for Fine Aggregate

a. Siphon 4 ± 0.1 in. of working calcium chloride solution into the plastic cylinder.

b. Pour the prepared test sample into the plastic cylinder using the funnel to avoid spillage (See Fig. 2.4g). Tap the bottom of the cylinder sharply on the heel of the hand several times to release air bubbles and to promote thorough wetting of the sample.

c. Allow the wetted sample to stand undisturbed for 10 ± 1 minutes.

d. At the end of the 10-minute soaking period, stopper the cylinder, then loosen the material from the bottom by partially inverting the cylinder and shaking it simultaneously.

e. Place the stoppered cylinder in the mechanical sand equivalent shaker and allow the machine to continuously shake the cylinder and contents for 10 minutes ± 15 seconds.

f. Following the shaking operation, set the cylinder upright on the work table and remove the stopper.

g. Insert the irrigator tube in the cylinder and rinse material from the cylinder walls as the irrigator is lowered. Force the irrigator through the material to the bottom of the cylinder by applying a gentle stabbing and twisting action while the working solution flows from the irrigator tip. This flushes the fine material into the suspension above the coarser sand particles. (See Fig. 2.4h).

h. Continue to apply a stabbing and twisting action while flushing the fines upward until the cylinder is filled to the 15-inch mark. Then raise the irrigator slowly without shutting off the flow so that the liquid level is maintained at about 15 in. while the irrigator is being withdrawn. Regulate the flow just before the irrigator is entirely withdrawn and adjust the final level to 15 in.

i. Allow the cylinder and contents to stand undisturbed for 20 minutes ± 15 seconds. Start the timing immediately after withdrawing the irrigator tube.

j. At the end of the 20-minute sedimentation period, read and record the level of the top of the clay suspension. This is referred to as the "clay reading". If no clear line of demarcation has formed at the end of the specified 20-minute sedimentation period, allow the sample to stand undisturbed until a clay reading can be obtained, then immediately read and record the level of the top of the clay suspension and the total sedimentation time. If the total sedimentation time exceeds 30 minutes, rerun the test using three individual samples of the same material. Read and record the clay column height of that sample requiring the shortest sedimentation period only.

k. After the clay reading has been taken, place the weighted foot assembly over the cylinder with the guide in position on the mouth of the cylinder and gently lower the weighted foot until it comes to rest on the sand.

l. While the weighted foot is being lowered, keep one of the centering screws in contact with the cylinder wall near the graduations so that it can be seen at all times.

m. When the weighted foot has come to rest on the sand, read and record the level of the centering screw. This reading is referred to as the "sand reading". (See Fig. 2.4i).

n. If clay or sand readings fall between 0.1-inch graduations, record the level of the higher graduation as the reading. For example, a clay level at 7.95 would be recorded as 8.0. A sand level at 3.22 would be recorded at 3.3.

G. CALCULATIONS AND REPORTING

1. Durability Factor of Coarse Aggregate

a. Compute the durability factor of the coarse aggregate to the nearest whole number by the following formula:

$$D_c = 30.3 + 20.8 \cot (0.29 + 0.15\ H)$$

Where:

D_c = Durability Factor

H = Height of Sediment in inches

Solutions of the above equation are given in Table No. 2.2.

2. Durability Factor of Fine Aggregate

a. Calculate the durability factor of the fine aggregate to the nearest 0.1 using the following formula:

$$D_f = \frac{\text{Sand reading}}{\text{Clay reading}} \times 100$$

b. If the calculated durability factor is not a whole number, report it as the next higher whole number. For example, if the durability factor were calculated from the example in paragraph 2n of Article F the calculated durability factor would be:

$$D_f = \frac{3.3}{8.0} \times 100 = 41.2$$

c. Since this calculated durability factor is not a whole number it would be reported as the next higher whole number which is 42.

d. If it is desired to average a series of values, average the whole number values determined as described above. If the average of these values is not a whole number, raise it to the next higher whole number as shown in the following example:

(1) Calculated D_f values: 41.2, 43.8, 40.9.

(2) After raising each to the next higher whole number they become: 42, 44, 41.

(3) The average of these values is then determined.

$$\frac{42 + 44 + 41}{3} = 42.3$$

e. Since the average value is not a whole number it is raised to the next higher whole number and the reported average durability factor is reported as "43"

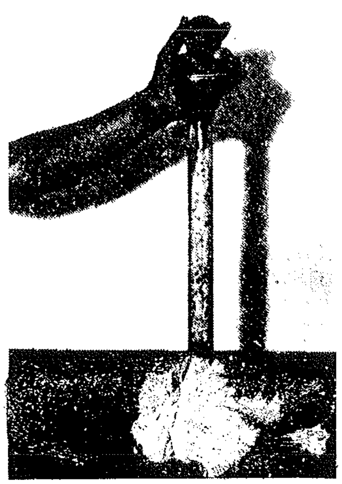

FIGURE 2.4g – TEST PROCEDURE FOR FINE AGGREGATE

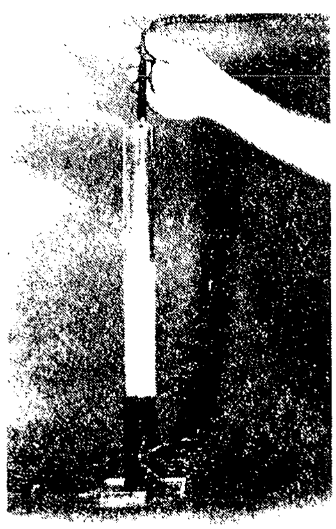

FIGURE 2.4h – TEST PROCEDURE FOR FINE AGGREGATE

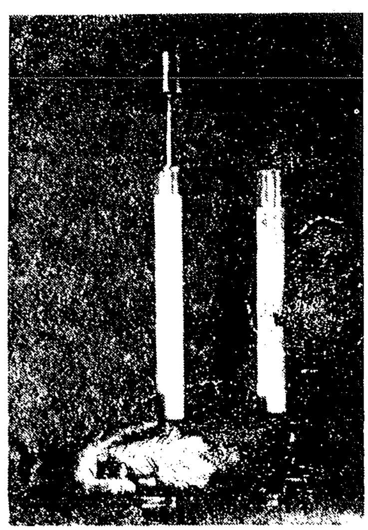

FIGURE 2.4i – TEST PROCEDURE FOR FINE AGGREGATE

TABLE 2.2

Durability Factor of Coarse Aggregate

$D_C = 30.3 + 20.8 \cot (0.29 + 0.15H)$

Sediment height (inches)	D_c	Sediment height (inches)	D_c	Sediment height (inches)	D_c	Sediment height (inches)	D_c	Sediment height (inches)	D_c
0.0	100	3.0	53	6.0	39	9.0	29	12.0	18
0.1	96	3.1	52	6.1	38	9.1	29	12.1	18
0.2	93	3.2	52	6.2	38	9.2	28	12.2	18
0.3	90	3.3	51	6.3	38	9.3	28	12.3	17
0.4	87	3.4	51	6.4	37	9.4	28	12.4	17
0.5	85	3.5	50	6.5	37	9.5	27	12.5	16
0.6	82	3.6	49	6.6	37	9.6	27	12.6	16
0.7	80	3.7	49	6.7	36	9.7	27	12.7	15
0.8	78	3.8	48	6.8	36	9.8	26	12.8	15
0.9	76	3.9	48	6.9	36	9.9	26	12.9	14
1.0	74	4.0	47	7.0	35	10.0	26	13.0	14
1.1	73	4.1	47	7.1	35	10.1	25	13.1	13
1.2	71	4.2	46	7.2	35	10.2	25	13.2	13
1.3	70	4.3	46	7.3	34	10.3	25	13.3	12
1.4	68	4.4	45	7.4	34	10.4	24	13.4	12
1.5	67	4.5	45	7.5	34	10.5	24	13.5	11
1.6	66	4.6	44	7.6	33	10.6	24	13.6	11
1.7	65	4.7	44	7.7	33	10.7	23	13.7	10
1.8	63	4.8	43	7.8	33	10.8	23	13.8	9
1.9	62	4.9	43	7.9	32	10.9	23	13.9	9
2.0	61	5.0	43	8.0	32	11.0	22	14.0	8
2.1	60	5.1	42	8.1	32	11.1	22	14.1	7
2.2	59	5.2	42	8.2	31	11.2	22	14.2	7
2.3	59	5.3	41	8.3	31	11.3	21	14.3	6
2.4	58	5.4	41	8.4	31	11.4	21	14.4	5
2.5	57	5.5	40	8.5	30	11.5	20	14.5	4
2.6	56	5.6	40	8.6	30	11.6	20	14.6	4
2.7	55	5.7	40	8.7	30	11.7	20	14.7	3
2.8	54	5.8	39	8.8	29	11.8	19	14.8	2
2.9	54	5.9	39	8.9	29	11.9	19	14.9	1
								15.0	0

H. PRECAUTIONS

1. Perform the test in a location free of vibrations, because vibrations may cause the suspended material to settle at a greater rate than normal.
2. Do not expose the plastic cylinders to direct sunlight any more than is necessary.
3. Frequently check the play between the cam and eccentric on the modified Tyler portable shaker by grasping one of the hanger rods and attempt to move the sieve base. If any play is noticed, replace the cam and/or bearing.
4. Lubricate the sieve shaker at least each three months.

REFERENCES

A California Test Method
Test Method No. Calif. 201
Test Method No. Calif. 202

End of Text on Calif. 229-C

12) *USING THE FHA SOIL PVC METER*

(Reproduced from "Guide to Use of the FHA Soil PVC Meter, by G.F. Henry and M.C. Dragoo, Federal Housing Administration, FHA No. 595, Jan. 1965, Washington).

A. General

The FHA Soil PVC Meter (Figure 2.5a) is used to perform a swell index test. This test is essentially a measurement of the pressure exerted by a sample of compacted soil when it swells against a restraining force after being wetted. The FHA Soil PVC Meter, in addition to yielding PVC values, can be used to estimate the plasticity index and shrinkage behavior of soils. These values are determined by comparing the results of the swell index test with appropriate values contained in Figures 2.5b, 2.5c, 2.5d and 2.5e in this guide and reading the corresponding extrapolations.

The following categories of PVC have been established:

PVC Rating	Category
Less than 2	Noncritical
2 to 4	Marginal
4 to 6	Critical
Greater than 6	Very critical

These ratings were established on the basis of the swelling and shrinking behavior of the soil.

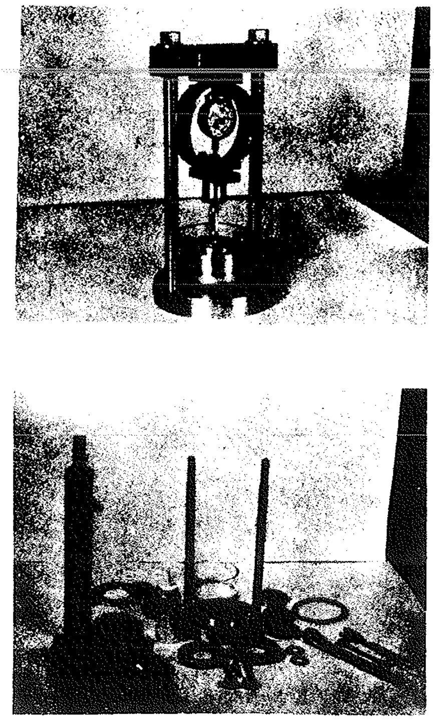

FIGURE 2.5a – PICTURES OF EQUIPMENT

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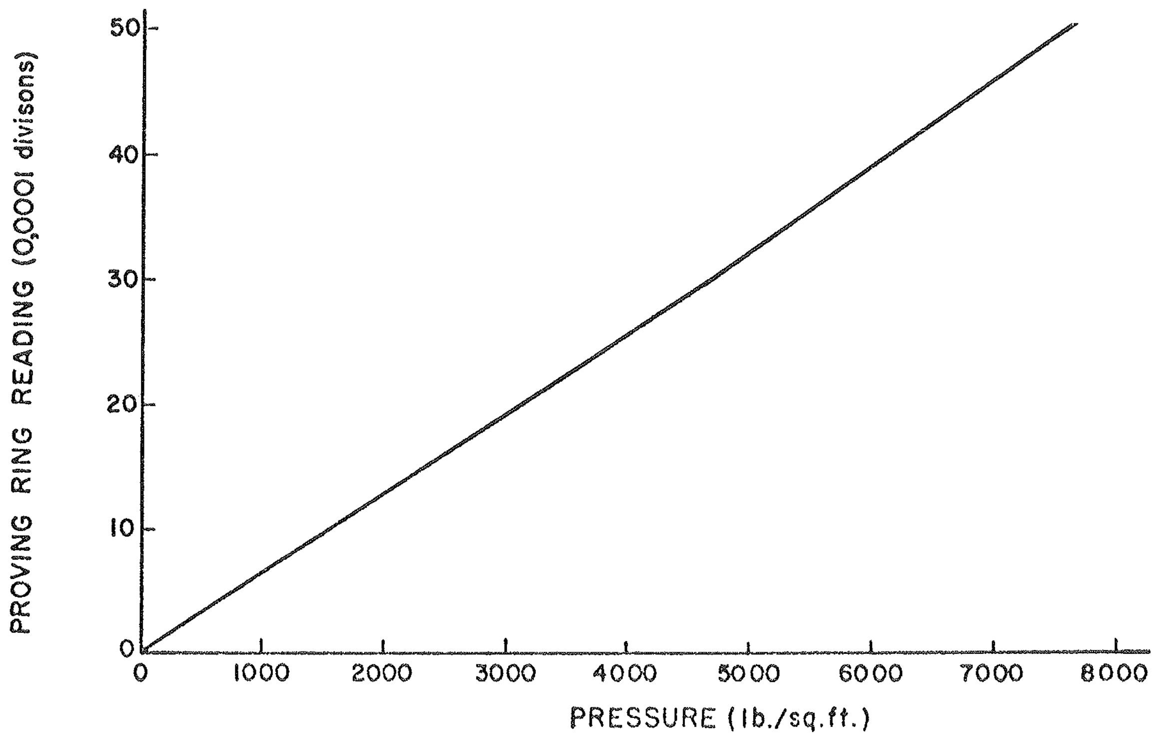

FIGURE 2.5b – PROVING RING CALIBRATION FROM "FHA SOIL PVC METER PUBLICATION" FEDERAL HOUSING ADMINISTRATION PUBLICATION Nº 701 (Lambe 1960)

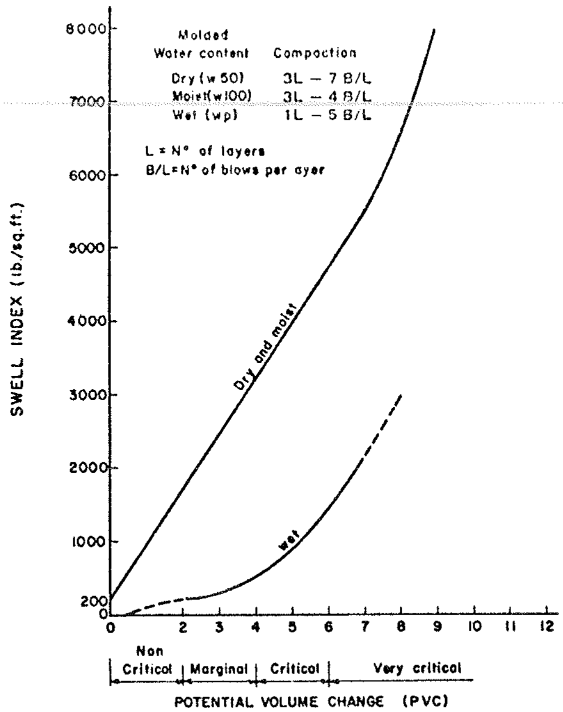

FIGURE 2.5c – SWELL INDEX VS. POTENTIAL VOLUME CHANGE FROM FEDERAL HOUSING ADMINISTRATION PUBLICATION No. 701 (Lambe, 1960)

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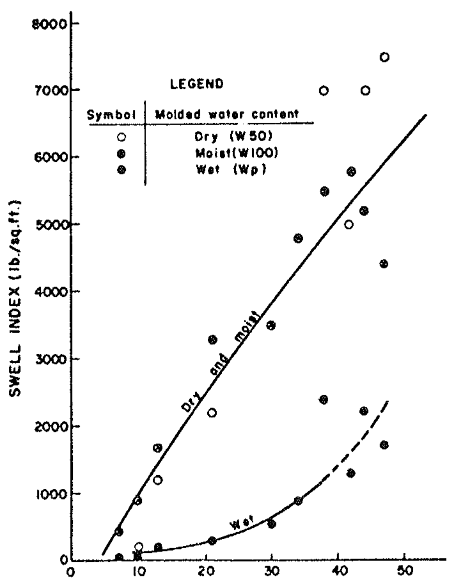

FIGURE 2.5d – SWELL INDEX VS. PLASTICITY FROM FEDERAL HOUSING ADMINISTRATION PUBLICATION No. 701 (Lambe, 1960)

B. Equipment

1. PVC Meter
2. Spacers, plate, and clamp for alternate compaction method.
3. No. 10 Sieve
4. Teaspoon
5. Compaction Hammer and Sleeve
6. Two Dry Porous Stones
7. Knife, (preferably serated)
8. Straight Edge
9. Water in Squirt Bottle with Pointed End
10. Wrenches

C. Preparation of Sample

For the test sample, take about a pint of soil from the soil layer in which the foundation member will rest. Although samples can be tested at three relative water contents (dry, moist, or wet), it is suggested that those being tested for FHA purposes be tested in the air dried condition only. The samples can be sufficiently air dried by breaking the soil into small lumps and leaving it in the sun for a few hours. The following procedures are for soil in the air dried condition. For information about soil in other conditions, see Lambe (1960).

D. Preparation for Compaction

Disassemble the PVC Meter with exception of the rods which can remain screwed into the base. Place proving ring and top bar where it will not be jarred during compaction. Wipe equipment with clean cloth.

E. Compaction

DEFINITIONS:

Compaction ring largest ring; identified by letter "c" etched on outside periphery.

Spacer ring smallest ring; identified by letter "s" etched on outside periphery.

1. To assemble meter for compaction, fit compaction ring on base so that "c" is backwards and at the top. Align bolt holes with those inbase. Place spacer ring on compaction ring so that "s" is at the top (radial grooves are at top). Align bolt holes with those in base. Insert the 3 bolts through both rings and the base and tighten firmly to base.

2. The soil sample is to be placed in the ring assembly in 3 layers of equal amounts. Each layer is to be compacted separately. Compaction is accomplished by use of the hammer, which is a tamping device encased in a metal sleeve.

3. Compact each layer of the sample in the following manner:

   a. Place 3 heaping teaspoonsfull of sample in ring assembly and smooth lightly with hammer to firm up the surface before applying the blows (This reduces the amount of soil "jumping" out of the mold during compaction). Place apparatus on a solid level floor.

b. Before each blow, lift sleeve 1/8 inch from soil and hold firmly against the inside of the spacer ring. *Make sure sleeve of hammer rests inside rings so that hammer does not damage them in falling.* Be sure to hold sleeve and hammer perpendicular and in line with supporting rods. Raise hammer to top of sleeve and let it fall free (not striking sides of sleeve). Space blows evenly over surface of sample by shifting hammer after each blow. Compact the first two layers with 7 blows each of the compaction hammer and the last with 8 blows. Repeat this process for each layer. (See F for Alternate Compaction Method).

4. At completion of the compaction of both the first and second layers scratch the top surface of the layer with a knife to assure proper bond with the next layer. After compaction, the last layer should extend approximately 1/4 inch into the spacer ring. If it is significantly below this point, remove entire sample and recompact.

5. Put assembly on table and remove the 3 bolts. Rotate spacer ring (to break bond between ring and soil) and remove carefully from base. Remove compaction ring containing sample in same way. Do not tilt compaction ring or spill soil.

6. Trim top of the sample with a knife. Hold knife against the compaction ring at all times during trimming to avoid dislodging sample. Trim in a sawing motion taking off only a small amount of soil at a time. Rotate the ring as you trim. Work from the edge toward the center. When sample is almost level, do final leveling by drawing a metal straight edge over sample.

7. The final surface of the soil sample should be firm and smooth. Any voids should be filled by pressing additional soil into them with the knife or spoon.

8. Clean soil from base and from all holes in rings and base. Remove soil in the groove of the spacer ring and from the holes in the spacer ring and the compaction ring with a toothpick or paperclip.

F. Alternate Compaction Method

1. After fitting rings to base as explained in E, paragraph 1, place one spacer on each rod, then set the plate on the spacers. Bolt these securely to the rods. Attach the clamp to the sleeve so that the sleeve extends about 1/4 inch insider the spacer ring. Place the soil sample in the ring assembly in the same manner as explained in E, paragraphs 1 and 3[a].

2. Before each blow, turn the "foot" of the clamp so that it points in the direction of the spot to be compacted. The sleeve and hammer must be held perpendicular and in line with the supporting rods. To assure this, the sleeve should be held firmly against the inside of the plate and the spacer ring. Raise hammer to top of sleeve and let it fall free (not striking sides of sleeve). Space blows evenly over surface of sample by shifting hammer after each blow. Compact the first two layers with 7 blows each of the compaction hammer and the last with 8 blows. Repeat this process for each layer.

3. The remaining compaction process is the same as E, paragraphs 4 through 8.

G. Swelling

1. Place spacer ring on base with "s" (and radial grooves) on top. Align bolt holes with those on base. Place *thoroughly dry* porous stone in spacer ring. Move assembled base to edge of working table. Place thumb under base and other fingers over spacer ring and stone, holding them firmly in place. Turn base upside-down retaining firm hold on stone and spacer ring. Pick up compaction ring containing sample trimmed side up and place flush against porous stone in spacer ring aligning bolt holes in the two rings. Move compaction ring with as little disturbance of sample as possible. Turn base with rings, stone, and sample rightside-up. Bolt rings tightly to base.

2. Place a dry porous stone on top of sample inside compaction ring. Place the rubber 0-ring on the base and screw the lucite container onto it tightly to insure water seal. Place metal cover on porous stone with the center indentation at the top.

3. Place top bar with proving ring on the steel rods (Be sure that the adjustable rod which extends down from the proving ring dial does not strike the cover). Add washers and nuts and tighten firmly.

4. Set proving ring dial to zero by moving the band around the dial. Tighten dial with the screw on band. Push up on proving ring dial to see that it appears to work properly. Turn adjustable rod exactly into the center of the indentation on top of the cover. Be sure that the cover is centered exactly over the stone. Tighten lock nut on adjustable rod firmly. Be sure adjustable rod does not stick in cover (receptable for adjustable rod may require slight enlargement). Turn adjustable rod until dial reads one division past zero. Tighten lock nut firmly again until adjustable rod has no play. 129

5. Record the time and the proving ring reading. Add water to sample by squeezing from squirt bottle into the holes located at the top of compaction ring until water level in lucite container has covered the spacer ring and tops of the bolts. (This procedure is used to reduce the amount of air entrapped in the ring assembly and thus insures that the sample has uniform access to water over its entire top and bottom surfaces).

H. Reading

1. Allow soil to expand until completely stabilized or for a maximum of 2 hours, then read dial to obtain PVC swell index value. On the dial the number 1 equals 10 divisions, the number 2 equals 20, etc.

2. Next, find the number corresponding to the proving ring dial reading on Figure 2.5b and subtract the one division that registered on the dial prior to swell. Read horizontally to intersection with sloping line. From point of intersection, read downward to baseline which indicates pressure in lbs./sq. ft.

3. Take this figure to Figure 2.5c find the number corresponding to it on left hand side of the chart. Read horizontally to intersection with the sloping line marked "Dry and Moist". From point of intersection, red downward to the baseline, which indicates PVC category.

4. Take the reading in lbs./sq.ft. to Figure 2.5d to determine the plasticity index.

5. It is also possible to obtain the approximate PVC category and plasticity index by taking the reading from the proving ring dial directly to Table 2.3.

TABLE 2.3
Table for Converting Proving Ring Readings to PVC Category and Approximate Plasticity Index

PROVING RING READING	SWELL INDEX (#/SF)	PVC CATEGORY	PLASTICITY INDEX (%)
5	775	0.8	8.5
6	925	1.0	9.5
7	1 075	1.2	10.7
8	1 250	1.4	11.7
9	1 375	1.6	12.7
10	1 550	1.8	13.8
10.8	1 675	2.0	14.6
11	1 700	2.0	14.8
12	1 875	2.2	15.8
13	2 025	2.4	17.0
14	2 175	2.65	18.0
15	2 350	2.85	19.0
16	2 500	3.05	20.0
17	2 675	3.3	21.5
18	2 800	3.45	22.5
19	2 975	3.7	23.8
20	3 150	3.9	25.0
20.3	3 200	4.0	25.5
21	3 300	4.1	26.0
22	3 450	4.3	27.5
23	3 600	4,5	28.5
24	3 775	4.75	29.8
25	3 925	4.95	30.8
26	4 075	5.15	31.8
27	4 225	5.4	33.0
28	4 375	5.55	34.0
29	4 525	5.75	35.3
30	4 700	5.95	37.0
30.2	4 725	6.00	37.1
31	4 850	6.2	38.0
32	4 975	6.35	39.0
33	5 125	6.5	40.4
34	5 275	6.7	41.7
35	5 425	6.9	43.4
36	5 575	7.1	44.2
37	5 725	7.25	45.5
38	5 850	7.4	46.6
39	6 000	7.5	48.0
40	6 150	7.65	49.5
40.5	6 225	7.7	50.0

Prepared by the Architectural Section, Federal Housing Administration Insuring Office San Antonio, Texas.

13) *SUGGESTED METHOD OF TEST OF ONE-DIMENSIONAL EXPANSION AND UPLIFT PRESSURE OF CLAY SOILS*

(Reproduced from "Special Procedures for Testing Soil and Rock for Engineering Purposes", 5th ed., ASTM Special Technical Publication 479, June 1970).

1. Scope

1.1 This method explains how to make expansion tests on undisturbed or compacted clay soil samples that have no particle sizes greater than $\frac{3}{16}$ in. (passing the No. 4 standard ASTM sieve). The test is made to determine (1) magnitude of volume change under load or no-load conditions, (2) rate of volume change, (3) influence of wetting on volume change, and (4) axial permeability of laterally confined soil under axial load or no-load during expansion. Saturation (no drainage) takes place axially. Permeant water is applied axially for determining the effect of saturation and permeability. The specimens prepared for this test may also be used to determine the vertical or volume shrinkage as the water content decreases. Total volume change for expansive soils is determined from expansion plus shrinkage values for different ranges of water content.

1.2 Expansion test data may be used to estimate the extent and rate of uplift in subgrades beneath structures or in structures formed from soils, and shrinkage tests may be used to estimate the volume changes which will occur in soils upon drying, provided that natural conditions and operating conditions are duplicated.

2. Significance

2.1 The expansion characteristics of a soil mass are influenced by a number of factors. Some of these are size and shape of the soil particles, water content, density, applied loadings, load history and mineralogical and chemical properties. Because of the difficulty in evaluating these individual factors, the volume-change properties cannot be predicted to any degree of accuracy unless laboratory tests are performed. When uplift problems are critical, it is important to test samples from the sites being considered.

2.2. The laboratory tests described herein are primarily intended for the study of soils having no particles larger than the No. 4 standard sieve size ($\frac{3}{16}$ in.). If the test is made on the minus No. 4 fraction of soils containing gravel material (plus No. 4), some adjustment is required in any analysis. Gravel reduces volume change because it replaces the more active soil fraction.

3. Apparatus

3.1 *Consolidometer* – Conventional laboratory consolidometers are used for the expansion test. Consolidometers most used in the United States are of the fixed-ring and floating-ring types. Figure 2.6 illustrates the fixed-ring type. Either of these is suitable. Both types are available commercially. In the fixed-ring container, all specimen movement relative to the container is upward during expansion. In the floating-ring container, movement of the soil sample is from the top and bottom away from the center during expansion. The specimen containers for the fixed-ring consolidometer and the floating-ring consolidometer consist of brass or plastic rings, and other component parts. Sizes of container rings most commonly used vary between 4 ¼ -in. diameter by 1 ¼ -in. deep and 2 ½ -in. diameter by 3/4-in. deep, although other sizes are used. However, the diameter should be not less than 2 in. and the depth not greater than three tenths of the diameter, except

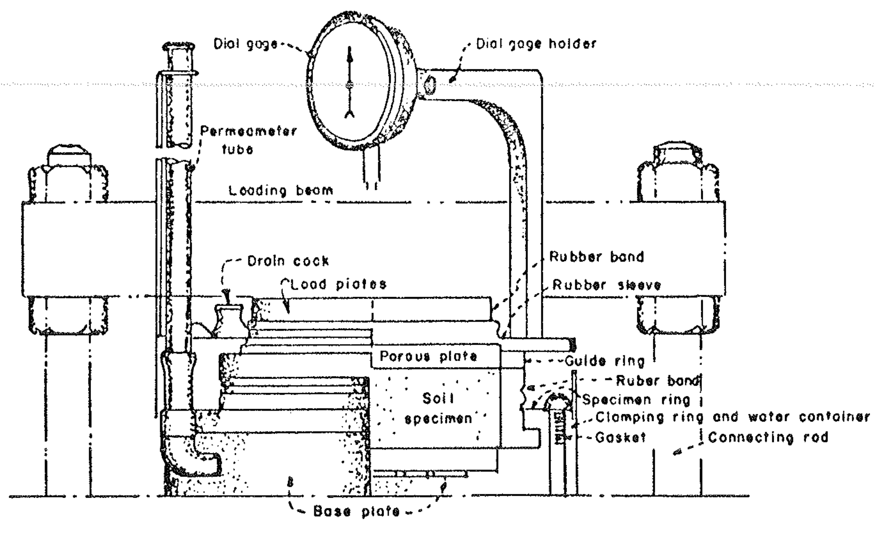

FIGURE 2.6 – FIXED-RING CONSOLIDOMETER

that the depth must not be less than 3/4 in. for specimens of small diameter. Lesser depths introduce errors caused by the magnitude of surface disturbance, while large depths cause excessive side friction. For expansion tests the larger diameter consolidation rings are preferred as they restrain the soil action to a lesser degree. In a test using the floating-ring apparatus, the friction between the soil specimen and container is smaller than with the fixed-ring apparatus. On the other hand, the fixed-ring apparatus is more suitable for saturation purposes and when permeability data are required. Porous stones are required at the top and bottom of the specimen to allow application of water. The apparatus must allow vertical movement of the top porous stone for fixed-ring consolidometers, or vertical movement for top and bottom porous stones for floating-ring consolidometers, as expansion takes place. A ring gage machined to the height of the ring container to an accuracy of 0.001 in. is required; thus, the ring gage for $1\frac{1}{4}$-in. high specimens will have a height of 1.250 in. Measure the diameter of the specimen container ring to 0.001 in.

3.2 Loading Device – A suitable device for applying vertical load to the specimen is required. The loading device may be platform scales of 1000 to 3000 lb capacity mounted on a stand and equipped with a screw jack attached underneath the frame. The jack operates a yoke which extends up through the scale platform and over the specimen container resting on the platform. The yoke is forced up or down by operating the jack, thus applying or releasing load to the soil specimen. The desired applied pressure, which is measured on the scale beam, becomes fully effective when the beam is balanced.

3.2.1 Another satisfactory loading device utilizes weights and a system of levers for handling several tests simultaneously. Hydraulic-piston or bellows-type loading apparatus are also very satisfactory if they have adequate capacity, accuracy, and sensitivity for the work being performed. Apparatus such as described in ASTM Method D 2435, Test for One-Dimensional Consolidation Properties of Soils[4] is satisfactory and may be used.

3.3 Device for Cutting Undisturbed Specimens – This apparatus consists of a cutting bit of the same diameter as the ring container of the consolidometer, a cutting stand with bit guide, and knives for trimming the soil. Wire saws or trimming lathes may be used if a uniform tight fit of the specimen to the container is obtained. 131

3.4 *Device for Preparation of Remolded Specimens* – Compacted soil specimens are prepared in the consolidometer ring container. In addition to the container, the apparatus consists of an extension collar about 4 in. in depth and of the same diameter as the container. A compaction hammer of the same type required in Method A of ASTM Method D 698, Test for Moisture-Density Relations of Soils, Using 5.5 lb Rammer and 12-in. Drop.[4]

4. Procedure-Expansion Test

4.1 *Preparation of Undisturbed Specimens* – Perform the tests on hand-cut cube samples or core samples of a size that will allow the cutting of approximately 1/2-in. of material from the sides of the consolidometer specimen. (Alternatively, obtain a core of a diameter exactly the same as the diameter of the consolidometer specimen container and extrude the core directly into the container. This procedure is satisfactory provided that the sampling has been done without any sidewall disturbance and provided that the core specimen exactly fits the container. Place the undistrubed soil block or core on the cutting platform, fasten the cutting bit to the ring container and place the assembly on the sample in alignment with the guide arms. With the cutting stand guiding the bit, trim the

4 Annual Book of ASTM Standards, Part 11.

excess material with a knife close to the cutting edge of the bit, leaving very little material for the bit to shave off as it is pressed gently downward. (Other suitable procedures to accommodate guides for wire saws, trimming lathes, or extrusion devices may be used in conformance with the use of alternative apparatus and samples). In trimming the sample, be careful to minimize disturbance of the soil specimen and to assure an exact fit of the specimen to the consolidometer container. When sufficient specimen has been prepared so that it protrudes through the container ring, trim it flush with the surface of the container ring with a straightedge cutting tool. Place a glass plate on the smooth, flat cut surface of the specimen, and turn the container over. Remove the cutting bit, trim the specimen flush with the surface of the container ring, and cover it with a second glass plate to control evaporation until it is placed in the loading device.

4.2 *Preparation of Remolded Specimens* – Use about 2 lb of representative soil that has been properly moistened to the degree desired and processed free from lumps and from which particles or aggregations of particles retained by a $\frac{3}{16}$ -in. (No. 4) sieve have been excluded. Compact the specimen to the required wet bulk density after adding the required amount of water as follows: Place the extension collar on top of the container ring and fasten the bottom of the container ring to a baseplate. Weigh the exact quantity of the processed sample to give the desired wet density when compacted to a thickness 1/4 in. greater than the thickness of the container ring. Compact the specimen to the desired thickness by the compaction hammer. Remove the extension collar and trim the excess material flush with the container ring surface with a straightedge cutting tool. Remove the ring and specimen from the baseplate and cover the specimen surfaces with glass plates until the specimen is placed in the loading device. If, after weighing and measuring the specimen and computing the wet density, as described below, the wet density is not within 1.0 lb/ft^3 of that required, repeat the preparation of the remolded specimen until the required accuracy is obtained.

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4.3 *Calibration of Dial Gage for Height Measurements* – Prior to filling the container ring with the soil specimen, place a ring gage in the specimen container with the same arrangement of porous plates and load plates to be used when testing the soil specimen. Place the assembly in the loading machine in the same position it will occupy during the test. After the apparatus has been assembled with the ring gage in place, apply a load equivalent to a pressure of 0.35 psi (or 0.025 kgf/cm^2) on the soil specimen. The dial reading at this time will be that for the exact height of the ring gage. Mark the parts of the apparatus so that they can be matched in the same position for the test.

4.4 *Initial Height and Weight of Soil Specimen* .– Clean and weigh the specimen container ring and glass plates and weigh them to ± 0.01 g before the ring is filled. After filling and trimming is completed, weigh the soil specimen, ring, and glass plates to ± 0.01 g. Determine the weight of the soil specimen. Assemble the specimen container and place it in the loading device. If the specimen is not to be saturated at the beginning of the test, place a rubber sleeve around the protruding porous plates and load plates to prevent evaporation. Apply the small seating load of 0.35 psi (or 0.025 kgf/cm^2) to the specimen. By comparing the dial reading at this time with the dial reading obtained with the ring gage in place, determine the exact height of the specimen. Use this information to compute the initial volume of the specimen, the initial density, void ratio, water content, and degree of saturation. The true water content of the specimen will be determined when the total dry weight of the specimen is obtained at the end of the test.

4.5 *Saturation and Permeability Data* – To saturate the specimen attach the percolation tube standpipe, fill it with water, and wet the specimen. Take care to remove any air that may be entrapped in the system by slowly wetting the lower porous stone and draining the stone through the lower drain cock. After the specimen is wetted, fill the pan in which the consolidometer stands with water. After saturation has been completed, permeability readings can be taken at any time during the test by filling the percolation tube standpipe to an initial reading and allow the water to percolate through the specimen. Measure the amount of water flowing through the sample in a given time by the drop in head.

4.6 *Expansion Test:*

4.6.1 *General Comments* – The expansion characteristics of an expansive-type soil vary with the loading history, so that it is necessary to perform a separate test or several specimens for each condition of loading at which *exact* expansion data are required. However, one procedure is to test only two specimens: (1) loaded-and-expanded, and (2) expanded-and-loaded. From these data, an estimate of expansion can be made for any load condition as shown by Curve C, Figure 2.7, in which Specimen No. 1 was loaded and expanded by saturation with water, (Curve B) and Specimen No. 2 was expanded by saturation with water and then loaded (Curve A).

4.6.2 *Loaded and Expanded Test* – To measure expansion characteristics where the soil specimen is saturated under full load and then allowed to expand, apply the seating load of 0.35 psi (or 0.025 kgf/cm^2) to Specimen No. 1, and secure initial dial readings. Then saturate the soil specimen as described in 4.5. (The permeameter tube head should be sufficiently low so that the specimen is not lifted). As the specimen begins to expand, increase the load as required to hold the specimen at its original height. Then reduce the load to 1/2, 1/4, and 1/8 of the maximum load and finally to the seating load of 0.35 psi (or

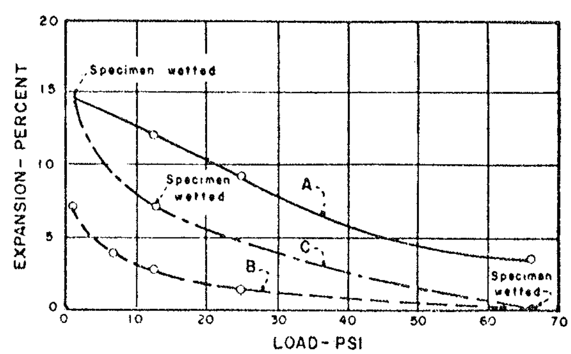

FIGURE 2.7 – LOAD-EXPANSION CURVES

0.025 kgf/cm^2) and measure the height with each load. Use a greater number of loadings if greater detail in the test curve is required. Maintain all loads for 24 h, or longer if needed, to obtain constant values of height. Remove the specimen from the ring container and weigh it immediately and again after drying to 105 C. From the water content, dry bulk density, and specific gravity of the specimen, calculate the volume of air and, assuming it to be the same as the volume of air following the determination of permeability, calculate the water content and degree of saturation.

4.6.3 *Expanded and Loaded Test* – To measure expansion characteristics where the soil is allowed to expand before loading, apply the seating load of 0.35 psi (or 0.025 kgf/cm^2) to Specimen No. 2, and secure initial dial gage readings. Then saturate the specimen as described in 4.5. Allow the specimen to expand under the seating load for 48 h or until expansion is complete. Load the specimen successively to 1/8, 1/4, 1/2 and 1 times the maximum load found in 4.6.2, to determine the reconsolidation characteristics of the soil. Use a greater number of loadings, if greater detail in the test curve is required. Follow the procedures specified in 4.6.2 for making loadings and all measurements and determinations.

4.6.4 *Individual Load-Expansion Test* – When it is desired to perform separate expansion tests for other conditions of loading aplly the seating load of 0.35 psi (or 0.025 kgf/cm^2) to the specimen and measure the initial height. Then load the specimen to the desired loading, saturate the specimen as described in 4.5, and allow the specimen to expand under the applied load for 48 h, or until expansion is complete. Measure the height of the expanded specimen. Reduce the load to that of the seating load. Allow the height to become constant and measure; then remove the specimen from the ring and make the determination specified in 4.6.2.

5. Procedure – Shrinkage Test

5.1 *Specimen Preparation* – When measurements of shrinkage on drying are needed, prepare an additional specimen as described in 4.1 or 4.2. Cut this specimen from the same undisturbed soil sample as the expansion specimens, or remolded to the same bulk density and water content conditions as the expansion specimens. Place the specimen in the container ring, and measure the initial volume and height as described in 4.4. Determine the water content of the soil specimen by weighing unused portions of the original sample of which the specimen is a part, drying the material in an oven to 105 C, and reweighing it.

5.2 *Volume and Height Shrinkage Determinations* – To measure volume shrinkage, allow the specimen in the ring to dry in air completely or at least to the water content corresponding to the shrinkage limit (ASTM Method D 427, Test for Shrinkage Factors of Soils). After the specimen has been air-dried, remove it from the ring container, and obtain its volume by the mercury-displacement method.

5.2.1 To perform the mercury displacement measurement, place a glass cup with a smoothly ground top in an evaporating dish. Fill the cup to overflowing with mercury, and then remove the excess mercury by sliding a special glass plate with three prongs for holding the specimen in the mercury over the rim. Pour the excess mercury into the original container and replace the glass cup in the evaporating dish. Then immerse the air-dried soil specimen in the glass cup filled with mercury using the special glass plate over the glass cup to duplicate the initial mercury volume determination condition. (See Method D 427 for general scheme of test and equipment.) Transfer the displaced mercury into a graduated cylinder, and measure the volume. If the shrinkage specimen is cracked into separate parts, measure the volume of each part, and add the individual volumes to obtain the total. (A paper strip wrapped around the specimen side and held by a rubber band is effective in holding the specimen intact during handling).

5.2.2 If the height of the air-dried specimen is desired, place the specimen and ring container in the loading machine. Apply the seating lòad of 0.35 psi (or 0.025 kgf/cm^2), and then read the dial gage.

6. Calculations

6.1 *Expansion Test Data* – Calculate the void ratio as follows:

$$e = \frac{\text{volume of voids}}{\text{volume of solids}} = \frac{h - h_o}{h_o}$$

where:

e = void ratio,
h = height of the specimen, and
h_o = height of the solid material at zero void content

Calculate the expansion, as a percentage of the original height, as follows:

$$\Delta e \text{ percent} = \frac{h_2 - h_1}{h_1} \times 100$$

where:

Δe = expansion in percentage of initial volume,
h_1 = initial height of the specimen, and
h_2 = height of the specimen under a specific load condition.

6.2 *Permeability Test Data* – Calculate the permeability rate by means of the following basic formula for the variable head permeameter:

$$k = \frac{A_p \times L_s}{A_s \times 12} \times \frac{1}{t} \ln \frac{H_i}{H_f}$$

where:

k = permeability rate, ft/year,
A_p = área of standpipe furnishing the percolation head, in.^{2},
A_s = area of the specimen, in.^{2},
L_s = length of the specimen, in.,
H_i = initial head, difference in head between headwater and tailwater, in.,
H_f = final head, difference in head between headwater and tailwater, in., and
t = elasped time, years.

6.3 *Shrinkage Test Data* – Calculate the volume shrinkage as a percentage of the initial volume as follows:

$$\Delta_s = \frac{v_i - v_d}{v_i} \times 100$$

where:

Δ_s = volume shrinkage in percentage of initial volume,
v_i = initial volume of specimen (height of specimen times area of ring container), and
v_d = volume of air-dried specimen from mercury displacement method.

Calculate the shrinkage in height as follows:

$$\Delta hs = \frac{h_i - h_d}{h_i} \times 100$$

where:

Δhs = height of shrinkage in percentage of initial height,
h_i = initial height of specimen, and
h_d = height of air-dried specimen.

6.3.1 To calculate the total percentage change in volume from "air-dry to saturated conditions," add the percentage shrinkage in volume on air drying Δs to the percentage expansion in volume on saturation Δe, as described in 6.1. This value is used as an indicator of total expansion but is based on initial conditions of density and water content. Since expansion volume data are determined for several conditions of loading, the total volume change can also be determined for several conditions of loading.

6.3.2 To calculate the total percentage change in height from saturated to air-dry conditions, add the percentage shrinkage in height Δhs to the percentage expansion Δe when the specimen is saturated under specific load conditions.

7. Plotting Test Data

7.1 *Expansion Test* – The test data may be plotted as shown on Figure 2.7.

8. Reports

8.1 *Expansion Test* – Include the following information on the soil specimens tested in the report:

8.1.1 Identification of the sample (hole number, depth, location).

8.1.2 Description of the soil tested and size fraction of the total sample tested.

8.1.3 Type of sample tested (remolded or undisturbed; if undisturbed, describe the size and type, as extruded core, hand-cut, or other).

8.1.4 Initial moisture and density conditions and degree of saturation (if remolded, give the comparison to maximum density and optimum water content (see Methods D 698)).

8.1.5 Type of consolidometer (fixed or floating ring, specimen size), and type of loading equipment.

8.1.6 A plot load versus volume change curves as in Fig. 1. A plot of void ratio versus log of pressure curve may be plotted if desired.

8.1.7 A plot log of time versus deformation if desired.

8.1.8 Load and time versus volume-change data in other forms if specifically requested.

8.1.9 Final water content, bulk dry density, and saturation degree data.

8.1.10 Permeability data and any other data specifically requested.

8.2 *Shrinkage Test* – For the report on shrinkage, include data on the decrease in volume from the initial to air-dried condition and, if desired, other information such as the total change in volume and total change in height. Report the load conditions under which the volume change measurements were obtained. Include also Items 8.1.1 through 8.1.5 and 8.1.9.

STANDARD SPECIFICATIONS

for

TRANSPORTATION MATERIALS

and

METHODS OF SAMPLING AND TESTING

PART II

METHODS OF SAMPLING AND TESTING

Adopted by
THE AMERICAN ASSOCIATION OF STATE
HIGHWAY AND TRANSPORTATION OFFICIALS

JULY 1978

TWELFTH EDITION

PUBLISHED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
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WASHINGTON, D.C. 20001

INTRODUCTION

PART II
METHODS OF SAMPLING AND TESTING

This Twelfth Edition of Transportation Materials is published in two parts, one dealing with specifications for materials and the second with methods of testing and specifications for testing equipment. Part II covering methods of testing and specifications for testing equipment, contains 195 Test Methods. Revisions have been made in a number of the test methods shown in the last edition, 14 new test methods have been added and 12 test methods have been deleted.

As stated in previous editions, the Association has followed the policy of indicating, for naturally occurring materials, test limits which may be considered the most liberal that may safely be allowed. This has been done with the understanding that where higher grade materials are locally available, more rigid requirements should be inserted. This policy has been followed in recognition of the necessity of adjusting test requirements to meet local demands. However the recommended test limits covering manufactured products such as cement, steel, asphalt, etc., may be considered as definite requirements for the materials for specific uses and under specific conditions, and not subject to modification in the same sense as would justify modifications in specifications for naturally occurring materials.

Many of these specifications agree with those of the American Society for Testing and Materials. In all cases where the Association and Society standards are technically identical, reference to the ASTM designation number is shown in the heading of the specification. Where the Association has adopted an ASTM standard, the courtesy of the Society in permitting publication of the standard is appreciated.

General jurisdiction over Association standards in this field is a function of the Subcommittee on Materials, which has members representing each of the 50 States, the Commonwealth of Puerto Rico, the District of Columbia, and the United States Department of Transportation. A number of specifications have been included in this publication at the request of the AASHTO Subcommittee on Bridges and Structures.

Interim Specifications are published each year, and a revised edition of this book is published every four years. The Interim Specifications have the same status as standards of the American Association of State Highway and Transportation Officials, but are tentative revisions approved by at least two-thirds of the Subcommittee on Materials. These revisions are voted on by the Association Member Departments prior to the publication of each new edition of this book, and if approved by at least two-thirds of the members, they are included in the new edition as standards of the Association.

Criticisms of these specifications are welcome and should be addressed to the Executive Director, AASHTO, 444 North Capitol St., N.W., Suite 225, Washington, D.C. 20001.

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CONCRETE

BRICK

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NUMERICAL SEQUENCE

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As an aid to the user of this volume, the following tabulation shows equivalences between AASHTO and ASTM specifications. Where an asterisk (*) follows the ASTM number, there is a difference between the two specifications. These differences are explained in the footnote accompanying each individual specification.

AASHTO Designation	*ASTM Designation*	*AASHTO Designation*	*ASTM Designation*
T19-76	C29-76	T132-74	C190-72
T20-42	C30-37 (1970)	T133-74	C188-72
T21-78	C40-73*	T137-74	C185-71
T22-74	C39-72	T140-70 (1978)	C116-68 (1974)
T23-76	C31-69 (1975)*	T141-74	C172-71
T24-78	C42-68 (1974)	T142-74	C70-72*
T32-70	C67-66	T143-78	D345-74
T44-78	D2042-76*	T144-74	D806-74
T47-76	D6-67 (1973)*	T148-49 (1978)	C174-49 (1975)
T48-78	D92-72 (1977)*	T151-78	D555-75
T49-78	D5-73*	T152-76	C231-75
T50-78	D139-77*	T153-74	C204-72
T51-74	D113-76*	T154-74	C266-71
T52-74	D20-72*	T155-74	C156-74
T53-78	D2398-76*	T157-78	C233-76
T54-61 (1974)	D1665-61 (1973)	T158-74	C232-71
T55-78	D95-70 (1975)*	T159-74	C234-71
T59-78	D244-72*	T160-76	C157-75
T60-42 (1974)	D38-33 (1970)	T161-76	C666-77
T61-70	D168-67 (1973)	T162-65 (1978)	C305-65 (1975)
T62-70	D246-67	T164-76	D2172-67*
T65-78	A90-69 (1973)*	T165-77	D1075-75*
T67-74	E4-72	T167-78	D1074-76*
T68-74	E8-69	T168-55 (1974)	D979-51 (1968)
T70-77	E10-73	T170-73	D1856-69
T71-76	C87-69 (1975)	T172-78	D290-67 (1974)
T72-78	D88-56 (1973)*	T177-68 (1978)	C293-68 (1974)
T73-77	D93-77*	T178-66 (1974)	C85-66 (1973)
T74-70	D369-67 (1973)	T179-76	D1754-76*
T78-78	D402-76*	T182-70	D1664-69 (1975)
T80-76	E18-74	T185-74	C359-69
T81-70	D367-67	T186-74	C451-72
T82-70	D368-67	T187-60 (1974)	D1191-64 (1976)
T83-70	D370-67	T189-70	C235-68
T84-77	C128-73*	T190-78	D2844-69 (1975)
T85-77	C127-77*	T191-61 (1974)	D1556-64 (1974)
T86-74	D420-69 (1975)	T192-74	C430-71
T97-76	C78-75	T195-67 (1978)	D2489-67 (1974)
T98-74	C115-70	T196-76	C173-75
T100-75	D854-58 (1972)*	T197-74	C403-77
T105-73	C114-69*	T198-74	C496-71
T106-77	C109-70	T200-70	E70-68
T107-74	C151-71	T201-76	D2170-76*
T110-70	D1461-60 (1973)	T202-77	D2171-66 (1972)*
T112-78	C142-71*	T203-64 (1974)	D1452-65 (1972)
T113-78	C123-69 (1975)*	T205-64 (1974)	D2167-66 (1972)
T115-78	D86-77	T207-75	D1587-74*
T119-74	C143-74	T208-70	D2166-66 (1972)
T121-76	C138-75	T209-74 (1978)	D2041-71 (1976)
T126-76	C192-76*	T213-78	A428-68 (1973)
T127-74	C183-73	T215-70	D2434-68 (1974)
T128-67 (1978)	C184-66	T216-74	D2435-70
T129-74	C187-71	T221-66 (1974)	D1195-64 (1971)
T131-74	C191-71	T226-68	D2664-67 (1974)

AASHTO Designation	ASTM Designation	AASHTO Designation	ASTM Designation
T227-68	D1298-67 (1977)	T242-78	E274-77*
T228-78	D70-76*	T243-77	A673-75
T229-74	D71-72a	T244-78	A370-76
T231-74	C617-76	T245-78	D1559-76*
T235-74	D1194-72	T246-74	D1560-71
** T238-76	D2922-71 (1976)	T247-74	D1561-71
** T239-76	D3017-72	T255-76	C566-67 (1972)*
T240-78	D2872-74*		

As an aid to the user of this volume, the following tabulation shows equivalences between AASHTO and ASTM specifications. Where an asterisk (*) follows the ASTM number, there is a difference between the two specifications. These differences are explained in the footnote accompanying each individual specification.

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A90-69 (1973)*	T65-78	C204-72	T153-74
A370-76	T244-78	C231-75	T152-76
A428-68 (1973)	T213-78	C232-71	T158-74
A673-75	T243-77	C233-76	T157-78
C29-76	T19-76	C234-71	T159-74
C30-37 (1970)	T20-42	C235-68	T189-70
C31-69 (1975)*	T23-76	C266-71	T154-74
C39-72	T22-74	C293-68 (1974)	T177-68 (1978)
C40-73*	T21-78	C305-65 (1975)	T162-65 (1978)
C42-68 (1974)	T24-78	C359-69	T185-74
C67-66	T32-70	C403-77	T197-74
C70-72*	T142-74	C430-71	T192-74
C78-75	T97-76	C451-72	T186-74
C85-66 (1973)	T178-66 (1974)	C496-71	T198-74
C87-69 (1975)	T71-76	C566-67 (1972)	T255-76
C109-70	T106-77	C617-76	T231-74
C114-69*	T105-73	C666-77	T161-76
C115-70	T98-74	D5-73*	T49-78
C116-68 (1974)	T140-70 (1978)	D6-67 (1973)*	T47-76
C123-69 (1975)*	T113-78	D20-72	T52-74
C127-77*	T85-77	D38-33 (1970)	T60-42 (1974)
C128-73*	T84-77	D70-76*	T228-78
C138-75	T121-76	D71-72a	T229-74
C142-71*	T112-78	D86-77	T115-78
C143-74	T119-74	D88-56 (1973)*	T72-78
C151-71	T107-74	D92-72 (1977)*	T48-78
C156-74	T155-74	D93-77*	T73-77
C157-75	T160-76	D95-70 (1975)	T55-78
C172-71	T141-74	D113-76*	T51-74
C173-75	T196-76	D139-77*	T50-78
C174-49 (1975)	T148-49 (1978)	D168-67 (1973)	T61-70
C183-73	T127-74	D244-72*	T59-78
C184-66	T128-67 (1978)	D246-67	T62-70
C185-71	T137-74	D290-67 (1974)	T172-78
C187-71	T129-74	D345-74	T143-78
C188-72	T133-74	D367-67	T81-70
C190-72	T132-74	D368-67	T82-70
C191-71	T131-74	D369-67 (1973)	T74-70
C192-76*	T126-76	D370-67	T83-70

ASTM Designation	AASHTO Designation	ASTM Designation	AASHTO Designation
D402-76*	T78-78	D2041-71 (1976)	T209-74 (1978)
D420-69 (1975)	T86-74	D2042-76*	T44-78
D555-75	T151-78	D2166-66 (1972)	T208-70
D806-74	T144-74	D2167-66 (1972)	T205-64 (1974)
D854-58 (1972)*	T100-75	D2170-76*	T201-76
D979-51 (1968)	T168-55 (1974)	D2171-66 (1972)*	T202-77
D1074-76*	T167-78	D2172-67*	T164-76
D1075-75*	T165-77	D2398-76*	T53-78
D1191-64 (1976)	T187-60 (1974)	D2434-68 (1974)	T215-70
D1194-72	T235-74	D2435-70	T216-74
D1195-64 (1971)	T221-66 (1974)	D2489-67 (1974)	T195-67 (1978)
D1298-67 (1977)*	T227-68	D2664-67 (1974)	T226-68
D1452-65 (1972)	T203-64 (1974)	D2844-69 (1975)	T190-78
D1461-60 (1973)	T110-70	D2872-74*	T240-78
D1556-64 (1974)	T191-61 (1974)	D2922-71 (1976)	T238-76
D1559-76*	T245-78	D3017-72	T239-76
D1560-71	T246-74	E4-72	T67-74
D1561-71	T247-74	E8-69	T68-74
D1587-74*	T207-75	E10-73	T70-77
D1664-69 (1975)	T182-70	E18-74	T80-76
D1665-61 (1973)	T54-61 (1974)	E70-68	T200-70
D1754-76*	T179-76	E274-77*	T242-78
D1856-69*	T170-73		

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Standard Method of Test for

Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)

AASHTO DESIGNATION: T 238-76
(ASTM DESIGNATION: D 2922-71 (1976))

INTRODUCTION

These methods describe determination of the density of soil and soil-aggregate in place through the use of nuclear equipment. In general, the total or wet density of the material under test is determined by placing a gamma source and a gamma detector either on, into, or adjacent to the material under test. These variations in test geometry are presented as the backscatter, direct transmission, or air gap approaches. The intensity of radiation detected is dependent in part upon the density of the material under test. The radiation intensity reading is converted to measured wet density by a suitable calibration curve. Principles of the nuclear test are discussed in the Appendix, as are some of its advantages and disadvantages. It should be noted that the density determined by these methods are not necesarily the average density within the volume involved in the measurement and that the equipment utilized radioactive materials which may be hazardous to the health of users unless proper precautions are taken.

1. SCOPE

1.1 This method covers the determination of the total or wet density of soil and soil-aggregate in place by the attenuation of gamma rays where the gamma source of gamma detector, or both, remain at or near the surface. The methods described are normally suitable to a test depth of approximately 2 to 12 in. (50 to 300 mm), depending on the test geometry used.

1.2 Three methods are described as follows:

	Sections
Method A—Backscatter	5 to 7
Method B—Direct Transmission	8 to 10
Method C—Air Gap	11 to 14

2. SIGNIFICANCE

2.1 The methods described are useful as rapid, non destructive tehchniques for the inplace determination of wet density of soil and soil-aggregate. The fundamental assumptions inherent in the methods are that Compton scattering is the dominant interaction and that the material under test is homogeneous.

2.2 The methods are suitable for control and acceptance testing of soils and soil-aggregate for construction, as well as for use in research and development. Test results may be affected by chemical composition, sample heterogeneity, and the surface texture of the material being tested. The techniques also exhibit spatial bias in that the apparatus is more sensitive to certain regions of the material under test.

3. CALIBRATION

3.1 Calibration curves are established by determining the nuclear count rate of each of several materials at different and known densities, plotting the count rate (or count ratio) versus each known density, and placing a curve through the resulting points. The nuclear count rate should be determined by averaging a minimum of four measurement-counting periods of at least 1 min. each. The method used in establishing the curve must be the same as will be used in determining the density. The density of materials used to establish the calibration curve must vary over a range that includes the density of materials that will be tested. The materials used for calibration must be of uniform density.

3.1.1 Calibration curves can be established using the following:

3.1.1.1 Blocks of known density. Materials considered satisfactory for use in blocks include granite, aluminum, magnesium/aluminum laminate, limestone, and magnesium. For direct transmission tests a hole may be drilled in the blocks to the dimensional tolerances given in 8.1.4.

NOTE 1: Due to the effect of differing chemical compositions, calibration curves may not be applicable to materials not represented in establishing the calibration curve. Metallic blocks are satisfactory only for determining the proper shape and slope of the calibration curve. The correct location of the curve must be determined by tests upon blocks or materials of composition similar to that encountered in field testing.

3.1.1.2 Prepared containers of soil-aggregate compacted to known densities.

NOTE 2: Use of blocks is advantageous because they are durable and provide stable density references. Blocks and prepared containers must be large enough to not change the observed count rate (or count ratio) if made larger in any dimension. Minimum dimensions of blocks shall be 12 inches wide by 14 inches deep by 22 inches long, (305 by 356 by 559 mm). Prepared containers must be large enough to allow rotation of the gage 90°. Minimum dimensions for containers using soil specimens shall be 22 inches wide by 22 inches long by 14 inches deep (559 by 559 by 356 mm). For calibration of backscatter only a depth of not less than 6 in. (150 mm) is adequate.

3.2 *Checking Calibration Curves*—The calibration curves for newly acquired instruments should be checked. Calibration curves should also be checked if for any reason routine test results are believed to be inaccurate. For the backscatter method, calibration curves should also be checked for tests in materials which are distinctly different from material types previously tested, and which may have different chemical compositions.

3.2.1 Calibration curves may be checked either on blocks (3.1.1.1) or prepared containers (3.1.1.2).

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3.3 *Adjusting Calibration Curve*—When blocks or prepared containers of materials of known density are being used to check the calibration, plot the count rate (or count ratio) versus each known density described in 3.1. If the points do not fall on the previously established calibration curve, replace the original calibration curve with a parallel curve through the plotted check points.

3.3.1 When the sand-cone, rubber-balloon or specific gravity method is being used to check the calibration, compare the average of at least five locations with one nuclear test and one sand-cone, rubber-balloon or specific gravity test at exactly the same location in each area, and proceed as follows to adjust the calibration curves.

3.3.1.1 If the density of each of the comparison tests determined by the sand-cone, rubber-balloon or specific gravity method varies by less than 5 lb/ft^3 (80 kg/m^3) from the density determined by the nuclear method and if the average of all sand-cone, rubber-balloon or specific gravity density tests is within 2 lb/ft^3 (32 kg/m^3) of the average of all corresponding nuclear tests, no adjustment of the calibration curve is necessary.

3.3.1.2 If the average of all density determinations by the sand-cone, rubber-balloon or specific gravity method is more than 2 lb/ft^3 (32 kg/m^3) above or below the average of all corresponding nuclear tests, subsequent nuclear tests shall be adjusted by the amount of the difference in averages. This difference shall be added to nuclear determinations if the nuclear average is lower, subtracted if the nuclear average is higher.

3.3.1.3 The average difference obtained by 3.3.1.2 may be used to plot a corrected calibration curve which shall be parallel to the original calibration curve and offset by the amount and direction as determined in 3.3.1.2.

NOTE 3: Adjusting calibration curves is a complex task and it should be attempted only be those knowledgable in this field.

4. PRECISION

4.1 Any equipment that is used under the requirements of this method shall satisfy these requirements for system precision.

4.2 Precision of the system is determined from the slope of the calibration curve and the statistical deviation of the signals (detected gamma rays) in counts per minute (cpm).

$$P = /s \quad \sigma \qquad (1)$$

where:

P = precision
σ = standard deviation, cpm
s = slope, cpm/lb/ft^3 (kg/m^3)

4.3 Determine the slope of the callibration curve at the 110 lb/ft^3 (1762 kg/m^3) point in counts per minute per pound per cubic foot (counts per minute per kilograms per cubic meter). Determine the standard deviation of ten repetitive readings of 1 min each (gage not moved after seating for first count) taken on material having a density of 110.0 ±5.0 lb/ft^3 (1762 ±80 kg/m^3). The value of P shall be less than 1.25 lb/ft^3 (20 kg/m^3).

METHOD A—BACKSCATTER

5. APPARATUS

5.1 *Gamma Source*, shall be an encapsulated and sealed radioisotopic source.
5.2 *Gamma Detector*, may be any suitable type.
5.3 *Readout Device*, shall be a suitable scaler. Usually the readout device will contain the high-voltage supply necessary to operate the detector, and a low-voltage power supply to operate the readout and accessory devices.
5.4 *Housing*—The source, detector, readout device, and power supply shall be in housings of rugged construction that are moisture and dust proof.

NOTE 4—The gamma source, detector, readout device, and power supply may be housed separately or combined and integrated with a nuclear moisture measuring system.

5.5 *Reference Standard*, of uniform, unchanging density shall be provided with each gage for the purpose of checking equipment operation and background count, and to establish conditions for determining count rate reproducibility.
5.6 *Site Preparation Device*—A steel plate, straightedge, or other suitable leveling tools may be used to plane the test site to the required smoothness.

6. STANDARDIZATION

6.1 Standardization of equipment on a reference standard is required at the start of each day's use and when test measurements are suspect.
6.2 Warm up the equipment in accordance with the manufacturere's recommendations.
6.3 Take at least four repetitive readings of at least 1 min each with the gage on the reference standard. The constitutes one standardization check.
6.4 If the mean of the four repetitive readings is outside the limits set by Eq2, repeat the standardization check. If the second standardization check satisfies Eq 2, the equipment is considered to be in satisfactory operating condition. If the second standardization check does not satisy Eq 2, the calibration should be checked (3.2). If the calibration check shows that there is no significant change in the calibration curve, a new reference standard count, N_O, should be established. If the calibration checks shows that there is a significant difference in the calibration curve, repair and recalibrate the instrument.

$$N_S = N_O \pm 1.96 \sqrt{N_O} \qquad (2)$$

where:

N_S = count currently measured in checking the instrument operation on the reference standard (6.3) and N_O = count previously established on the reference standard (mean of ten repetitive readings).

7. PROCEDURE

7.1 Select a test location where the gage in test position will be at least 6 in. (150 mm) away from any vertical projection.

7.2 Prepare the test site in the following manner:

7.2.1 Remove all loose and distrubed material and additional material as necessary to expose the top of the material to be tested.

Note 5—The spatial bias should be considered in determining the depth at which the gage is to be seated.

7.2.2 Prepare a horizontal area sufficient in size to accommodate the gage, by planing the area to a smooth condition so as to obtain maximum contact between the gage and material being tested.

7.2.3 The maximum void beneath the gage shall not exceed 1/8 in. (3 mm). Use native fines or fine sand to fill these voids and smooth the surface with a rigid plate or other suitable tool.

Note 6—The placement of the gage on the surface of the material to be tested is critical to the successful determination of density. The optimum condition is total contact between the bottom surface of the gage and the surface of the material being tested. This is not possible in all cases and to correct surface irregularities use of sand or similar material as a filler is necessary. The depth of the filler should not exceed 1/8 in. (3 mm) and the total area filled should not exceed 10 percent of the bottom area of the gage. Several trial seatings may be required to achieve these conditions.

7.3 Proceed with the test in the following manner:

7.3.1 Seat the gage firmly.

7.3.2 Keep all other radioactive sources away from the gage to avoid affecting the measurement.

7.3.3 Warm up the equipment in accordance with the manufacturer's recommendation.

7.3.4 Secure and record one or more 1 min readings.

7.3.5 Determine the in-place wet density by use of the calibration curve previously established.

METHOD B—DIRECT TRANSMISSION

8. APPARATUS

8.1 The direct transmission system shall consist of the following units. The exact details of construction of the apparatus may vary; however, the general requirements of 5.1 through 5.6 shall apply in addition to the following:

8.1.1 *Probe*—Either the gamma source or the detector shall be housed in a probe for inserting in a preformed hole in the material to be tested. The probe shall be marked in increments of 2 in. (or 50 mm) for tests with probe depths from 2 to 12 in. (or 50 to 300 mm). The probe shall be so made mechanically that when moved manually to the marked depth desired it will be held securely in position at that depth.

8.1.2 *Housing*—The source, detector, readout device, probe, and power supply shall be in housings of rugged construction that are moisture and dust proof.

8.1.3 *Guide*, for forming a hole normal to the prepared surface.

8.1.4 *Hole -Forming Device*, such as an auger or pin, having a nominal diameter equal to or slightly larger than the probe but not to exceed the diameter of the probe by more than 1/8 in. (3mm) for forming a hole in the material to be tested to accommodate the probe.

9. STANDARDIZATION

9.1 Standardization of equipment on a reference standard is required at the start of each day's use and when test measurements are suspect.

9.2 Warm up the equipment in accordance with the manufacturer's recommendations.

9.3 With the gage on the reference standard provided by the manufacturer and with the probe in the position prescribed by the manufacturer for measuring standard count, take at least four repetitive readings of at least 1 min each. This constitutes one standardization check.

9.4 If the mean of the four repetitive readings is outside the limits set by Eq 2, repeat the standardization check. If the second set standardization check satisfies Eq 2, the equipment is considered to be in satisfacotory operating condition. If the second standardization check does not satisfy Eq 2, the calibration should be checked (3.2). If the calibration check shows that there is no significant change in the calibration curve, a new reference standard count, N_0, should be established. If the calibration check shows that there is a significant difference in the calibration curve, the instrument should be repaired and recalibrated.

10. PROCEDURE

10.1 Select a test location where the gage in test position will be at least 6 in. (150 mm) away from any vertical projection.
10.2 Prepare the test site in the following manner:
10.2.1 Remove all loose and disturbed material, and remove additional material as necessary to expose the top of the material to be tested.
10.2.2 Prepare a horizontal area, sufficient in size to accommodate the gage, by planing the area to a smooth condition so as to obtain maximum contact between the gage and material being tested.
10.2.3 The maximum void beneath the gage shall not exceed approximately 1/8 in. (3 mm). Use native fines or fine sand to fill these voids and smooth the surface with a rigid plate or other suitable tool. The depth of the filler should not exceed approximately 1/8 in. (3 mm).
10.2.4 Make a hole perpendicular to the prepared surface using the guide (8.1.3) and the hole-forming device (8.1.4). The hole shall be of such depth and alignment that insertion of the probe will not cause the gage to tilt from the plane of the prepared area.
10.3 Proceed with testing in the following manner:
10.3.1 Tilt the gage and extend the probe in the position required for the desired depth of test.
10.3.2 Insert the probe in the hole.
10.3.3 Seat the gage firmly by rotating it about the probe with a back and forth motion.
10.3.4 Pull gently on the gage in the direction that will bring the side of the probe to face the center of the gage so that the probe is in intimate contact with the side of the hole.
10.3.5 Keep all other radioactive sources away from the gage to avoid affecting the measurement.
10.3.6 Warm up the equipment in accordance with the manufacturer's recommendation.
10.3.7 Secure and record one or more 1 min readings.
10.3.8 Determine the in-place wet density by use of the calibration curve previously established.

METHOD C—AIR GAP

11. APPARATUS

11.1 All apparatus described in Sections 5 and 8.
11.2 *Cradle or Spacers*, to support the gage at the optimum air gap above the material being tested. The cradles or spacers shall be so designed as to support the gage at optimum height without shielding the base of the gage. Figure 1 shows a typical air-gap cradle that demonstrates the principle. The cradle shown in Fig. 1 is not the only satisfactory method. Other methods which support the gage at the optimum air gap without shielding the base of the gage are satisfactory.

Note 7—Air-gap calibration curves and optimum air gap may be furnished for each gage by the manufacturer and can be readily checked by the user.

12. DETERMINATION OF OPTIMUM AIR GAP

12.1 To determine the optimum air gap for use in the air-gap method, proceed as follows:
12.1.1 Use three or more different areas on which to make determinations. These areas may be either blocks (3.1.1.1) or prepared containers (3.1.1.2) used for calibration or field areas of compacted soil or soil-aggregate (3.1.1.3). The density of materials at the selected areas should vary through a range including the densities of the materials which will be tested.
12.1.2 Place the density gage over the test area. Support the gage by blocks placed at the extreme edges of the gage so as not to obstruct the space between the bottom of the gage and the surface of the test area.
12.1.3 Take and record two 1 min readings in counts per minute and determine the average of the readings.
12.1.4 By adding additional blocks or spacers, increase the air gap by 1/4 in. (6.4 mm). Take and record, and average two additional 1-min readings.
12.1.5 Continue increasing the air gap by increments of 1/4 in. (6.4 mm), securing average readings for each air gap (12.1.3) until there is a decrease in the counts per minute readings with an increase of air gap.

12.1.6 On an arithmetic scale plot counts per minute as the ordinate *versus* each air gap (in inches or millimeters) and draw a smooth curve through the resulting points. Record the peak air gap determined at the peak of the curve.

12.1.7 Repeat procedures 12.1.2 through 12.1.6 over two or more additional areas of materials of different density, and record the peak air gap for each area.

12.1.8 Determine the average of the peak air gaps determined on all areas. This is the optimum air gap. Use this optimum air gap for establishing the calibration curve for the air-gap method, and for all determinations of density by the air-gap method.

13. STANDARDIZATION

13.1 Standardization of equipment on a reference standard is required at the start of each day's use and when test measurements are suspect.

13.2 Warm up the equipment in accordance with the manufacturer's recommendations.

13.3 Take at least four repetitive readings of at least 1 min each with the gage on the reference standard. This constitutes one standardization check.

NOTE 8—The standard count determined in 13.3 is not used in the determination of density by the air-gap method. The purpose of checking the standard count as required by 13.3 is to check that the equipment is in satisfactory operating condition.

13.4 If the mean of the four repetitious readings is outside the limits set by Eq 2, repeat the standardization check. If the second standardization check satisfies Eq 2, the equipment is considered to be in satisfactory operating condition. If the second standardization check does not satisfy Eq 2, the calibration should be checked (3.2). If the calibration checks show that there is no significant change in the calibration curve, a new reference standard count, N_O, should be established. If the calibration check shows that there is a significant difference in the calibration curve, the instrument should be repaired and recalibrated.

14. PROCEDURE

14.1 Select a test location where the gage in test position will be at least 6 in. (150 mm) away from any vertical projection. Plane sufficient area to accommodate the gage and cradle.

14.2 Prepare the test site in the following manner:

14.2.1 Remove all loose and disturbed material, and additional material as necessary to expose the top of the material to be tested. (See Note 6.)

14.2.2 Prepare a horizontal area, sufficient in size to accommodate the gage and cradle, by planing the area to a smooth condition so as to obtain maximum contact between the gage and the material being tested.

14.2.3 The maximum void beneath the gage shall not exceed approximately 1/8 in. (3 mm). Use native fines or fine sand to fill these voids and smooth the surface with a rigid plate or other suitable tool.

NOTE 9—The air-gap method requires taking one or more readings in both the backscatter position and the air-gap position. The placement of the gage on the surface of the material to be tested is critical to the successful determination of density. The optimum condition is total contact between the bottom surface of the gage and the surface of the material being tested. This is not possible in all cases and to correct surface irregularities use of sand or similar material as filler is necessary. The depth of the filler should not exceed 1/8 in. (3 mm), and the total area filled should not exceed 10 percent of the bottom area of the gage. Several trial seatings may be required to achieve these conditions.

14.3 Proceed with the test in the following manner:

14.3.1 Seat the gage firmly.

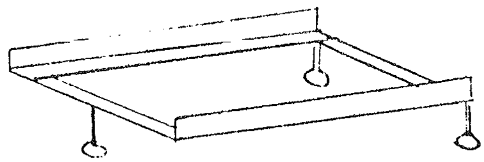
14.3.2 Keep all other radioactive sources away from the gage to avoid affecting the measurement so as not to affect the readings.

14.3.3 Warm up the equipment in accordance with the manufacturer's recommendation.

14.3.4 Secure and record one or more 1 min readings in the backscatter position.

14.3.5 Place the cradle, set at optimum air gap, on the test site, and place the gage on the cradle so that the gage is directly over the same site used for backscatter reading. When a direct-transmission type gage is used, set the probe in the retracted or backscatter position for this reading.

14.3.6 Take the same number of 1 min readings in the air-gap position as in the backscatter position (14.3.4).



Welded metal approx. 1 by 1 by 1/8 in.
(25 by 25 by 3 mm) angle.

Fig. 1. Typical Air-Gap Cradle

14.3.7 Determine the air-gap ratio by dividing counts per minute obtained in the air-gap position (14.3.6) by counts per minute obtained in backscatter position (14.3.4).

14.3.8 Determine the in-place wet density by use of the applicable calibration curve previously established.

NOTE 10—The air-gap ratio may be determined by dividing counts per minutes obtained in backscatter position by counts per minute obtained in the air-gap position or *vice versa*. Whichever ratio is used, a calibration curve using the same ratio must also be used.

APPENDIX

A1. NOTES ON THE NUCLEAR TEST

A1.1 The equipment used in this method is of the surface type as opposed to that designed for use in deep borings. In general, and neglecting the associated electronics, this equipment consists of three principal elements: (*1*) a nuclear source emitting gamma rays, (*2*) a detector sensitive to these rays as they are modified by passing through the material being tested and (*3*) a counter or scaler with provisions for automatic and precise timing, for determining the rate at which the modified gamma rays arrive at the detector. While rate meters are suitable, in principle, scalers are commonly used. In general any source of gamma rays that are sufficiently numerous and properly energetic can be used in measuring the denstiy of soil and soil-aggregate. Source stability with time, in terms of half-life is an important design consideration and the sources most commonly used are celsium-137 and radium-beryllium with the latter normally being used in dual-purpose instruments that are designed to also determine moisture content. The two detectors most commonly used are gas-filled tubes of the Geiger-Müller type and scintillation crystals, usually of sodium-iodide. Detectors of the latter type offer the potential of electronically varying the range of energies of the gamma rays that are counted. With detectors of the Geiger-Müller type this range is fixed in the design. For most available equipment the source-detector geometry is fixed for backscatter gages and is adjustable to various preselected depths of direct transmission gages.

A1.2 Measurements are made using gamma rays that largely reflect at reduced energy by scattering in, or by, direct transmission through the material under test. In backscatter the rays are emitted into the material from near its surface and some are deflected at reduced energy back to the detector, largely by Compton scattering. In direct transmission the source or detector is inserted in the test materials and, in contrast to the backscatter method, some of the emitted and unshielded rays can presumably follow a straight-line path to the detector. In either source-detector arrangement the number of rays reaching the detector is, over-all, a nonlinear function of the density of the material being tested. For the usual range of soil and soil-aggregate densities the relationship is such that the higher the density of a given material, the lower the count rate.

A1.3 The determination of density by the nuclear means of this method is indirect. To date no theoretical approach has been developed that predicts the count rate for given equipment, geometry, material, and density. As a result the relationship between material density and nuclear-count rate is determined by correlation tests of materials at known average densities. Individual equipment manufacturers supply a calibration curve with each set of their equipment. It has been found that these curves do not necessarily hold for all soils and soil-aggregates because of differences in chemical composition. Apparent variations in calibration curves may also be induced by differences in seating, in background count, and other test variations. Because of these considerations, provisions are included in this method for checking for variations or changes. Different approaches may be used in checking calibration curves and those in more general use are given. For good practice these calibration procedures should be followed with newly purchased equipment and with major component replacements of in-service equipment.

A1.4 The density determined by this method is the wet or total density. It should be noted that the volume of soil or soil-aggregate represented in the measurements is indeterminate and will vary with the source-detector geometry of the equipment used and with the characteristics of the material tested. In general, and with all other conditions constant, the more dense the material, the smaller the volume involved in the measurement. The density so determined is not necessarily the average density within the volume involved in the measurement. Although for the usual surface backscatter test equipment and materials the gages are influenced by 3 to 5 in. (75 to 125 mm) of material, the top 1 in. (25.4 mm) of the material determines about one half of the measured count rate with the result that the observed density is largely determined by the density of the upper layers. For the usual density conditions the total count is largely determined by the upper 3 to 4 in. (75 to 100 mm) of soils and soil-aggregates. Where these materials are of uniform density, this characteristic of this method is of no effect. With the air-gap method the penetration of the backscattered rays is relatively shallow. With direct-transmission gages the effect of vertical density variations may be eliminated. Other problems, however, can be introduced in the mechanics of inserting the source or the detector.

A1.5 The number of gamma rays emitted from a given source over a given time period are statistically random and follow a Gaussian distribution. Because of this the actual number of modified rays that are detected and counted in the density-measuring process should be sufficiently large to minimize the probability that the observed count reflects unacceptable variations. This is reflected in the standard deviation which is the square root of the total count. The over-all system accuracy in detrmining densities is also statistical in nature and appears to vary with the equipment used, the test conditions of laboratory *versus* field, as well as with materials and operators. Because of these variables it is not possible to give precise numbers for system accuracy and precision for these methods. It is believed, however, that if the procedures herein are carefully followed, the standard deviation of the nuclear measured values, in terms of accuracy, will not be greater than on the order of some 3 to 5 lb/ft^3 (48 to 80 kg/m^3) while in terms of precision or repeatability, determined without moving the test equipment, this should not be greater than on the order of 1 lb/ft^3 (16 kg/m^3).

A1.6 One of the most commonly used sources, cesium-137, is man made and as such its use is regulated by the Federal Government through the Atomic Energy Commission as well as by some state and local governments. Because radium is a naturally occurring material, its use is not now regulated by the Federal Government but is by some state and local governments. Among others, the objectives of these regulations are the use of radioactive materials in a manner safe to the operator and all others. While detailed safety procedures are beyond the scope of this method, the originating committee emphasizes its support of these objectives. It strongly recommends that users of this equipment become completely familiar with possible safety hazards and that they establish effective operator instruction and use procedures together with routine safety procedures such as routine source-leak tests, the routine recording and evaluation of film badge data, the use of survey meters, etc. in connection with the operation of equipment of this type.

A1.7 The in-place nuclear density tests of this method offer several advantages over the older conventional methods (sand-cone, rubber-balloon, etc.), particularly in tests performed for the continuing control of construction. Among these, perhaps the principal advantage is the relative ease with which the test can be performed, thus freeing the operator from the physical tasks of digging holes and collecting and weighing bulky samples. However, it sacrifices the opportunity to examine the soil in depth. If information is sought on in-place densities only, and test determinations of maximum density are not involved, many more tests can be performed per day than by the older methods. In addition, apparently erratic measurements can be immediately detected and checked since the nuclear tests are more nearly nondestructive. These advantages accrue to organizations that are engaged in density measurements on a more or less continuous basis. Organizations that make infrequent or occasional density determinations may find that the advantages of the nuclear method can be offset by maintenance and start-up considerations such as periodically charging batteries, maintaining radiation exposure records, etc.

Standard Method of Test for

Moisture Content of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)

AASHTO DESIGNATION: T 239–76
(ASTM DESIGNATION: D 3017–72)

INTRODUCTION

This method covers determination of the moisture content of soil and soil-aggregate in place through the use of nuclear equipment. The equipment is calibrated to determine moisture content, as mass of water per unit volume of material (pounds per cubic foot or kilograms per cubic meter). Moisture content as normally used is defined as the ratio, expressed as a percentage, of the mass of water in a given soil mass to the mass of solid particles. It is determined with this procedure by dividing the moisture content (pounds per cubic foot or kilograms per cubic meter) by the dry density of soil (pounds per cubic foot or kilograms per cubic meter). Therefore, computation of moisture content using the nuclear equipment also requires the determination of the dry density of the material under test. Most available nuclear equipment has provision for measuring both moisture content (pounds per cubic foot or kilograms per cubic meter) and wet density. The difference between these two measurements gives dry density.

The moisture content determined by this method is not necessarily the average moisture within the volume of sample involved in the measurement for reasons discussed in the Appendix. The principles of this method as well as the advantages and limitations are also discussed in the Appendix.

The equipment utilizes radioactive materials which may be hazardous to the health of the users unless proper precautions are taken.

1. SCOPE

1.1 This method covers determination of the moisture content of soil and soil-aggregate in place by moderation or slowing of fast neutrons where the neutron source and the thermal neutron detector both remain at the surface.

2. APPLICABLE DOCUMENTS

2.1 AASHTO Test Methods:

T 191 Density of Soil in Place by the Sand-Cone Method
T 205 Density of Soil in Place by the Rubber-Balloon Method

2.2 ASTM Test Methods:

D 2216 Laboratory Determination of Moisture Content of Soil

3. SUMMARY OF METHOD

3.1 The moisture content of the material under test is determined by placing a fast neutron source and a thermal neutron detector on or adjacent to the material under test. The intensity of slow or moderated neutrons detected is dependent upon the moisture content of the material under test. Moisture is determined by the relationship of nuclear count to mass of water per unit volume of soil.

4. SIGNIFICANCE

4.1 The method described is useful as a rapid, nondestructive technique for the inplace determination of moisture content of soil and soil-aggregate. The fundamental assumptions inherent in the method are that the hydrogen present is in the form of water as defined by ASTM D 2216, and that the material under test is homogeneous.

4.2 The method is suitable for control and acceptance testing of soils and soil-aggregate for construction, research, and development. Test results may be affected by chemical composition, sample heterogeneity, and, to a lesser degree, material density and the surface texture of the material being tested. The technique also exhibits spatial bias in that the apparatus is more sensitive to certain regions of the material under test.

5. APPARATUS

5.1 *Fast Neutron Source*—A sealed isotope material such as americium-beryllium, radium-beryllium, or an electronic device such as a neutron generator.

5.2 *Slow Neutron Detector*—Any type of slow neutron detector such as boron trifluoride, a scintillator crystal, or a fission chamber.

5.3 *Readout Device*—A suitable scaler. Usually the readout device will contain the high-voltage supply necessary to operate the detector, and low-voltage power supply to operate the readout and accessory devices.

5.4 *Housing*—The source, detector, readout device, and power supply shall be in housings of rugged construction which shall be moistureproof and dustproof.

NOTE 1—The neutron source, detector, readout device, and power supply may be housed separately or may be combined and integrated with a nuclear density measuring system.

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5.5 *Reference Standard*, for checking equipment operation and background count, and to establish conditions for a reproducible count rate.

5.6 *Site Preparation Device*—A steel plate, straightedge, or other suitable leveling tools may be used to plane the test site to the required smoothness.

6. CALIBRATION:

6.1 Calibration curves are established by determining by test the nuclear count rate of each of several samples of different known moisture content, plotting the count rate (count ratio) versus each known moisture content and placing a curve through the resulting points. The method and test procedure used in establishing the curve must be the same as used for determining moisture content of the material to be tested. The moisture content of materials used to establish the calibration curve must vary through a range to include the moisture content of materials to be tested. The materials used for calibration must be of uniform density as well as uniform moisture content.

NOTE 2: Due to the effect of chemical composition, some calibration curves supplied with the apparatus may not be applicable to all materials under test. Therefore, calibration curves must be checked, and adjusted if necessary, in accordance with 6.2 and 6.3.

6.1.1 Calibration curves can be established using the following methods:

6.1.1.1 Prepared containers of compacted soil and soil-aggregate of known moisture content. Prepared containers must be large enough to not change the observed count rate (or count ratio) if made larger in any dimension (Note 3). If the hydrogen density of a material can be calculated from its specific gravity and chemical formula, provided these are accurately known, a much more reliable calibration point can be obtained in comparison to oven drying methods. The absorbed water in the stone, which may be removed by drying at 110°C for twenty-four hours, must also be considered in comparison to oven drying methods.

6.1.1.2 Permanent calibration blocks or standards containing chemically bound hydrogen which will produce nuclear gauge responses equivalent to known moisture content (Note 3). A non-hydrogenous material, such as magnesium, may be used for zero water content.

NOTE 3: Dimensions of approximately 24 inches long by 24 inches wide by 15 inches deep (approximately 610 by 610 by 381 mm) have proven satisfactory for equipment now available. (1975)

6.2 *Checking Calibration Curves*—The calibration curves for newly acquired instruments and repaired instruments should be checked. Calibration curves should be checked prior to tests in materials that are distinctly different from material types previously used in obtaining the calibration curves. Calibration curves may be checked using the procedures given in 6.1.1.

6.3 *Adjusting Calibration Curves*—When permanent standards or blocks are used to check moisture calibration, plot the count rate (or count ratio) versus each known moisture content as described in 6.1. If a curve drawn through these plotted checkpoints indicates moisture contents with 1.0 lb/ft³ (16 kg/m³) of those shown by the regular calibration curve for identical count rates (or ratios), no adjustment of the calibration curve is necessary. If the difference exceeds this amount, the curve established by the checkpoints will replace the original calibration curves.

6.3.1 If the plot of each of the comparison moisture test results, determined by calibration method 6.1.1.1, form a scatter pattern which randomly straddles the previously established calibration curve and the average of all computed or oven-dried comparison tests is with 1.0 lb/ft³ (16 kg/m³) of the average of all corresponding nuclear moisture determinations, adjustments of the calibration curve is not necessary.

6.3.2 If the average difference obtained in 6.3.1 is more than 1.0 lb/ft³ (16 kg/m³), subsequent nuclear moisture tests shall be adjusted by either adding the difference in averages if the nuclear average is lower or subtracting the difference in averages if the nuclear average is higher.

6.3.3 The average difference obtained by 6.3.2 may be used to plot a corrected calibration curve which shall be parallel to the original calibration curve and offset by the amount and direction as indicated in 6.3.2. A corrected calibration curve may also be determined by plotting the count rate (or ratio) versus each known moisture content as described in 6.1, and drawing a curve through these plotted checkpoints.

7. PROCEDURE

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7.1 *Standardization*—Standardization of equipment on a reference standard is required at the start of each day's use as follows:

7.1.1 Warm up the equipment in accordance with the manufacturer's recommendations.

7.1.2 Take at least four repetitive readings of at least 1 min each with the gage on the reference standard. This constitutes one standardization check.

7.1.3 If the mean of the four repetitive readings is outside the limits set by Eq 1 below, repeat the standardization check. If either the first or the second attempt satisfies Eq 1, the equipment is considered to be in satisfactory operating condition; continue with the procedure in 7.2. The empirical relationship for the standardization check is as follows:

$$N_s = N_O \pm 1.96\sqrt{N_O} \qquad (1)$$

where:

N_s = mean of repetitive readings (see 7.1.2) and
N_O = previously established count for the reference standard (mean of ten repetitive readings).

7.1.4 If the second attempt in 7.1.3 does not satisfy Eq 1, check the system and repair the instrument if necessary. It is possible to use the instrument in this condition if a satisfactory calibration relationship can be established.

7.1.5 Establish a new N_O by computing the mean of ten repretitive readings on the reference standard.

7.1.6 Check the calibration curve in accordance with 6.2 and, if necessary, adjust the calibration curve in accordance with 6.3.

7.2 *Test Site Preparation*—Select a location for test where the gage in test position will be at least 6 in. (approximately 150 mm) away from any vertical projection.

7.2.1 Prepare the test site in the following manner:

7.2.1.1 Remove all loose and disturbed material, and remove additional material as necessary to reach the top of the vertical interval to be tested (Note 4).

7.2.1.2 Prepare a horizontal area, sufficient in size to accommodate the gage, by planing to a smooth condition so as to obtain maximum contact between the gage and material being tested.

7.2.1.3 The maximum depressions beneath the gage shall not exceed 1/8 in. (3 mm). Use native fines or fine sand to fill voids and level the excess with a rigid plate or other suitable tool.

NOTE 4—The spatial bias should be considered in determining the depth at which the gage is to be seated.

NOTE 5—The placement of the gage on the surface of the material to be tested is critical to the successful determination of moisture. The optimum condition is total contact between the bottom surface of the gage and the surface of the material being tested. This is not practically possible in all cases and therefore it becomes necessary to use a sand filler or similar material. The depth of sand fill should not exceed 1/8 in. (approximately 3 mm) and the total area filled should not exceed 10 percent of the bottom area of the gage. Several trial seatings may be required to achieve these conditions.

7.3 *Testing*—Proceed with testing in the following manner:

7.3.1 Seat the gage firmly.

7.3.2 Keep all other radioactive sources away from the gage (as recommended by the manufacturer) so as not to affect the readings.

7.3.3 Use sufficient warm-up time as in standardization (7.1.1).

7.3.4 Place the source in the use position and take one or more 1-min readings.

8. CALCULATIONS

8.1 Average the readings obtained in 7.3.4.

8.2 Determine the moisture content by use of an applicable calibration curve.

8.3 Calculate the moisture content, *w*, in mass percent of the dry soil as follows:

$$w = (W_m/W_d) \times 100 \quad (2)$$

where:

W_m = moisture content of soil, lb/ft³ (or kg/m³) and
W_d = dry density of soil, lb/ft³ (or kg/m³).

9. REPORT

9.1 The report shall include the following:
9.1.1 Location,
9.1.2 Elevation of surface,
9.1.3 Visual description of material,
9.1.4 Identification of test equipment (make, model, and serial number),
9.1.5 Count rate for each reading,
9.1.6 Moisture content in pounds per cubic foot (kilograms per cubic meter),
9.1.7 Wet density,
9.1.8 Dry density, and
9.1.9 Moisture content mass percent of dry soil.

10. PRECISION

10.1 Determine the precision of the system, *P*, from the slope of the calibration curve, *S*, and the standard deviation, σ, of the signals (detected neutrons) in counts per minute, as follows:

$$P = \sigma / S \quad (3)$$

10.2 Where the slope of the calibration curve is determined at the 10-lb/ft³ (160-kg/m³) point and the standard deviation is determined from ten repetitive readings of 1-min each (the gage is not moved after the first seating) taken on material having a moisture content of 10.0 ± 0.6 lb/ft³ (160 ± 10 kg/m³) the value of *P* shall be less than 0.30 lb/ft³ (4.8 kg/m³).

APPENDIX

A1. NOTES ON THE NUCLEAR TEST

A1.1 The equipment used in this method is of the surface type as opposed to that designed for use in deep borings. In general, and neglecting the associated electronics, this equipment consists of three principal elements: (*1*) a suitable nuclear source emitting fast neutrons; (*2*) a detector sensitive to these neutrons after they are modified by passing through the material being tested; and (3) a counter, with provisions for automatic and precise timing, for determining the rate at which the modified neutrons arrive at the detector. While rate meters are suitable, in principle, counters are commonly used. In general, any source of sufficiently numerous and properly energetic neutrons can be used in measuring the moisture content of soil and soil-aggregate. The sources most commonly used, however, are americium-beryllium and radium-beryllium with the latter normally being used in dual-purpose instruments that are designed to also determine wet or total density of soil and soil-aggregates. Detectors used are gas filled tubes of boron trifluoride and scintillation crystals or fission chambers. Detectors of the latter type offer the potential of varying the range of energies of the neutrons that are counted.

A1.2 Measurements are made using fast neutrons that reflect modification by back-scattering through the material under test. When high-energy neutrons are scatttered into the soil a loss in velocity of each neutron occurs as it collides with the nuclei of the atoms of the soil. The rate at which this slowing down process occurs depends upon: (*1*) the mass of the nucleus in collision with the neutron, and (*2*) the probability that the two will collide.

A1.2.1 The mass of the hydrogen nucleus is nearly equal to the neutron mass. Collison with hydrogen atoms therefore, reduces the velocity of neutrons more quickly than collison with heavier nuclei. The large difference between the masses of hydrogen atoms and those normally encountered in soils means that the relative effectiveness of hydrogen atoms in slowing down neutrons is very pronounced.

A1.2.2 The probability that a neutron will collide with the nucleus of an atom is dependent on the atom's scattering cross section. For most elements, this value is low, usually increasing with decrease in neutron energy. The scattering cross section of the hydrogen atom for high energy neutrons, however, is larger than for most of the other atoms present in soils.

A1.2.3 These two features in combination make hydrogen the most effective medium for reducing the velocity of fast neutrons. If a detector of thermal (slow) neutrons is placed near a neutron source in a soil containing hydrogen, the activity registered is due almost entirely to neutrons that have been slowed down by hydrogen atoms. Other atoms present in the soil play a negligible part in this process. In natural soils hydrogen may be present in several forms but, with some exceptions, it occurs principally in the water held by the soil particles. Therefore, the "slow neutron" activity registered by a suitable detector can be related to the concentration of water in a soil by calibration.

A1.3 The determination of moisture content by the nuclear means of this method is indirect. To date no theoretical approach has been developed that predicts the count rate for given equipment, geometry, material, density, and moisture content. As a result the relation between soil moisture and nuclear count rate is determined by correlation tests of materials at known moisture content. Individual equipment manufacturers supply a calibration curve with each set of their equipment. It has been found that these curves do not necessarily hold for all soils and soil-aggregates because of differences in chemical composition. Apparent variations in calibration curves may also be induced by differences in seating, in background count, and other test variations. Because of these considerations, provisions are included in this method for checking for variations or changes. Different approaches may be used in checking calibration curves and those in more general use are given. For good practice these should be followed with newly purchased equipment and with major component replacement of in-service equipment.

A1.4 The moisture content determined by this method is the amount of moisture that is contained in a given volume of soil. It should be noted that the volume of soil and soil-aggregate represented in the measurement is indeterminate and will vary with the source-detector geometry of the equipment used and with the characteristics of the material tested. In general, and with all other conditions constant, the greater the moisture content of the material, the smaller the volume involved in the measurement. Unlike oven drying tests, the moisture content so determined is not necessarily the average moisture within the volume involved in the measurement. For the usual surface test equipment and materials, for example, about 50 percent of the measured count rate is determined by the upper 3 to 4 in. (75 to 100 mm) of soils and soil-aggregate.

A1.5 The number of fast neutrons emitted from a given source over a given time period are statistically random and follow a Gaussian distribution. Because of this, the actual number of modified neutrons that are detected and counted in the moisture measuring process should be sufficiently large to minimize the probability that the observed count reflects unacceptable variations. This is reflected in the standard deviation which is the square root of the total count. The over-all system accuracy in determining moisture is also statistical in nature and appears to vary with equipment used, test conditions of laboratory *versus* field, materials, and operators. Because of these variables it is not possible to give precise numbers for system accuracy and precision for these test methods. It is believed, however, that if the procedures herein are carefully followed, the standard deviation of the nuclear measured values, in terms of accuracy, will not be greater than on the order of some 0.5 to 1.0 lb of water per cubic foot (8 to 16 kg/m^3) of soil. Precision, determined without moving the test equipment, should be better than 0.3 lb of water per cubic foot (5 kg/m^3) of soil.

A1.6 One of the fast neutron sources used, americium, is manmade and as such its use is regulated by the Federal government through the Atomic Energy Commission as well as by some State and local governments. Because radium and beryllium are naturally occurring materials, their use is not now regulated by the Federal Government but is regulated by some State and local governments. Among others, the objectives of these regulations are the use of radioactive materials in a manner safe to the operator and all others.

A1.7 The in-place nuclear moisture tests of this method offer several advantages over conventional methods, such as oven drying of samples, particularly in tests performed for the continuing control of construction. Its greatest advantage is perhaps the short time required to obtain a moisture content. An answer is available on the spot in a manner of minutes after completing the test. When conducting both moisture and density tests many more tests per day can be conducted than by older methods in current use. In addition, apparently erratic measurements can be immediately detected and checked since the nuclear tests are more nearly nondestructive. These advantages accrue to organizations that are engaged in moisture measurements on a more or less continuous basis. Organizations that make infrequent or occasional moisture determinations may find that the advantages of the nuclear methods can be offset by maintenance and start up considerations such as periodically charging batteries, maintaining radiation exposure records, etc.

DEPARTMENT OF HIGHWAYS, ONTARIO

THE CONSTANT DRY WEIGHT METHOD – A NO-WEIGHING FIELD COMPACTION TEST

by

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Presented at the 48th Annual Meeting of the Canadian Good Roads Association, Toronto.

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September, 1968

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THE CONSTANT DRY-WEIGHT METHOD – A NO-WEIGHING FIELD COMPACTION TEST

INTRODUCTION

Using conventional test methods it is not possible to make quick and reliable decisions in the field about the state of compaction of the earth grade or the base course.

Even when a field laboratory is available it takes hours to obtain the 'Maximum Dry Density' of a soil sample. The samples are obtained from boreholes, earth borrow pits, gravel pits, quarries, stockpiles or cuts where they may have been taken days or weeks before the material is placed in the fill. The compaction test results can be meaningful only if the material checked in the field and the material tested in the laboratory closely resemble each other. Inevitably, an element of doubt remains, since even in carefully selected borrow pits the material varies from place to place. For instance, in Report No. 2 of the AASHO Road Test (1) the following variations were reported:

	NUMBER OF TESTS TAKEN IN PIT	MAXIMUM DRY DENSITY		
		HIGHEST	LOWEST	STANDARD DEVIATION
		pcf	pcf	pcf
Borrow Pit No. 1	275	126	108	3.0
Borrow Pit No. 2	205	126	100	5.4
Borrow Pit No. 3	76	124	111	2.5

It is even more difficult under ordinary contract conditions to correlate field and laboratory samples, when the inspector has to choose from several samples taken from the different strata of a single beach, drumlin or esker formation. It is, therefore, not surprising to find wide discrepancies between the maximum dry density values used in the field and the maximum dry density of a test site sample determined subsequently in the laboratory. Many soils inspectors know the weakness of the conventional test method and are therefore reluctant to follow up test results. Furthermore, the maximum dry density of the field test location during a normal grading operation, where a scraper bucket may contain a mixture of several different soils, is simply not known, and to hold up grading operations whilst a laboratory maximum dry density test is made is out of the question.

Test delays conflict with the speed of present day earth moving equipment and for this reason attempts have been made to adapt the conventional test method to modern requirements. This is also one of the reasons why nuclear testing appears to be so attractive in spite of the new problems it poses.

The Constant Dry Weight (CDW) compaction test solves two fundamental problems inherent in conventional field test methods - it substantially reduces testing time to a degree where work progress is unimpaired, and it is self-sufficient, i.e., it is independent of a separate laboratory sample which inevitably differs from the inspected test-site soil.

The CDW test is a volumetric test. It measures compaction in terms of reduction in volume of the soil mass and, therefore, compares the volume of the soil sample in the field with the volume of the same soil sample after standard compaction in the Proctor mould *(see Figure 1).*

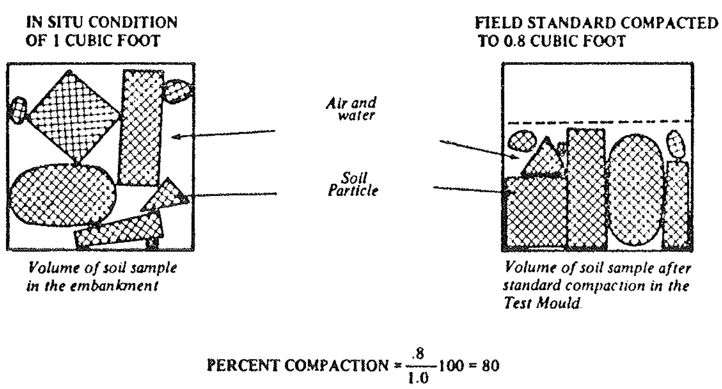

FIGURE 1, VOLUMETRIC COMPACTION TEST

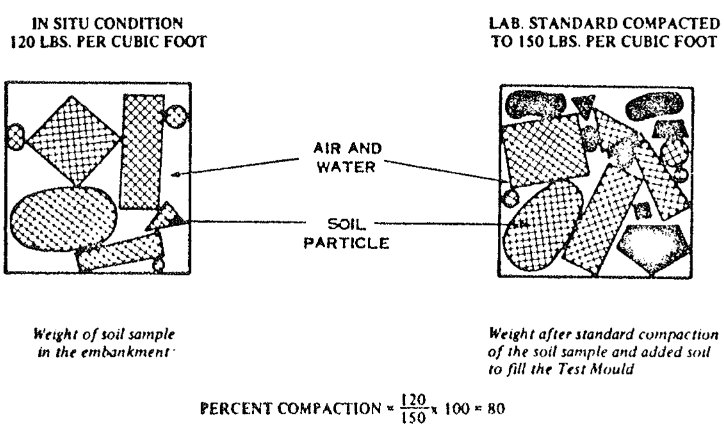

FIGURE 2, DENSITY COMPACTION TEST

By contrast, most other compaction tests measure the soil weight, as in the conventional tests where the weight of the soil sample in situ, contained in one cubic foot, is compared with the weight of all the soil that could be compacted into one cubic foot by standard compaction. In the illustrated case in Figure 2, more soil has to be added to the test sample in order to fill up the cubic foot volume, but adding soil complicates matters because of the different sizes and shapes of the added soil particles, and this in turn necessitates corrections to the test result (e.g., the stone correction). Further complications arise since it is not often known how representative of the test site the laboratory sample is.

A REVIEW OF FIELD COMPACTION TEST METHODS

The Ohio Typical Moisture-Density Curves Method (2) measures the wet density of the field sample but leaves out the time consuming moisture test. It uses instead a chart of Proctor curves based on thousands of compaction tests, combined with penetration resistance tests. The Inspector determines the wet density with the aid of the balloon or sand cone method (3), the penetration resistance by means of the Proctor needle and plots the maximum dry density on a chart. The use of the chart is limited to the geographic area where it was prepared. Test results then have to be corrected according to the sample's stone content.

The Hilf Method (4), also, leaves out the moisture test. Instead, the inspector estimates the maximum dry density, and the optimum moisture, by plotting a Proctor type wet density curve of the augmented test hole sample at three moisture levels. This method is best suited to fine-graded material on the dry side of optimum.

The Humphres Method plots the maximum unit-weight curve of granular materials against different percentages of fine aggregates compacted by normal construction equipment. In this method the inspector dries and analyzes the field sample and relates fines-content and unit-weight.

The CDW Method, as previously stated, measures the in situ volume of the soil sample by balloon or sand cone. The inspector, using visual and tactile criteria wets or dries the sample, compacts it on the spot in a cylindrical mould and reads the compacted volume by means of a calibrated dipstick.

Nuclear Methods: Unlike the conventional method of measuring soil density (weighing a quantity of soil whose volume is known) the nuclear scattering method uses a gamma emitting isotope which is placed a fixed distance from a radiation detector. A shield is placed between the radiation source and the detector to prevent direct radiation from being detected. The number of radiations detected in this manner is a function of the amount of gamma-ray scattering, and the assumption is made that this scattering is a function only of the soil density and therefore the detector response is a direct measure of soil density (5).

The Portable Nuclear Gauge measures wet density or moisture density (pounds of water per cubic foot). Calibration curves are prepared in terms of count ratio (back-scatter number of the soil divided by the back-scatter number of a reference block). Soil is removed to the depth of disturbance caused by the compactor. The surface has to be levelled and covered with a thin layer of fine material and the gauge must be rubbed down into firm contact with the soil. The probe measurement expresses mainly the density of the surface inch of the soil layer and is affected by the seating technique used and by the surface condition. The instrument needs careful adjustment before reliable readings can be obtained. Calibrations by the manufacturer generally do not agree with the operator's calibration and opinions about the accuracy of measurement and the technique to be used vary widely.

The Lane-Wells Road Logger records back-scattered radiation on a strip chart. The moving logs represent continuous average measurements through integration of the count rate over a fixed distance of past travel. The nuclear probes are mounted on two-wheeled carriages. During testing the carriages are lowered until their wheels touch the ground and this leaves a controlled gap between the soil and probe. Road Logger readings are less biased towards the surface and it is claimed that the effect of the soil's chemical composition on the radiation reading is smaller than in the case of the portable nuclear gauge. High cáliber personnel are required to operate the Road Logger and the down-time is stated to be high. It is less suitable on rough surfaces such as sheepsfoot compacted fills.

DEVELOPMENT OF THE CDW TEST

The relative ease with which weight measurements can be performed may well be the reason for basing conventional compaction measurements on weight. By comparison volume measurements of solids are generally more difficult. However, if the in situ volume of a soil sample can be compared with the volume of the same soil sample after standard compaction in the Proctor mould (at approximately optimum moisture), weight and moisture measurements are no longer needed. These volume measurements would give a direct indication of the state of compaction of the in situ material which would be readily understood by the man in the field.

Following this line of thought, a calibrated gauge or dipstick was used to measure the volume of soil in a Proctor mould. The dipstick was calibrated in one-thousandths of a cubic foot and volumes were readable to the nearest ten-thousandth of a cubic foot. *(See Figure 3)* Since it was necessary to bring the soil sample to its optimum moisture for standard compaction, the question arose whether the optimum moisture condition could be identified with sufficient accuracy by an inspector in the field.

Field experiments indicated that the approximate optimum moisture content of a sample could be recognized visually in the sample pan. This conclusion is supported by experience reported in the literature (2).

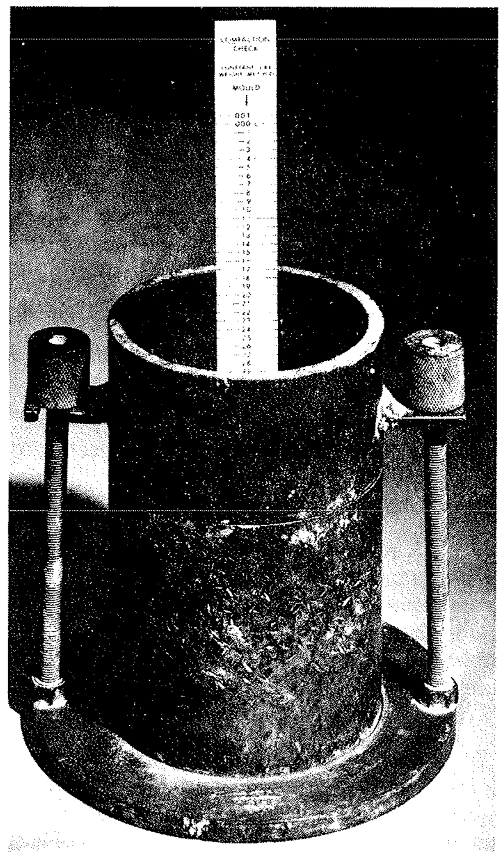

FIGURE 3, CALIBRATED DIPSTICK AND THE PROCTOR MOULD

The prospect of obtaining a quick and convincing estimate of the percent compaction based on a simple volume measurement, was a very attractive one, and it led to the development of the CDW method.

Principle of the CDW Test Method

The principle of the Constant Dry-Weight compaction test method is that **the volume of a fixed weight of soil is inversely proportional to its dry density, irrespective of its moisture content.** This can be expressed as follows:

$$D_1 = \frac{S}{V_1} \qquad D_2 = \frac{S}{V_2}$$

where: D_1 and D_2 represent different dry densities of the same soil sample
V_1 and V_2 represent the corresponding volumes of the soil sample
S = the dry weight of soil in the sample
It follows that:

$$\frac{D_1}{D_2} = \frac{V_2}{V_1}$$

and that the sample will be at maximum dry density (D_{max}) when its volume is smallest (V_{min}) as follows:

$$\frac{D_{max}}{D_2} = \frac{V_2}{V_{min}}$$

If V_2 is the volume of the test hole and V_{min} is the smallest volume of the sample measured in the Proctor mould, then:

$$\text{Percent Compaction } C = \frac{V_{min}}{V_2} \times 100$$

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Preliminary Tests

Preliminary tests were made to see whether this principle could be used for field compaction control. From a 15 lb. soil sample, 10 lbs. were sent to the laboratory for a full standard test for maximum density. The remaining 5 lbs. were standard compacted in a Proctor mould in the field and the compacted volume, V_1, was measured with the aid of the calibrated dipstick. The sample was then returned to the pan and 0.1 lb. of water (about 2 percent of the sample weight) was added or removed, according to whether the sample appeared to be below or above the optimum moisture content for compaction. The sample was recompacted and the volume, V_2, was measured. If V_2 was smaller than V_1, the same operation was repeated to obtain a further value, V_3. If V_2 was larger than V_1, the moisture change was reversed (the sample was dried instead of wetted). The lowest measured volume (V_{min}) corresponded to the maximum dry density (dry weight of the sample by V_{min}) and was compared with the laboratory maximum dry density. These tests showed that it was feasible to read volumetric measurements in the Proctor mould with a calibrated dipstick, and that very little was to be gained by repeating the compaction test at three or more moisture conditions (one moisture adjustment usually came very close to the smallest volume of the sample).

The CDW Test Procedure

The CDW testing procedure is as follows:

Step 1. The material extracted from the field density test hole (approximately 1/30 c.f.) is placed in a pan and examined (by sight and touch) with regard to its moisture content *(See Figure 4).*

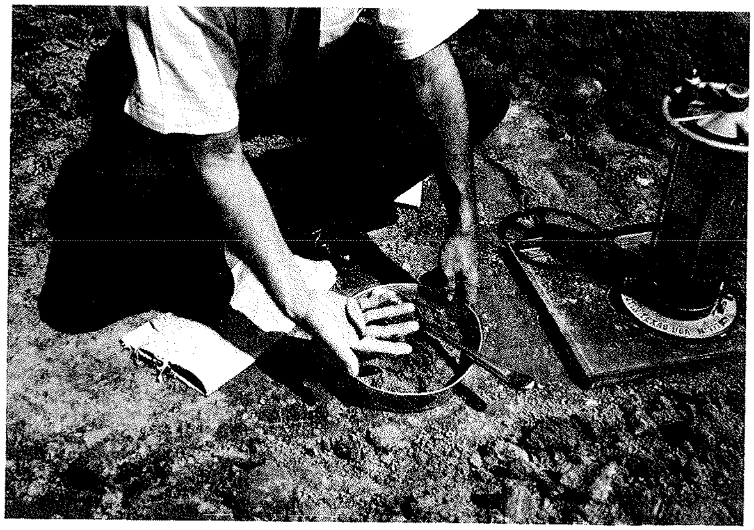

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FIGURE 4, CLAY SAMPLE BEING EXAMINED FOR MOISTURE CONTENT

(a) **Cohesive Soils.** In the case of cohesive soils, by attempting to roll a 1/8 in. thick thread, it can readily be seen whether the soil moisture content is below or above the plastic limit. The optimum moisture appears to be always below the plastic limit (6), and if the sample seems to be distinctly above the plastic limit, the material is allowed to dry in the sun or over a gasoline stove. The soil is dried until it loses much of its cohesiveness but can still be formed into a ball which retains its shape when pressure is released. If the soil sample is dry, the reverse applies; water is added in small amounts until the soil begins to show signs of cohesiveness.

(b) **Non-Plastic Soils.** In the case of dry silty-sands and silts, the sample is wetted until vigorous shaking of the sample in the pan causes soil lumps to show moisture at the surface as a sign of dilatancy. If the soil is on the wet side of optimum, the sample is dried until it ceases to show signs of dilatancy.

Coarse-grained granular soils which include sands and gravels and dense-graded crushed-rock can be placed in the mould at a moisture content somewhat above saturation. Drainage during the test reduces the water content to a value commensurate with the voids at maximum density (7).

Step 2. The sample, now believed to be at optimum moisture content, is compacted in the Proctor mould. (*See Figure 5)* The specified compactive effort for compaction test

ASTM D-698-58T consists of 75 blows of the Proctor hammer for a sample volume of .033 c.f. Since the test hole volume varies from the standard volume, the number of hammer blows is moderated as indicated on the reverse side of the dipstick *(See Figure 6).*

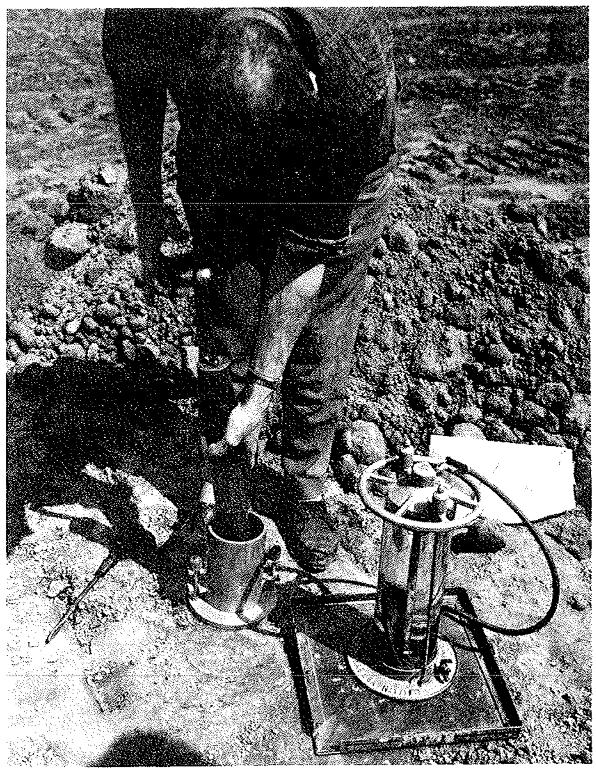
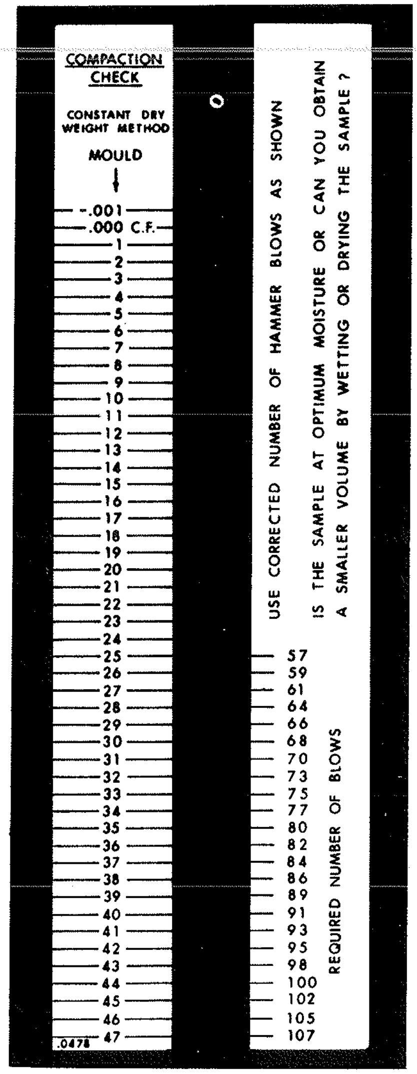


FIGURE 5, COMPACTING SOIL SAMPLE IN PROCTOR MOULD

Step 3. The volume of the sample in the mould is measured with the calibrated dipstick *(See Figure 7)*, using the average of 5 readings (i.e., 4 at the circumference of the compacted surface, and 1 in the centre). Each measurement is made with the calibrated dipstick resting lightly on the surface of the soil, and the dipstick reading that is in line with the top edge of the Proctor mould is noted.



FRONT VIEW

REAR VIEW

FIGURE 6, THE CALIBRATED DIPSTICK

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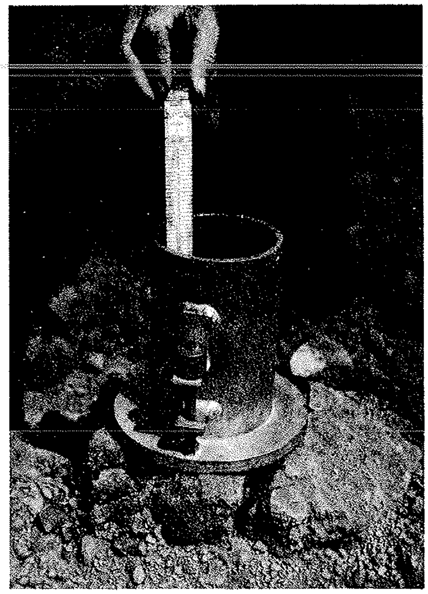

FIGURE 7, DIPSTICK READING OF VOLUME IN MOULD

The 'percent compaction' of the soil in the test hole is the ratio of the volume in the Proctor mould to the test hole volume. If there is any doubt about the estimate of the optimum moisture content additional Proctor runs can be performed after the moisture content of the sample has been re-adjusted. The lowest volume obtainable is used for the estimate of percent compaction.

Figure 8 shows the field compaction test form used by the Department of Highways, Ontario, and Figure 9 shows the test pad cover with guide lines for the periodic dispatch of control samples to the laboratory.

Distinguishing Features of the CDW Test

The CDW compaction test method does not use the moisture-unit weight curve for the computation of optimum moisture and maximum dry unit weight, but uses visual and tactile criteria suitable in field conditions. It measures only volumes, not weights.

The Inspector plays a more active part in the field testing because he produces specimens of the desired compaction himself. Unlike other compaction test methods no correction for stone content of the in situ dry density is necessary (calculation of the stone correction is controversial because of the effect of stone size and shapes). (8) This problem does not arise with the CDW method because each test is self-contained.

DEPARTMENT OF HIGHWAYS — ONTARIO
FIELD COMPACTION SHEET

CONTRACT NO. ________ HWY NO ________ LOCATION ________
DIST. ________ MATERIAL SOURCE ________
TWP. ________ STA ________ DIST FROM C.L. ________
DEPTH BELOW GRADE ________ TEXTURAL CLASSIF. ________

SOIL + PAIL =
PAIL =
WT. OF SOIL (gms.) =
(A) WT. OF SOIL (lbs.) =

(C) WET DENSITY = $\frac{A}{B}$
D DRY DENSITY = $\frac{C \times 100}{100 + \% \text{ MOISTURE}}$

FINAL READING =
INITIAL READING =
B VOLUME =

INITIAL WT. OF SAND =
FINAL WT. OF SAND =
WT OF SAND USED =
CONE CORR'N WT. (no.) =
NET WT. SAND (gms.) =
NET WT. SAND (lbs.) =
∴ DENSITY OF SAND =
B VOLUME =

CONSTANT DRY WEIGHT METHOD
SAMPLE WAS WETTED ☐ DRIED ☐
DIPSTICK READINGS
1
2
3
4
5
TOTAL ________

(E) VOLUME AVERAGE OF 5 READINGS =

SAMPLE FOR LABORATORY CHECK
WILL ☐ WILL NOT ☐ BE SENT
SEE NOTE INSIDE COVER

ESTIMATED % COMPACTION $\frac{D}{\text{LAB. MAX. DRY DENS.}}$ = $\frac{E}{B}$ =

GRADING OPERATION — GOOD ☐, FAIR ☐, POOR ☐, ________
COMPACTION — GOOD ☐, FAIR ☐, POOR ☐, ________
COM. EQUIPMENT ________ IN USE ________
DEPTH OF LIFTS ________ MOISTURE - BELOW OPT. ☐, AT OPT. ☐, ABOVE OPT. ☐
CROWN — GOOD ☐, FAIR ☐, POOR ☐, DISHED ☐, WATER TRAPPED ☐
REMARKS ________

SAMPLED BY ________ DATE ________ WEATHER ________
DATE REQUIRED ________ DATE RECEIVED ________
FIELD SAMPLE NO. ________ LAB. SAMPLE NO. ________

FORM OS-MT-3—67-1478

FIGURE 8, FIELD COMPACTION TEST FORM

NOTE FOR GRADE INSPECTORS

CHECK SAMPLES FOR LABORATORY PROCTOR TEST

Check samples should be sent from time to time to the laboratory so that the field compaction test results obtained by the inspector can be compared with the standard laboratory compaction test results.

The laboratory check indicates how accurately the field test is performed and it enables the inspector to perfect his field testing.

Since the sample from the density test hole used for the conventional or dry weight compaction tests are not large enough for a laboratory proctor test, the check sample for the laboratory consists of two parts: one part, large enough to do a proctor test, is taken from around the test hole, and the second part is the test hole sample which was used in the field test. The purpose for sending the second sample is to see whether the laboratory proctor test material closely enough resembles the field test hole material.

PREPARATION OF CHECK SAMPLES FOR LAB. PROCTOR TEST:

1. Two samples must be taken, each with its own sample sheet.

2. First Sample:
(a) This consists of 10 pounds (for earth), or 25 pounds (for granular base coarse materials) taken from near the test hole, and of a moisture sample from it in a quarter pint glass jar.
(b) Record wet density on field compaction sheet.
(c) Record % compaction on field compaction sheet.
(d) In remarks space write: proctor, moisture, hydrometer and p.i. (or sieve analysis) required.
Attention: Regional Materials Engineer

3. Second Sample:
(a) This sample consists of the material from the proctor mould and of a moisture sample from it in a quarter pint glass jar.
(b) In the remarks space write: moisture, hydrometer and p.i. (or sieve analysis) required. Attention: regional materials engineer. See sample No. (number of above mentioned first sample)

WHEN USING C.D.W. METHOD CHECK PROCTOR MOULD:
See whether the dipstick reading in the empty proctor mould is zero. If it is not zero, but, for example zero plus .0002, or zero minus .0003, mark this mould correction on the mould and subtract (for zero plus), or add (for zero minus) the correction to the average dipstick reading before calculating the % compaction.

FIGURE 9, GUIDE LINES FOR INSPECTORS

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Accuracy and Precision of the CDW Test

Ideally, the CDW compaction result (Volume in mould/Volume in situ) should be compared with the percent compaction (the laboratory maximum dry density/in situ dry density). This is not possible because the field test sample is too small for a laboratory standard compaction test. The following procedure is adopted instead:

A 10 lb. sample taken from around the test hole and a moisture sample channelled from the side of the density test hole is sent to the laboratory (sample A) and the laboratory maximum dry density and the field moisture content are ascertained.

The CDW maximum dry density is calculated by dividing the CDW percent compaction into the field dry density and multiplying by 100. For instance, for a CDW compaction of 94 percent and a field dry density of 120 pcf, the CDW maximum dry density is 128 pcf.

The soil sample from the density test hole (sample B), used in the field Proctor (CDW) mould, is also sent to the laboratory.

Sieve or hydrometer analyses are then carried out, and for clays the Atterberg Limits are ascertained on the A and B samples, since the difference in these parameters is expected to be a major factor in the variation between the CDW and the laboratory maximum dry densities.

Nine testers in different parts of southern Ontario took part in the first correlation study in 1962. Their daily task was to dispatch to the laboratory an A-sample, a B-sample and the wet density measurement from one of their routine CDW tests chosen at random.

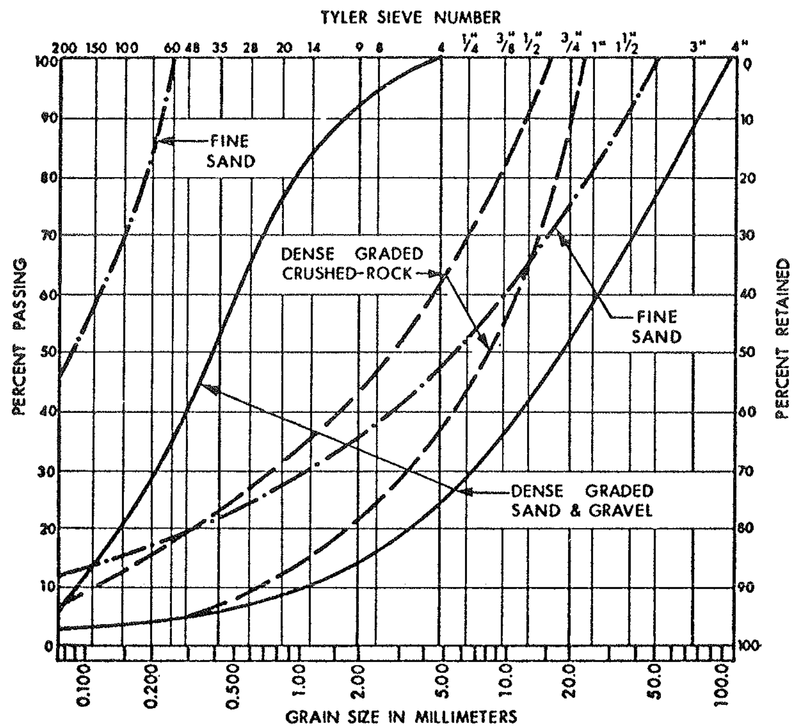

FIGURE 10, TYPICAL RANGE OF GRADATION

The samples in some areas (Hamilton, Brantford and St. Catharines) were numerous enough to be grouped separately into Fine Sands, Dense Graded Sand and Gravel, Dense Graded Crushed-Rock *(See Figure 10 for gradation)* and Clays. From the Toronto area only 10 check samples were received which were not separated into groups. Table 1 shows the differences between CDW and Laboratory test results.

Some of the differences between the CDW and the laboratory test results can probably be attributed to the difference in the gradation between the test hole sample-B and the laboratory compaction sample-A. For instance, if the gradations of the crushed-rock laboratory samples in Figure 11 are compared with the Weymouth curve, the A-sample would be expected to be capable of a higher density than the B-sample. In this

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TABLE 1, COMPARISON OF DENSITIES OBTAINED BY THE CDW METHOD AND THE LABORATORY MAXIMUM DENSITY

SOIL	NO. OF TESTERS	NO. OF SAMPLES	CDW DENSITY MINUS LABORATORY MAXIMUM DRY DENSITY (pcf)			
			LOW	HIGH	AVERAGE	STANDARD DEVIATION
Fine Sand	3	23	-5	+5	0	3
Dense Graded Sand Gravel	3	14	-4	+5	0	3
Dense Graded Crushed-Rock	3	13	-4	+8	+1	4
Clays	3	24	-8	+5	-2	4
(Hamilton Area)						
Fine Sand	4	14	-9	+3	-2	3
Dense Graded Sand Gravel	4	30	-12	+8	0	5
Dense Graded Crushed-Rock	4	31	-10	+8	0	4
Clays	4	38	-6	+7	-2	3
(Brantford Area)						
Fine Sand	1	6	-4	+4	-1	3
Dense Graded Sand Gravel	1	8	-3	+3	0	2
Dense Graded Crushed-Rock	1	9	-4	+2	0	2
Clays	1	6	-3	+4	0	3
(St. Catharines Area)						
Fine Sand, Dense Graded Sand Gravel, Dense Graded Crushed-Rock, Clays *(Toronto Area)*	1	10	-6	+3	0	3

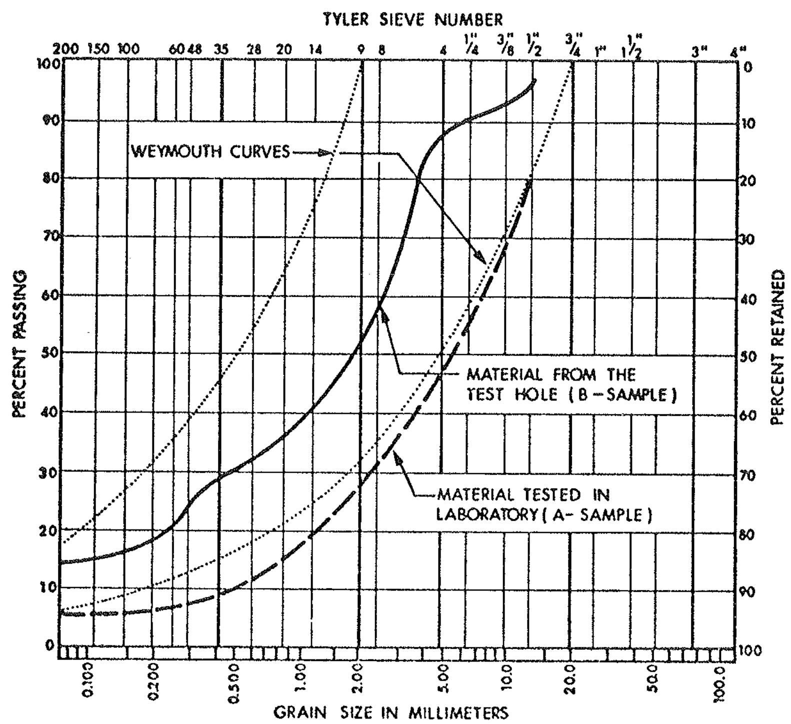

FIGURE 11, GRADATION DIFFERENCES BETWEEN EDGE OF TEST HOLE (SAMPLE A) AND TEST HOLE (SAMPLE B) particular case, the laboratory Proctor maximum dry density for the A-sample was 140 pcf whilst the CDW maximum dry density of the B-sample was 132 pcf. In another instance of a sand-gravel material *(See Figure 12)* the A-sample would be expected to be capable of only lower densities than the B-sample, and, in fact, the laboratory Proctor for sample-A was 124 pcf, and the CDW maximum dry density for sample-B was 132 pcf. Where the grading curves of the A- and B-samples were very similar *(See Figure 13)* the laboratory maximum density was 143 pcf and the CDW maximum dry density was 141 pcf.

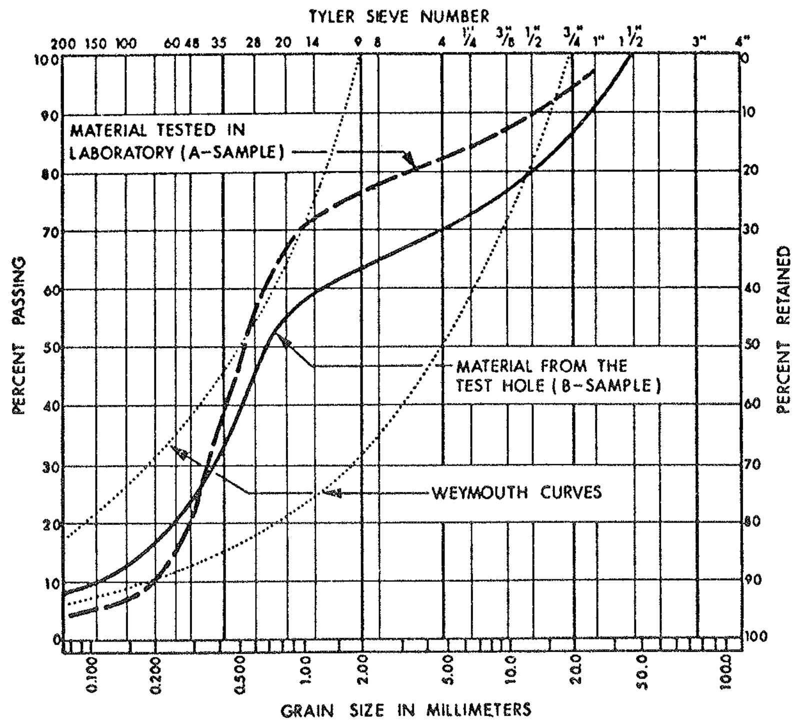

FIGURE 12, GRADATION DIFFERENCES BETWEEN EDGE OF TEST HOLE (SAMPLE A) AND TEST HOLE (SAMPLE B)

The quite commonly found differences in the gradation of A-samples (taken from around the density test hole) and B-samples (taken from the density test hole) emphasize the advantage of a self-sufficient field compaction test, independent of tests on samples other than those obtained from the test hole itself.

REPRODUCIBILITY OF COMPACTION TEST RESULTS

The maximum dry unit weight obtained by the ASTM D698-58-T, Method C test, does not, on repetition, vary by more than ±1 pcf (8), but if the soil contains a high proportion of large aggregate, or aggregates that degrade under compaction, or if the soil is thixotropic or has high expansion characteristics, variations of ±2 pcf or more from the median are not unusual (9).

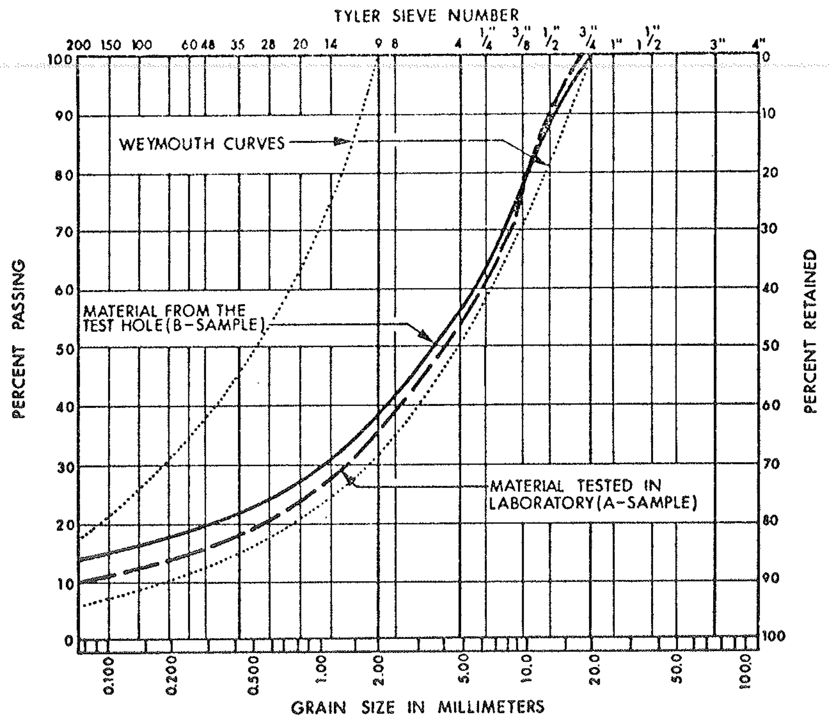

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FIGURE 13, GRADATION DIFFERENCES BETWEEN
EDGE OF TEST HOLE (SAMPLE A) AND TEST HOLE (SAMPLE B)

Co-operative tests show that the variations are greater for different testers in different laboratories (10). Forty-four agencies performed the Standard AASHO compaction test T99-57, Method A, on a light clay. The reported maximum dry unit weights varied from 114.0 to 125.1 pcf (i.e., 11.1 pcf between 'high' and 'low') with a standard deviation of 2.2 pcf.

By comparison, the CDW-minus-Laboratory difference for maximum dry unit weights of clay, as shown in Table 1, was 13 pcf (i.e., a low of −8 pcf and a high of +5 pcf): The largest standard deviation was 4 pcf.

The variance in compaction test results of other co-operative tests (7) is shown in Table 2. In this case, several co-operators determined the gradation of the samples, and some determined the moisture-density relationships using ASTM Method D698 and modified ASTM Method D698. The gradation data showed considerable variations in results even though the samples for each co-operator were carefully prepared and should have been identical.

Table 3 shows that the field-minus-laboratory test result variance is of the same order as the reproducibility of standard laboratory test results (11).

TABLE 2, RESULTS OF CO-OPERATIVE STUDY OF THE STANDARD COMPACTION TEST (5) (METHOD ASTM D698-57T, AASHO T99-57)

SOIL	NO. OF TESTERS	MAXIMUM DRY UNIT WEIGHT (pcf)				
		LOW	HIGH	HIGH-LOW	AVERAGE	STANDARD DEVIATION
Fine Sand	5	101	107	6	103.4	2.5
Dense Graded Sand Gravel	4	133	141	8	137.5	3.4
Dense Graded Crushed-Rock	4	132	143	11	135.5	5.1

TABLE 3, COMPARISON OF CO-OPERATIVE TEST VARIANCES

SOIL	GROUP OF TESTERS	NO. OF TESTERS	NO. OF SAMPLES	HIGH-LOW pcf	AVERAGE VARIATION	STANDARD DEVIATION
Fine Sand	ASTM	5	5	6	does not apply	2.5
	Hamilton	3	23	10	0	3
	Brantford	4	14	12	-2	3
	St. Catharines	1	6	8	0	3
Sand-Gravel	ASTM	4	4	8	does not apply	3.4
	Hamilton	3	14	9	0	3
	Brantford	4	30	20	0	5
	St. Catharines	1	8	6	0	2
Crushed-Rock	ASTM	4	4	11	does not apply	5.1
	Hamilton	4	13	12	2	4
	Brantford	4	31	18	0	4
	St. Catharines	1	9	6	0	2
Clay	AASHO	44	44	11	does not apply	2.2
	Hamilton	3	24	13	-2	4
	Brantford	4	38	13	-2	3
	St. Catharines	1	6	7	0	3

ANALYSIS OF DIFFERENCE BETWEEN CDW-AND-LABORATORY PROCTOR RESULTS

A statistical analysis of the previously mentioned first correlation study is detailed in the Appendix to this report. The following is a summary: Because of the operational character of the test series it was not possible to separate all sources of variation. The precision of the methods could not be analyzed because the tests on each sample pair were not repeated, but the effect of soil type and tester on the differences between the results of the two methods was evaluated by an analysis of variance from which the following conclusions were drawn;

(a) **Influence of Tester.** The tester's influence on the differences between the CDW and laboratory test methods is almost significant. Since differences between the two methods, not the methods themselves, were analyzed, it cannot be said whether the CDW test or the laboratory test, or both test methods are subject to this variation. The average differences for testers A, B and C are −0.6, +0.9 and +0.2 pcf respectively. These differences are not significant. However, since according to the variance analysis the probability that differences exist between the testers is almost significant, it is desirable to obtain more test data before this source of variation is dismissed.

(b) **Influence of Material.** The test materials have no significant effect on the differences between the test results obtained by the CDW and laboratory methods and it may be concluded that both methods are of equal merit in this respect.

(c) **Influence of Interaction.** There is no significant interaction between testers and type of soil tested.

(d) **Bias.** The average of all differences, according to the table in Appendix A, would indicate that the CDW method runs about 0.4 lbs. lower than the laboratory method. However, since the results of tester B, who performed more than half of all the tests, also ran low, more test data from more testers are required before an estimate of bias can be made.

(e) **Standard Deviation.** The estimate of the standard deviation, 3.5 pcf, of the differences between the CDW and laboratory test results shows how the differences are distributed and with what frequency they can be expected.

(f) **Significance of Bias.** In order to determine whether the previously mentioned bias of the analyzed CDW test results is significant, the Student's 't' test for Paired Differences was applied. The value of 't' is 1.4169 for a probability of 0.05 and 173 degrees of freedom. This is well below the critical 't' value of 1.970. It was, therefore, concluded that there is no significant difference in the level of results between the two test methods.

THE CDW TEST AND THE STATISTICAL APPROACH TO COMPACTION CONTROL

A statistical quality control study of highway construction in southern Ontario was carried out in which the conventional laboratory compaction test and the CDW test were used. The test series was modelled on the guide lines suggested by the Statistical Quality Control Task Force of the U.S. Bureau of Public Works (12), for compacted embankments, bases and sub-bases. The program was designed to obtain data for establishing statistical parameters pertaining to density, percent compaction and moisture content.

Duplicate samples were taken in 50 random locations in each of 8 projects. Each project contained a minimum of 50,000 cu. yds. of embankment or 10,000 cu. yds. of base material. Duplicate tests were taken within a square yard unit to calculate the 'testing error' and each square yard unit was tested by the CDW method in the field and by the standard ASTM test method in the laboratory. The test results in Table 4 show that: the test errors, i.e., the difference between the test results within a one square yard test location unit, are very nearly the same for standard and CDW tests.

Table 5 shows that the over-all average percent compaction measured by laboratory compaction test was 96.9 with a standard deviation from the mean of 4.8 percent. The corresponding results for the CDW test were 97.7 and 4.4 percent.

Table 6 shows the mean difference and standard deviation from the mean between CDW test and the Standard Laboratory test. The average of the standard deviations given in this table shows that the variation from the mean Laboratory Maximum Dry Density in the large mass of road material is of the same order as that of the difference between CDW and Standard Laboratory test. From this it could be inferred that the CDW-standard test difference actually reflects the variation of the material within the square yard of the test unit and that this variation is of the same order as the variation within a large mass of the same material. It may also be noted that E.T. Selig, in his important report (13) on the variability of compacted soils, in which nuclear moisture and density data were used, quotes a standard deviation of 3.8 pcf for wet density, within lift, excluding known instrument errors (i.e., 2.7 percent for an average wet density of 140 pcf).

TABLE 4, PRECISION OF TEST

NUMBER OF TESTS	SOIL TYPE	PERCENT COMPACTION			
		CDW TEST ERROR		STANDARD TEST ERROR	
		MEAN PRECISION	STANDARD DEVIATION	MEAN PRECISION	STANDARD DEVIATION
150	Clays	2.9	2.5	3.2	2.7
100	Sand Cushion or Granular Base 'B'	2.5	2.3	3.0	2.7
100	Granular Base 'A'	2.6	2.3	2.5	2.0

TABLE 5, COMPACTION OBTAINED BY STANDARD COMPACTION METHOD, AND BY CDW METHOD

SOIL TYPE	PERCENT COMPACTION			
	STANDARD LABORATORY TEST		CDW TEST	
Clay	94.3	5.7	95.1	4.2
Sand Cushion or 'B'	98.0	4.3	98.9	3.5
'A'	101.8	3.6	103.3	5.3
Average	98.0	4.6	99.1	4.4

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FIGURE 6, COMPARISON OF CDW AND CONVENTIONAL FIELD COMPACTION TEST RESULTS, AND THE VARIATION OF THE MAXIMUM DRY DENSITY OF LABORATORY SAMPLES

NUMBER OF TESTS	SOIL TYPE	CDW TEST PERCENT COMPACTION MINUS LABORATORY TEST COMPACTION		AASHO T99-57 % OF MAXIMUM DRY DENSITY
		MEAN DIFFERENCE	STANDARD DEVIATION	STANDARD DEVIATION
150	Clays	1.0	4.2	4.5
100	Sand Cushion or Granular Base 'B'	.3	2.5	4.7
100	Granular Base 'A'	1.8	3.5	2.6

USE OF THE CDW TEST WITH STATISTICAL PROCEDURES

The average percent compaction and the standard deviation from the mean of several CDW tests can be ascertained in a sufficiently short time to make it suitable for a continuous type of statistical compaction control.

The State of California Division of Highways describes the statistical 'area concept' (14) of compaction control as follows:

"An area of embankment is selected to be tested for acceptance, with this area varying from a few hundred to a few thousand feet in length. Six or more in-place density determinations are performed at random locations throughout this area with a nuclear soil density gauge. A standard impact maximum density test is performed on a soil sample, from the test site, whose density is just below the average density of the area. The relative compaction is then calculated at each test site. The average relative compaction of the area is then calculated and must be above the required value. Also at least two-thirds of the individual test site relative compactions must be above the required relative compaction value."

The declared intention of the method is to reduce the number of time consuming moisture tests. This can be realized with the aid of the CDW test, and if nuclear gauges and trained personnel are available, the CDW test can be used in conjunction with the nuclear gauge, as described later in this report. It is not the intent of the authros of the area concept to restrict its application to nuclear tests and it is here suggested that the CDW test could, in this context, be used to advantage since the difficulties of multiple calibration curves for different soil types in the case of backscatter gauges, the problem of instrument setting, the need of specially trained personnel for maintenance and for the assurance of health and safety are avoided. The area concept can then be used also on small projects with a minimum of staff and equipment.

INTERPRETATION OF CDW TEST RESULTS WITHOUT THE USE OF STATISTICAL PROCEDURES

Most construction tests (crushed concrete cylinders, steel shear strength, etc.) are carried out on individual members which, statistically speaking, represent the population to be judged. In these circumstances an **indirect** judgment about the condition of other members of the population can be made. In the case of the CDW test the circumstances are different insofar as the member tested is the investigated object itself and not just representative of it. It is, therefore, not necessary when judging the individual location, to have reservations about the relevance ot the test result, on the grounds that it may not be truly representative of the tested location. A fill location, which the CDW test shows to be compacted less than the specification requires, can be improved to the required standard. Although the soil mass is by its nature variable, the level of compaction can be locally controlled and corrected.

It is a feature of the CDW test, that, without using the statistical technique, especially on small and medium size projects, a direct compaction measurement of the tested location is obtained and a decision can be made immediately as to the required action.

THE CDW TEST USED IN CONJUNCTION WITH THE NUCLEAR GAUGE

Whilst the nuclear gauge lends itself to comparing density levels it is less suitable for use on projects where compaction requirements are specified in terms of ASTM standard compaction. The CDW test was used to supplement the nuclear probe in the following manner: Nuclear and CDW tests were taken in two locations - one in compacted and one in uncompacted material, and a calibration line was plotted with the nuclear count as the ordinate and the CDW percent compaction as the abscissa. This calibration line was used for converting nuclear test results in the same material into 'percent compaction'.

CONCLUSION

The accuracy and the precision of the CDW compaction test method are comparable with that of the standard laboratory test and because of its speed and simplicity the CDW test is suitable for field compaction control.

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APPENDIX A, STATISTICAL ANALYSIS OF CORRELATION DATA FOR CDW TEST AND LABORATORY PROCTOR TEST

The correlation study was intended to interfere as little as possible with normal construction operations. In order to prepare a quasi-symmetrical table of data some tests had to be discarded. Only the results made by three testers who had tested several samples in each of four categories of soil were used, with the result that 174 pairs of data were retained for the analysis. The table was not symmetrical, however, with respect to the number of tests performed by each tester on each soil type, nor with respect to the total number of tests performed by each tester.

When data of this type are subjected to an Analysis of Variance it is usual to determine:

1) if the two analytical methods differed in level of results or in precision
2) if either method was subject to variation due to different testers
3) if either method varied due to soil type or if soil type affected the correlation between the methods
4) if any other interactions not mentioned above existed between testers, soils and methods.

The arrangement of the correlation tests did not permit the separation of the above sources of variation for the following reasons:

1) the operational nature of the test method did not permit replication, i.e., repeated analyses on the same material
2) the Laboratory Proctor results were run by any one of several testers who happened to be available at the time
3) it must also be noted that the laboratory and the field operator were not testing exactly the same material. 185

Consequently, each difference between the CDW density test and the Laboratory Proctor test was listed. Each difference represented the algebraic sum of variations from the following sources:

1) variations among the analysts who performed the Laboratory Proctor tests
2) variations inherent in the Laboratory Proctor methods
3) variations inherent in the CDW method
4) variations between the material analyzed in the field and that analyzed in the laboratory.

None of these variations can be separated and their over-all effect will appear in the mean square for error.

Thus, a series of positive and negative differences was obtained for each CDW tester and for each soil type.

Table A-1 shows the algebraic sums of each series of test differences. The figures in brackets show the number of test differences in each series. The estimate of the standard deviation of the difference is:

$$S_{(diff)} = \sqrt{12.08} = 3.5 \text{ pcf}$$

The following is the interpretation of the above analysis:

1) The tester effect on the differences between the two methods is almost significant at P = 0.05. Since we are dealing with a difference we do not know

APPENDIX A, STATISTICAL ANALYSIS OF CORRELATION DATA
FOR CDW TEST AND LABORATORY PROCTOR TEST (CONTINUED)

whether the Proctor test, the CDW test or both tests are subject to this variation. If, however, Table A-1 is examined it can be seen that the average differences of the three testers are small. Tester A runs about 0.6 pcf low while B runs 0.9 pcf high and C runs 0.2 pcf high. These small differences are insignificant when the level of the densities and the uses to which they are put, are considered. The differences would be real at P = 0.07 but since they are so small the tester effect may be of little significance. Since the results of only three testers were analyzed more data on other testers would have to be obtained before this source of variation can be dismissed.

2) The soil type has no significant effect on the differences and it may be concluded that both methods are equal for all types of soil surveyed in this experiment.

3) There is no significant interaction variation between tester and soil types.

4) The average of all of the differences in Table A-1 might be thought to indicate that the CDW method is biased about 0.4 pcf lower than the Proctor method. In interpreting this result it must be remembered that tester B who performed over half of the total tests also ran low. Before an estimate of bias can be made, more data from more testers would have to be obtained.

5) The estimate of the standard deviation of the differences indicates how the differences are distributed and with what frequency they can be expected.

To determine whether the two test methods produce different results, the Students 't' test for Paired Differences was applied. The 't' value for 174 pairs was 1.4169, which is well below the 't' value (1.970) at a probability of 0.05. It was concluded, therefore, that there was no significant difference between the two test methods.

TABLE A-1, DIFFERENCE BETWEEN CDW DENSITY AND LABORATORY DENSITY, IN POUNDS PER CUBIC FOOT

SOIL TYPE	TESTER A	TESTER B	TESTER C	TOTAL	AVERAGE
Fine Sand	+6 (18)	-25 (11)	-3 (6)	-22 (35)	-0.63
Sand-Gravel	-2 (13)	-11 (25)	-1 (8)	-14 (46)	-0.30
Crushed Rock	+17 (12)	-8 (30)	-4 (9)	+5 (51)	0.10
Clay	+7 (4)	-42 (32)	+1 (6)	-34 (42)	-0.81
Total	+28 (47)	-86 (98)	-7 (29)	-65 (174)	
Average	+0.60	-0.88	-0.24		-0.37

TABLE A-2, ANALYSIS OF VARIANCE RESULTS

SOURCE OF VARIATION	SUM OF SQUARES	DEGREE OF FREEDOM	MEAN SQUARE	'F' RATIO	SIGNIFICANCE
Analysts	69.56	2	34.78	2.879	N.S. at P = 0.05 Sig. at P = 0.07
Soil Type	21.81	3	7.27	0.6018	N S
Interaction	45.47	6	7.58	0.627	N.S
Error Within Each Set	1957.88	162	12.08		
Total	2094.72	173	....		

STATE OF VERMONT
DIVISION OF ENGINEERING AND CONSTRUCTION
133 State Street, Montpelier, Vermont 05602

June 13, 1979

Mr. Lloyd R. Crowther, JH513
Transportation Research Board
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

Dear Mr. Crowther:

Enclosed, is a copy of our Constant Dry Weight Test Procedure which I agreed to forward to you during our telephone conversation on June 6, 1979. I believe that everything in this procedure is plainly stated and can be easily understood.

If there are any questions or additional information needed, please call us at any time. We are glad to be of assistance.

Very truly yours,

R. F. Nicholson, P.E.
Materials & Research Engineer

By: Henry Haggerty
for Donald C. Brown, P.E.
Chief Research & Testing Engineer

Enclosure

Prepared By: D. C. Brown
Date: June 8, 1979
Page 1 of 2

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

CONSTANT DRY WEIGHT (C.D.W.) TEST PROCEDURE

SCOPE:

The Constant Dry Weight Method was developed for a quick and convenient compaction test based on simple volume measurement. The "percent compaction" of the soil in an embankment is determined from the ratio of the compacted volume in the standard Proctor mold to the volume of the test hole. This method eliminates the need for moisture-density curves and stone corrections.

The formula used for "percent compaction" is:

$$C = \frac{V_1}{V_2} \times 100$$

Where: C = Percent Compaction
V_1 = Volume of compacted soil in mold
V_2 = Volume of test hole

APPARATUS:

All of the below - listed apparatus shall meet the requirements of AASHTO T-134 with the exception of the Calibrated Dipstick.

1. Sand Cone Apparatus
   a. 6" cone with matching plate
   b. 1 gallon jug, with G mason top
   c. Free flowing, calibrated sand
2. Scales - 25 lb. capacity

   Reading to nearest .01 lb.
3. 4" Proctor mold
4. 5.5 lb. rammer
5. Dipstick (calibrated to one thousandths of a cubic foot)
6. Miscellaneous Tools: mixing pan, rubber mallet, spoons, brushes, chisel, etc.

State of Vermont A.O.T.
Materials & Research Div.
Constant Dry Weight (C.D.W.) Test Procedure

June 8, 1979
Page 2 of 2

PROCEDURE:

1. The test site shall be cleared of all loose surface material and leveled to a size sufficient to accommodate the sand cone plate. The material is extracted from the test hole (approx. 1/3 of a gallon pail) and the sand cone apparatus installed. After the test hole is filled with sand, remove the jug and cone and recover the clean sand.

2. The jug and sand is weighed and recorded on the CDW worksheet. This weight is subtracted from the original weight of the sand and jug. The result is entered as weight of sand used. The sand cone correction is then subtracted from the weight of sand used to obtain the actual amount of sand used to fill the test hole. (net sand in hole). The test hole volume is obtained by dividing the net sand in the hole by the sand density. The sand density is the weight of sand in pounds per cubic foot predetermined by the laboratory.

3. The extracted soil from the test hole is examined for optimum moisture content. Moisture is added if it is too dry or dried back if it is too wet. When the material is at optimum moisture, it is neither mushy nor dry but contains sufficient moisture to make a firm cast when squeezed in the hand; water can not be squeezed out of the material and little will appear on the hand. With a little experience the correct amount of moisture can be determined within practical limits by feel.

4. The total sample now at or slightly below optimum is compacted in three equal layers in the Proctor mold. The specified compactive effort from AASHTO T-99 is 75 blows (3 layers at 25 blows each) with the 5.5 lb. rammer for a sample volume of .033 cubic feet. Since the CDW procedure must use all the material from the test hole, the number of rammer blows per layer shall be adjusted to the volume of the hole.

   a. NUMBER OF RAMMER BLOWS.

      The volume of the test hole is read on the dipstick volume side and by turning the dipstick over read the corresponding number of blows required. Divide this number by three to obtain the required blows per layer.

5. After compacting the sample in the mold the volume is measured with the dipstick using the average of five readings (4 at the circumference and 1 at the center). Each measurement is made with the dipstick resting lightly on the surface of the soil, and the volume readings taken at the top of the collar attached to the mold. The average of the readings is then divided by the net hole volume to determine the percent compaction. The "in place moisture condition" is noted on the worksheet as dry, optimum, or wet. See FIG.

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH DIVISION

COMPACTION TEST WORK SHEET DIPSTICK METHOD

PROJECT:____________________ CONTRACTOR:____________________

TEST BY:____________________ DATE:____________________

1. Test No.									
2. Station									
3. Offset									
4. Depth Below Subgrade									
5. Soil Type									
6. Wgt. Sand Jug (before)									
7. Wgt. Sand Jug (after)									
8. Wgt. Sand Used (6-7)									
9. Cone Correction									
10. Net Sand in Hole (8-9)									
11. Sand Density									
12. Net Hole Volume (10-11)									
13. Dipstick Readings 1.									
2.									
3.									
4.									
5.									
14. Total of Readings									
15. Aver. Volume of Readings									
16. % Compaction (15-12)									
17. Moisture Condition									

XVIth WORLD ROAD CONGRESS VIENNA SEPTEMBER 16–21, 1979

TECHNICAL COMMITTEE REPORT

ON

TESTING OF ROAD MATERIALS

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INTRODUCTION

During the preparatory work for the Mexico conference, the Committee on Testing of Road Materials established that there was a very wide variety of test procedures in use in the various countries concerned. While some of the differences result from different concepts of road functions and so are to that extent understandable, others by contrast have no sound basis at all. Consequently, the situation as a whole is very much open to criticism for it considerably hinders exchange of information between countries and the progress of road technology.

In view of this, the main aim of the Committee has been to contribute towards putting matters in order, both in respect of methods used in testing and of terminology. The work described in the present report concentrates on this dual objective. It seems to us to be of the greatest importance that the recommendations put forward here should, after examination and being put into final form (taking account of any comments which may be received by the Committee and the discussions to take place at the Vienna conference), be strictly applied by PIARC member countries. Only such a collective discipline within PIARC seems capable of remedying matters. We hope too that the recommendations will be published in the technical press of all member countries so that they will become widely known.

In conclusion, we should like to make it clear that the Committee has not worked in isolation but has set up close collaboration with RILEM whose work is on somewhat parallel lines. Accordingly, the Committee has only dealt with areas not covered by RILEM and proposes to submit the results of its work to that organization. Conversely, the Committee has made a contribution to RILEM's work in establishing terminology in the field of bituminous binders. The Committee wishes to stress the excellent spirit of cooperation shown in the joint work. Collaboration was also established with ASTM who were concerned in the report on the Marshall test; we hope that this cooperation will carry on in the future.

We now comment briefly on the various Sections of the report.

I. Recommendations on methods to be used for testing aggregates (draft)

The recommendations were prepared by a Working Group under Mr Moraldi (Italy). Generally speaking these are not operational test procedures as such but are recommendations on all matters which appear essential to ensure that tests are carried out uniformly in all countries. In those cases where an existing method of operation seems to be accepted in the majority of countries, it has been adopted with the addition of further details where required. For certain tests where recognized procedures are lacking, it has been necessary to go further and provide all details needed to ensure that the tests are carried out in the same way in all countries.

II. Analysis of replies to an international questionnaire on the Marshall test and its applications

The analysis was carried out by a Working Group under Mr Goodsall (UK). The Committee's report to the Mexico conference brought to light the wide-spread use made of the Marshall test in most countries. It also showed up the lack of uniformity of test procedures, some variations being justified by a concern to adapt the test to suit local conditions.

Given this, the Committee thought it would be useful to make a thorough analysis of the problem by way of a survey carried out by a questionnaire. Data from the survey are analysed in Section II, where the conclusions to be drawn are also presented.

III. Draft terminology relating to the treatment, improvement and stabilization of soils and materials for road foundations

At the present time, the treatment of soils and materials for road foundations is an area which is in a state of continuous flux. This results, on the one hand, from the arrival of new binders — often industrial by-products — and, on the other, from an increasing awareness of the value of these techniques. As a consequence, terminology too is changing all the time with new terms appearing in the various languages to interpret the technical evolution occurring.

The Committee decided that it was essential to consider materials terminology before tackling test procedures. The work presented here represents a first draft; it is a joint effort of the Committee, carried out under the leadership of Mr Paulmann (FGR). Any comments we receive on this subject will be particularly useful in enabling the proposals to be developed and finalised.

I. RECOMMENDATIONS ON METHODS TO BE USED IN TESTING AGGREGATES

PROPOSALS

1. Particle-size distribution by sieving.
2. Los Angeles test
3. Sand equivalent
4. Polished-stone-value test
5. Quantity of fine material passing a 0.075mm sieve
6. Density measurements
   - Relative density
   - Apparent relative density and proportion of absorbed water
   - Bulk density with or without compaction
7. Aggregate shape
   - Shape index based on the shape-coefficient using calipers
   - Flakiness index based on the flakiness coefficient
8. Sample reduction to provide the test sample
9. Sensitivity to freezing

1. PARTICLE SIZE DISTRIBUTION BY SIEVING

A. GENERAL

A1 Purpose of test

To determine particle-size distribution, by mass, of an aggregate.

A2 Principle of the test

The aggregate sample is sub-divided, using a series of sieves of decreasing aperture size.

A3 Remarks

A3.1 Most countries use square-mesh sieves; and a few, round-hole sieves for the fractions >2mm. Accordingly it appears advisable, in line with the Mexcico report, to recommend the use of square-mesh sieves. However, for the benefit of countries using round-hole sieves, an average conversion factor is recommended to enable data to be compared with those obtained with the square-mesh sieves.

A3.2 Two series of sieves are recommended, the ISO and the ASTM. Since the particle-size-distribution results are usually converted to graphical form, the actual values of the sieve apertures used are not important. However, it is recommended that 0.075mm (or ASTM No 200) sieve be used to separate the sand from the "filler".

A3.3 Wet sieving is recommended when the aggregate is dusty or contains clay, whether sticking to the grains or present in the form of lumps.

A3.4 The choice between hand and machine sieving is left to the user but criteria are laid down to define the completion of sieving.

B. RECOMMENDED PROCEDURE

B1 Equipment

B1.1 Special equipment.

B1.1.1 The series of sieves selected shall be from those standardized by ISO or ASTM, mesh sizes being chosen so as to define conveniently (or as laid down by the specifications) the sizes of aggregate components concerned. It is recommended, however, that 0.075mm sieve should always be used to separate the "sand" fraction from "filler". Round-hole sieves are not recommended. If, however, a country considers it desirable to use them, data so derived should be transformed into equivalent square-mesh sieve measurements by the following conversion ratio. *Side of square mesh aperture* = 0.8 × *round hole diameter.*

B1.2 Standard laboratory equipment

B1.2.1 Scales with a capacity appropriate to the test-sample mass, M (See B2.1) and whose sensitivity is equal to or better than 0.1% of M.

B1.2.2 A drying oven which is thermostatically controlled at a temperature of 110±5°c.

B1.2.3 A sampling device (optional)

B1.2.4 Sieving machine (optional)

B2 Preparation of test sample

B2.1 Minimum mass, M expressed in grams:
For $D_{max} \leqslant 20mm$, $M \geqslant 200D_{max}$
and
for $D_{max} > 20mm$, $M \geqslant 600D_{max}$
where D_{max} is the maximum aggregate-dimension in mm.

B2.2 Dry in the oven until mass remains constant.

B3 Test Procedure

B3.1 Sieve in the dry state if the test sample is clean, ie free from dust or clayey material.

B3.2 If the aggregate contains dust or clay use wet sieving to separate the material passing the 0.075mm sieve. The fabric of this sieve must always be protected by placing a 1 or 2mm sieve above it.

B3.3 Sieving end-point. The end point for any sieve, is when the mass of material passing through the sieve during one minute of hand-sieving is less than 1% of the mass retained on the sieve.

B4 Check of validity of test

After completion of sieving the sum of the masses of all of the fractions should be within ±.0.5% of the total mass.

B5 Expression of results

B5.1 Results are tabulated or more frequently, presented graphically. In the latter case, the abscissa is the sieve-mesh size shown on a logarithmic scale; the ordinate is usually the cumulative mass passing each sieve, the mass being expressed as a % of the total.

B5.2 Round off % values to nearest whole number.

B6 Repeatability and reproducibility

Not assessed.

2. LOS ANGELES TEST

A. GENERAL

A1 Purpose of test

To measure the resistance of an aggregate to impact.

A2 Principle of the test

The test involves submitting the material to impacts from standard steel balls in a Los Angeles machine. The mass of particles smaller than 1.6mm produced in this way is expressed as a ratio of the mass of the sample before the test.

A3 Remarks

A3.1 The test is very widely used throughout the world.

A3.2 There are no appreciable differences between the equipment used in the various countries which is that standardized by ASTM. The only variation noted was in the width of the shelf used in the GDR where 60mm is used instead of 89mm.

A3.3 The mesh-size of the control sieve is unified at 1.6mm except in Bulgaria and USSR where it is 1.25mm. In Switzerland a round-hole sieve with 2mm holes is used as the control sieve; this is equivalent to a 1.6mm square-mesh sieve.

A3.4 The test is carried out in the manner specified by ASTM, except in some countries where the particle-size used and the sample weights differ.

A3.5 No significant variations occur in the number of balls used for the various particle sizes used.

A3.6 Accordingly, it is considered appropriate to recommend adoption of the standardised ASTM procedure. Countries using particle sizes differing from those of ASTM should make every effort to adjust the number of balls used so as to obtain the same results as given by the ASTM test.

B. **RECOMMENDED PROCEDURE**

B1 **Equipment and test methods**

As specified in ASTM standard(s) C 131-76 and C 535-75

B2 **Repeatability and reproducibility**

Not assessed.

3. SAND EQUIVALENT

A. **GENERAL**

A1 **Purpose of test**

To determine the amount of very fine material present in a sand.

A2 **Principle of the test**

A sample of the aggregate, passing a 5mm sieve, is transferred to a cylinder of specified dimensions containing a solution of flocculating agent in water. After shaking under standard conditions and then allowing the contents to settle, the heights of the layers of coarse and fine particles are measured.

A3 **Remarks**

A3.1 The test is in very widespread use.

A3.2 Almost all countries using it follow the procedure standardized by ASTM. However, in Denmark and France the test material is not dried and is of known mass rather than volume. In addition, in France the depth of sand is measured either visually or by means of an ASTM piston.

A3.3 It is accordingly considered appropriate to recommend the standardized ASTM procedure.

B. **RECOMMENDED PROCEDURE**

B1 **Equipment and test method** as laid down in ASTM Standard D 2419. Although the Standard permits the use of either manual or mechanical shaking, the mechanical method is recommended.

B2 **Repeatability and reproducibility**

Not assessed.

4. POLISHED-STONE-VALUE

A. **GENERAL**

A1 **Purpose of test**

To assess the resistance of an aggregate to the polishing action of traffic.

A2 **Principle of the test**

The test consists of separating aggregate particles passing a 10mm normal sieve and retained on a 14mm to 10mm flake sorting sieve (ie a special slotted sieve with slots 40mm × 7.2mm). Test specimens are very carefully prepared from 35 to 50 particles of the separated aggregate. The specimens are subjected to a polishing action under a rubber tyre, water and 2 grades of emery (corn and flour). On completion of the polishing cycle the condition of the surface of the specimen is determined using a friction tester.

A3 **Remarks**

A3.1 The test was developed in the UK and is the only one enabling an assessment to be made of the loss of micro-roughness produced on aggregate due to the effect of traffic. It is in use by most countries without modification. Some, however, have introduced small changes relating to

(a) the reference aggregate used for calibrating the machine,

(b) abrasive powder or

(c) the area and plane of rotation of the tyre so as to obtain a more uniform polishing action. A similar test is used by the GDR but a different procedure is followed.

A3.2 Having examined these various modifications it is considered best to recommend the original British procedure since it is the only one for which repeatability and reproducibility values are available.

B. **RECOMMENDED PROCEDURE**

B1 It is recommended that the equipment and test procedure described in Standard BS 812·Part 3: 1975 Clause 10 be used. It is permissible, however, to use locally available chippings for the calibration of the machine, provided the chippings are of uniform properties, the polished stone value is known and is close to that of the standard rock specified in BS 812.

B2 According to that Standard the repeatability and reproducibility of the test are 4.9 and 6.0 respectively.

5. QUANTITY OF MATERIAL PASSING A 0.075mm SIEVE

A. **GENERAL**

A1 **Purpose of test**

To assess the quantity of fine material (clay, mud/silt and dust) which passes a 0.075mm sieve. No information is provided by the test on the degree to which the fine material is harmful.

A2 **Principle of the test**

Wet sieving is used to determine the mass of fine material passing a 0.075 sieve.

A3 Remarks

A3.1 There are two procedures that are generally used to determine the quantity of fine material in an aggregate. The first employs wet-sieving to measure the mass of material passing through a reference sieve; this is the method most commonly used. The second procedure is based upon a sedimentation test.

A3.2 For the first test, a 0.075mm (ASTM No 200) sieve is the one in most common use. However, a 0.063mm sieve is used in the FGR and Switzerland, 0.080mm in Spain and 0.1mm in Hungary.

A3.3 The RILEM Aggregate Committee recommends this method but with a 0.063mm sieve.

A3.4 As wet sieving is often used for the particle-size distribution determination, and a 0.075mm sieve has been recommended for use in that test, accordingly wet sieving through a 0.075mm sieve is also recommended for the test under present consideration.

B. RECOMMENDED PROCEDURE

B1 Equipment

B1.1 Special Equipment
An ISO 0.075mm or an ASTM No 200 sieve, either being protected by a 1 or 2mm sieve.

B1.2 Normal laboratory equipment.

B1.2.1 Balance with a capacity appropriate to the mass M of the test sample (see B2.1) and sensitivity equal to or better than 0.05% of M.

B1.2.2 An oven thermostatically controlled at a temperature of 110 ± 5°C

B2 Preparation of test sample

B2.1 Minimum mass (in grams)
For D_{max}<20mm, $M > 200D_{max}$
For D_{max}>20mm, $M > 600D_{max}$
where D_{max} is the maximum aggregate-dimension in mm.

B2.2 Dry in oven until mass reaches constant value (M_1).

B2.3 Immerse in water until clay lumps are completely disintegrated. It may sometimes be necessary to boil for 10 min.

B3 Test procedure

Wash through the 0.075mm sieve until the water coming through is clear. Then dry the material retained by the sieve until its mass reaches a constant value (M_2).

B4 Expression of results

The %, m, by mass passing through the 0.075mm sieve is given by $m = 100(M_1 - M_2)/M_1$ where M_1 is the mass of sample before, and M_2 is the mass retained after washing through the sieve.

The result should be rounded to the nearest 0.1.

B5 Repeatability and reproducibility

Not assessed.

6. DENSITY MEASUREMENTS

A. GENERAL

There are three densities to consider.

A1 Relative Density

This is defined as the ratio of the mass of the solid matter of the particles to its volume (*excluding* the volume of pores/voids within the particles.)

Generally, measurements are made on ground material, the only variation in technique found being in respect of the fineness of grinding which ranges from <0.063 to <0.2mm. It is considered that the ISO 0.075mm sieve or ASTM No 200 (0.074mm) should be recommended for use.

A2 Apparent relative density

This is defined as the ratio of the mass of solid matter to the volume of the particles (including the volume of pores/voids within the particles).

Most countries carry out the test on aggregate which has been saturated and then the surface is dried, different procedures being used according to particle size.

It has been decided to recommend the ASTM standardized procedure and to carry out the test on saturated aggregate, the surface of which has been dried. In the case of materials which pass through a 5mm (ASTM No 4) sieve a pycnometer should be used, and if the material is such that it is wholly retained by the 5mm sieve, a weighing measurement with a hydrostatic balance should be carried out.

A3 Bulk density

This is defined as the ratio of the mass of the solid matter to the overall volume of the particles (including the volume of pores/voids *within*, and that of voids *between*, particles).

Some countries use loose, uncompacted material for the test while others use compacted material. The only variations occuring in experimental technique are in respect of the method of compaction — blows of a steel tamping rod, a vibration table, a dropping table etc.

It is considered appropriate to recommend that the test be done on either loose, uncompacted material or on compacted material; in the latter case provision is made for various compaction procedures to be used, as selected by the user.

B1 Relative density

B1.1 Definition
Relative density is the ratio of the mass of the solid matter to its volume. It is determined from measurements on material which has been finely ground (<0.075mm).

B1.2 Special equipment
B1.2.2.1 Grinding mill or mortar
B1.2.2.2 A 0.075mm sieve
B1.2.2.3 Pycnometer having a capacity between 100 and 500 cm^3.
B1.2.2.4 Water bath thermostatically controlled at temperatures between 10 and 50°C; sensitivity ± 0.1°C.
B1.2.2.5 Balance with weighing capacity appropriate to the mass of the pycnometer when filled with water and to that of the test material. Sensitivity to be better than 0.1% of mass of test material.

B1.3 Preparation of sample for test
3.1 The minimum mass to be used for the sample depends on the uniformity of the mineral composition of the aggregate particles and on their maximum size. If the particles are homogenous, the mass of sample selected may lie between 50 and 200g. If, on the other hand, the material is heterogenous (eg alluvial materials), a larger amount should be used, its mass being also adjusted to take account of the maximum diameter of the particles. To give an idea of orders of magnitude:
for D_{max} >20mm, M = 2000g and for D_{max}<5mm, M = 200g.

3.2 Grind the material until all of it passes through a 0.075mm sieve. From the powdered material select a representative test-specimen of at least 50 g. Dry until its mass reaches a constant value M.

B1.4 Test Procedure
4.1 Place the test material in the pycnometer, cover with distilled water and expel the air either by boiling or by reducing the pressure to <4000 Pa for at least 20 min. If necessary cool, before placing in the thermostatically controlled bath at temperature T, then fill the pyconometer with de-aerated distilled water and weigh; let this mass be M_1.

4.2 Empty the pycnometer and fill with distilled water at temperature T and weigh; let the mass be M_2.

B1.5 Presentation of results
The relative density γr at temperature T is given by $\gamma r = M/(M + M_1 - M_2)$ kg/dm^3 where M is the mass of the dry specimen, M_1 is

the mass of pycnometer when containing the specimen and filled with water and M_2 is the mass of the pycnometer when filled with water alone. All masses are in kg.
Round the result to nearest second decimal place.

B1.6 Repeatability and reproducibility
Not assessed.

B2 Apparent relative density and proportion of absorbed water

B2.1 Definition

The apparent relative density is the ratio of the mass of solid matter to the volume of the particles (including pores/voids within the particles).

The proportion of absorbed water is the ratio of the mass of absorbed water to the dry mass of the aggregate.

B2.2 Note.

Measurement of the volume of aggregate and absorbed water is usually carried out on saturated surface-dry material. Volume measurement may also be done on dry material, in which case the volume of pores that can be reached by water is neglected; this method can be applied to low porosity aggregates.

B2.3 Equipment and test procedure

B2.3.1 When the material is such that *none* of it passes through a 5mm sieve, follow ASTM Standard C 127.

B2.3.2 When the material is such that *all* of it passes through a 5mm sieve, follow ASTM Standard C 128.

B2.3.3 When the material is such that some of it passes through a 5mm sieve and some does not, use the sieve to separate the two fractions.
Then apply procedure 3.1 to the part which does not pass through and 3.2 to the part which does.
The apparent relative density of the material and the proportion of absorbed water are the weighted means of the values given by the two fractions.

B2.4 Repeatability and reproducibility

B2.4.1 Apparent relative density

Repeatability — 0.02, and reproducibility — 0.04 except for certain porous, low-density aggregates (< 2.6g/cm^3) where the values are 0.04 and 0.08 respectively.

B2.4.2 Proportion of absorbed water
Repeatability — 5% and reproducibility — 10% both relative to the value determined.

B3 Determination of bulk density, with or without compaction

B3.1 Definition

Bulk density of a material is the ratio of the mass of the solid matter to the overall volume of the particles (including the volume of pores/voids *within*, and that of voids *between* particles). The test may be done on either compacted or uncompacted materials.

B3.2 Equipment

B3.2.1 Cylindrical container of known volume with dimensions chosen according to the maximum diameter, D_{max}, of the particles as shown in the Table. The wall of the container must be thick enough to withstand the compaction operations without distorting.

B3.2.2 Balance suitable for the mass of the test material and whose sensitivity is equal to or better than 0.1% of that mass.

B3.2.3 Steel tamping rod 16mm diameter, 600mm long and rounded at one end; or a vibrating table; or a dropping table.

B3.2.4 Usual laboratory equipment (oven, desiccator etc).

B3.3 Preparation of sample for test

The test may be done on a sample

(a) which has been dried in an oven at 110 ± 5°C until constant mass is achieved or

(b) which is in a naturally damp condition or

(c) which is in a saturated surface-dry condition.

The condition actually used in the test should be stated in the test report. The saturated surface-dry condition should be obtained by following the procedure adopted in the test for measuring the apparent relative density.

B3.4 Test procedure.

B3.4.1 Fill to one third the container referred to in para 2.1 above, pouring the material from a height of less than 50mm above the rim of the container. Take care to work uniformly around the circumference and avoid segregation.

This layer is left loose or compacted, according to the requirements specified. Compaction may be carried out with the rod described in para 2.3 by dropping it from a height of 50mm on to the surface of the layer for the number of times shown in Table, 1 below. Care must be taken to distribute the impacts uniformly over the surface. Alternatively the layer may be compacted by placing the container on a vibrating table for a given time or on a dropping table to which a given number of impulses is applied. The test report should give details of the compaction procedure used in the case of the last two methods, including frequency of oscillation of the vibrating table and distance of drop of the platform of the dropping table.

B3.4.2 Repeat the above operations for the second layer, then for the last layer, filling until there is a surcharge of material. Finally, level the surface with the rod, removing particles proud of the rim and filling small holes with smaller particles.

Table 1

Particle size D max (mm)	Container Internal dia (mm)	Container Height (mm)	Number of blows per layer
50	350	300	100
25	250	300	50
15	200	225	30
5	150	150	20

B3.4.3 Weigh the material

B3.4.4 Repeat the above procedures 4.1, 4.2 and 4.3 twice more. Average the results.

B3.5 Expression of results

The bulk density of the material is given by $\gamma = M/V$ kg/dm³ where M is mass of aggregate in kg (mean of 3 measurements) and V is the volume of the container in dm³.

Round the results to the nearest 10g/dm³.

B3.6 Repeatability and reproducibility

10 and 20g/dm³ respectively.

7. AGGREGATE SHAPE

A. GENERAL

A1 There are three quantities that may be used to describe the shape of a particle component of aggregate:

Length L, defined as the greatest distance between two parallel planes, each tangential to the particle.

Thickness, E, defined as the width of the narrowest slot through which the particle can pass.

Width, G, defined as the smallest square mesh through which the particle can pass.

Hence a shape coefficient L/E, an elongation coefficient L/G and a flakiness coefficient G/E may be defined.

A2 Some countries use the shape coefficient L/E as a basis for specifying shape (occasionally it is called the overall shape coefficient). They consider a particle acceptable if its shape coefficient does not exceed 5, 3 or 2.5, depending on country. Other countries use the flakiness coefficient G/E as a basis for acceptance, provided the coefficient does not exceed either 1.58 or 1.87 depending upon the country. Very few countries take the elongation L/G into consideration for acceptance.

A3 Accordingly, it is considered appropriate to take a particle as being "non-cubic" if its shape coefficient (L/E) exceeds 3 or "flaky" if its flakiness coefficient is greater than 1.58. Specifications should lay down the percentage(s) of non-cubic and flaky particles that is (are) acceptable; the percentages will vary from country to country according to usage.

A4 L, G, and E are usually determined directly by measuring individual particles using calipers or gauges or indirectly by sieving through slotted screens or sheets.

A5 Given this situation, it is considered appropriate to recommend two test procedures based on the shape coefficient, measured with slide calipers, and the flakiness coefficient determined by sieving through slotted sieves.

I SHAPE INDEX BASED ON THE SHAPE COEFFICIENT USING CALIPERS

A. GENERAL

A1 Principle of the test

Measurements of L and E are made, using calipers, on an adequate number of representative particles taken from the sample of aggregate. The percentage by mass of particles having a shape coefficient, L/E, greater than 3 is determined.

B. RECOMMENDED PROCEDURE

B1 Special equipment

B1.1 Calipers with two sets of jaws, the separation of one set being maintained (automatically) at one third that of the other.

B2 Normal laboratory equipment

B2.1 Balance with a capacity appropriate to the mass, M, of the test material and sensitivity equal to or better than 0.1% of M.

B2.2 An oven thermostatically controlled at a temperature of 110 ±5°C.

B3 Preparation of test sample

B3.1 Minimum mass, M, in grams.
For D_{max}<20mm, $M>200\ D_{max}$
For D_{max}>20mm, $M>600D_{max}$.
D_{max} is the size of the largest particle in mm.

B3.2 The sample is passed through a 4mm sieve and the fraction passing the sieve is discarded.

B4 Test method

B4.1 From the fraction selected as in B3.2, 100 to 200 individual particles are taken out at random and dried to constant mass (M_o, say).

B4.2 The particles derived as in B4.1, are measured, in turn, using the calipers. The length of any given particle is measured first and then the smallest dimension of the particle is offered to the second set of jaws having a separation of 1/3. The particles that pass through the smaller jaws are collected and weighed: let their mass be M_1

B5 Expression of results

The shape index is given by $100M_1/M_o$ where M_1 is the total mass of particles passing the smaller jaws of the calipers and M_o is the total mass of particles checked.

B6 Repeatability and reproducibility

Not assessed.

II FLAKINESS INDEX BASED ON THE FLAKINESS COEFFICIENT

A. GENERAL

A1 Principle of the test

The test consists of two parts

i) split the sample into a number of particle-size fractions, the maximum (D) and minimum (d) size in any fraction being related by D = 1.25 d.

ii) take each fraction and pass it through a sieve with parallel slots of width d/1.58 where d is the minimum size of the fraction concerned.

The flakiness index for any of the particle-size fraction is the percentage of the fraction by mass passing through the corresponding slotted sieve.

The overall index for the sample is the weighted mean of the indices for the individual particle-size fraction making up the sample.

B RECOMMENDED PROCEDURE

B1 Equipment

B1.1 Special equipment

B1.1.1 A series of slotted sieves, each consisting of a number of equidistant parallel cylindrical rods mounted on a square frame. The gaps between the rods are 20, 16, 12.5, 10, 8, 6.3, 5, 4, 3.15 and 2.5mm respectively.

B1.1.2 A series of sieves with the following square meshes: 40, 31.5, 25, 20, 16, 12.5, 10, 8, 6.3, 5 and 4mm.

B2 Normal laboratory equipment

Equipment required for a dry determination of the particle-size distribution.

B3 Preparation of test sample

B3.1 Minimum mass, M, in grams.
For D_{max}<20mm, $M>200D_{max}$
For D_{max}>20mm, $M>600D_{max}$
where D_{max} is size of largest particle in mm.

B3.2 The sample is passed through a 4mm sieve, the material retained on the sieve, dried and weighed. Call this mass M_o.

B4 Test method

B4.1 Dry sieve through the series of sieves, collecting and keeping the various fractions separate. (Each fraction has $D = 1.25d$). Determine the mass Mg of each of these fractions.

B4.2 Hand-sieve each of these fractions through the appropriate slotted screen; that is having the gap given in the Table below. Let M_e be the mass to the nearest gram of the material which passes through the slotted sieve.

Fraction d/D	Slotted sieve in mm
31.5 to 40	20
25 to 31.5	16
20 to 25	12.5
16 to 20	10
12.5 to 16	8
10 to 12.5	6.3
8 to 10	5
6.3 to 8	4
5 to 6.3	3.15
4 to 5	2.5

B5 Expression of results

The flakiness index for each fraction is given by 100 M_e/M_g. The overall flakiness index is given by 100 $\Sigma M_e/(\Sigma M_g)$.

8. SAMPLE REDUCTION TO PROVIDE THE TEST SAMPLE

A. GENERAL

A1 Purpose of this procedure

To select a representative test sample from the complete sample delivered to the laboratory.

A2 Principle used in the procedure

The complete batch is split up into smaller samples either by quartering or by using a special sampling device.

A3 Remarks

A.3.1 All countries carry out the procedure. The complete sample received by the laboratory must itself be representative of the aggregate to be studied. Accordingly, the greatest care must be taken with the selection and sampling operations at the quarry or on site (or at works) to ensure that this is achieved. Relevant rules are given in Clause 5 of British Standard BS 812 Part 1/1975 and in ISO/DIS 4847 Standard.

A.3.2 It is essential that the quantity of aggregate reaching the laboratory is more than that required to carry out the tests planned. This enables tests to be repeated for purposes of confirmation or to carry out different supplementary tests not originally planned for in the investigation.

A.3.3 It is considered appropriate to recommend adoption of the procedure defined in any of the following three Standards: British Standard BS 812, ISO/DIS 4847 or French Standard NF P18 — 553. In the last of these, however, no instructions are given concerning sampling techniques at the quarry or on site (or at works).

B. RECOMMENDED PROCEDURE

B1 Equipment and experimental method

Follow British Standard BS812, Part 1/1975 Clause 5 or ISO/DIS 4847. Or else French Standard NF P18 — 553/1978 para 4 et seq.

B2 Repeatability and reproducibility

Not assessed.

9. SENSITIVITY TO FREEZING

A. GENERAL

A1 Purpose of the test

To assess the resistance of an aggregate to repeated cycles of freezing and thawing.

A2 Principle of the test

The properties, eg Los Angeles and grading, of an aggregate are determined on a sample of the original aggregate and on the sample that has been submitted to a specified number of freezing and thawing cycles; the results are compared.

A3 Remarks

A3.1 In the various countries which use the freezing and thawing test, two basic procedures are followed. One procedure assesses the change in granularity produced in the sample by freezing and thawing cycles. The other measures the change in some mechanical property (Los Angeles, fragmentation, compressive strength etc).

A3.2 Since the second method is more effective at showing up any micro-cracking which may have been produced, it is considered appropriate to recommend this, making reference to the Los Angeles test which is the mechanical test most widely used. The first method, however, has the advantage of simplicity and can be more readily carried out on site (or at the works).

Accordingly, it is recommended that the test sample should be sieved through a 1.6mm sieve before and after the freezing and thawing cycles, and before the Los Angeles test is performed. By this means two measures of the effect of freezing and thawing will be obtained, and the results compared.

B. RECOMMENDED PROCEDURE

B1 Equipment

B1.1 Special and ordinary laboratory equipment as specified in ASTM Standards C 131-76 and C 535-75.

B1.2 Equipment required for determining the particle-size distribution by sieving.

B1.3 Refrigerating cabinet, with or without programmed cycling facilities, having the following performance characteristics: to be capable of cooling to a temperature of -25 ± 5°C in 2 to 3 hours, maintaining it for 3 to 4 hours, then warming up to $+25 \pm 5$°C in 2 to 3 hours and maintaining this temperature for 3 to 5 hours.

B1.4 A desiccator, large enough to hold 5kg of material and whose pressure can be reduced to about 4000 Pa.

B1.5 Three trays for holding the test samples; size 430 × 470 × 50mm approx.

B2 Preparation of sample and test procedure

Paras 5 and 6 of French Standard NF P18-593/1978 should be complied with. In addition particle-size distribution measurements should be carried out on the sample before and after the freezing and thawing cycles, using a 1.6mm sieve.

B3 Expression of Results

B3.1 The sensitivity to freezing of the aggregate subject to the Los Angeles test is given by $G = 100\,(LA_g - LA)/LA$ where LA and LA_g are respectively the Los Angeles ratios of the material in the initial state and after being subjected to freezing.

B3.2 The sensitivity to freezing of the aggregate subjected to sieving is given by $G = 100\,(M_1 - M_2)/M_1$ where M is the mass in grams retained by the 1.6mm sieve, *before* the freezing and thawing cycles and M_2 (grams) is the mass retained by the same sieve *after* cycling.

B3.3 Results are rounded to the nearest whole number.

B4 Repeatability and reproducibility

Not assessed

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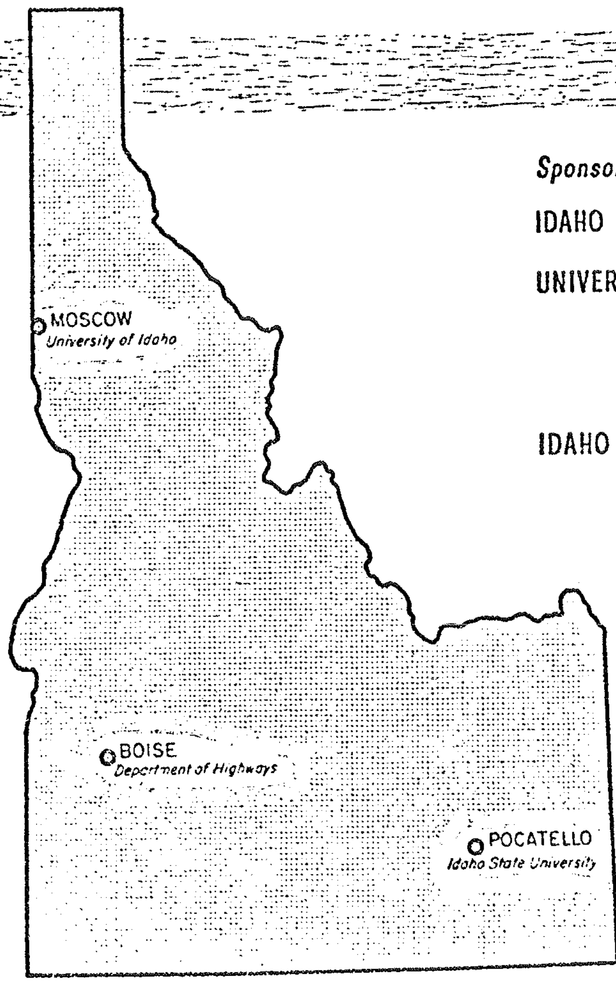
ENGINEERING GEOLOGY AND SOILS ENGINEERING SYMPOSIUM



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* The above listed paper was not delivered to the Symposium Committee for publication.

** The above paper was delivered to the Committee at a date in which it could not be included in the 9th Annual Proceedings. The Symposium Committee agreed to include this report in the volume for the 10th Annual Proceedings.

HAND-FEEL TESTS FOR GUIDING THE COMPACTION OF SOILS

By

John K. McDonald
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ABSTRACT

A classification system is proposed whose groupings follow the patterns of optimum compaction moisture, and which is simple enough so that hand tests may be used in the field to implement it. The groups are: 1) Shot Rock or Boulders; 2) Clean Sand or Gravel; 3) Sand or Gravel with Silt Fines; 4) Sand or Gravel with Clay Fines; 5) Clay; and 6) Silt. Shortened identifying hand tests are described. Rules of thumb for optimum moisture are given. Silts should be much drier than the plastic limit. Clays should be near the plastic limit. In mixed soils the fewer the fines the wetter they should be. Clean granular soils are best compacted sopping wet. The accuracy and worth of routine laboratory tests are questioned and a field procedure for judging compaction is described.

INTRODUCTION

204 Recent trends in soil compaction have been toward end result specifications. This has freed the contractor from some of the inefficient practices of the past but he still has no basic directions to follow in exploiting his new freedom. On the other hand, the engineer has retreated to his laboratory compaction curves and his mysterious black boxes. Communication between the two groups has become limited to whether or not the desired soil densities have been achieved.

This article is an attempt to reach a simpler state of affairs where soils are identified by using hand tests and where optimum compaction moistures are also decided on the basis of hand tests. To do this it is necessary to start with a simplified soil classification system for strictly compaction purposes. To go along with the simplified classifications, up-to-date compactor recommendations are made and a field procedure for estimating compactor progress is introduced.

The three engineering soil classifications used in this country are the Unified, the AASHO and the FAA. Many foreign countries use systems closely related to the Unified system. Figure 1 compares the plasticity diagrams for the three American systems as well as for one under consideration in Great Britain (1). All of the diagrams have vertical divisions. In contrast, Figure 2 shows the relationship between the soil Plastic Limit and the optimum moisture for Standard AASHO compaction for 940 soils. These figures have been assembled from technical articles (2) and unpublished sources. The contours of optimum moisture variation are roughly parallel to the "A" line and have no vertical trends. Figure 2 implies that for Standard AASHO compaction all clays act similarly and that silts can be dealt with in a fairly regular fashion. It is also a basis for using hand tests to judge optimum moisture in the field.

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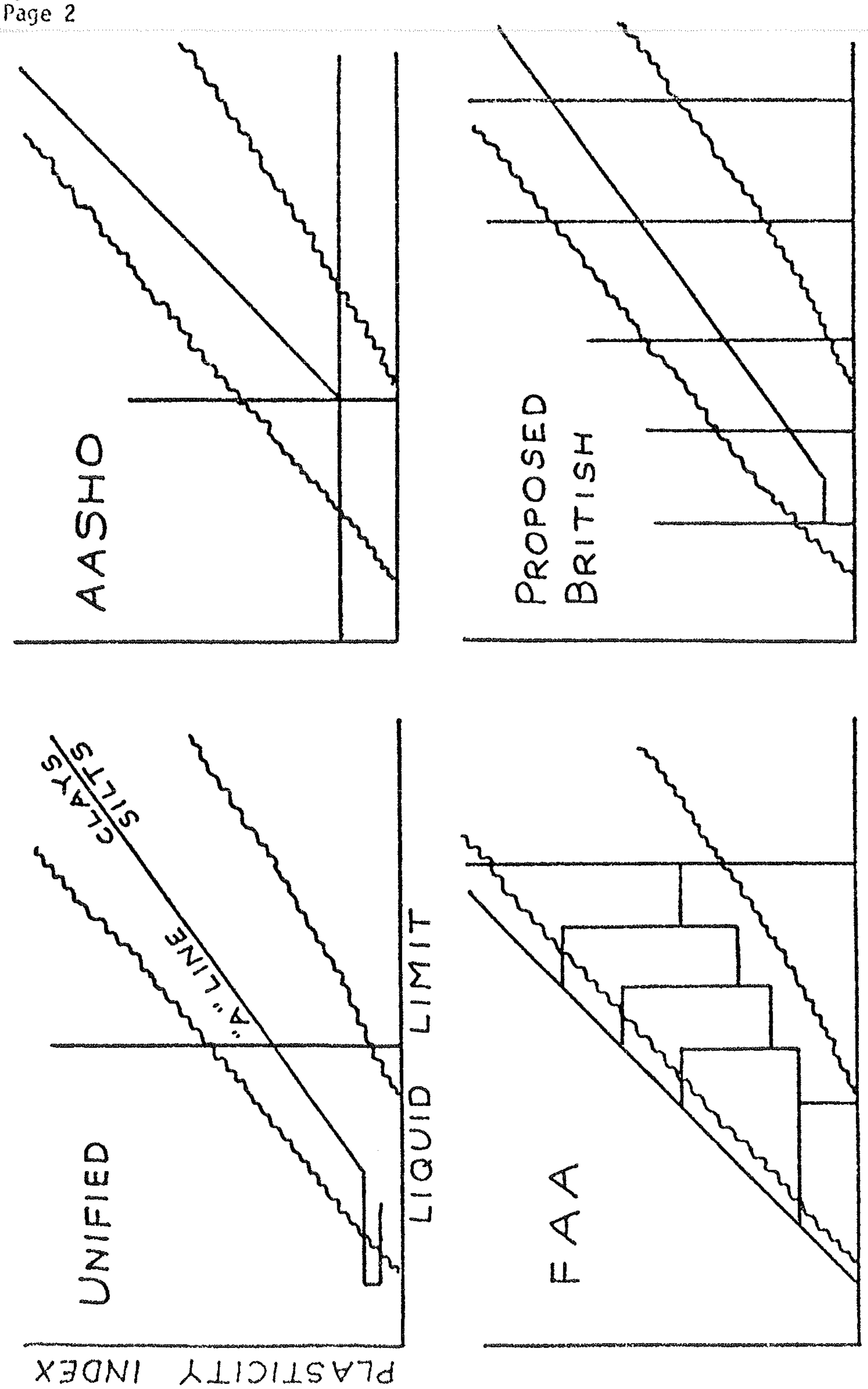

FIGURE 1. Plasticity Diagrams For Engineering Soil Classification Systems Showing Normal Ranges of Soils

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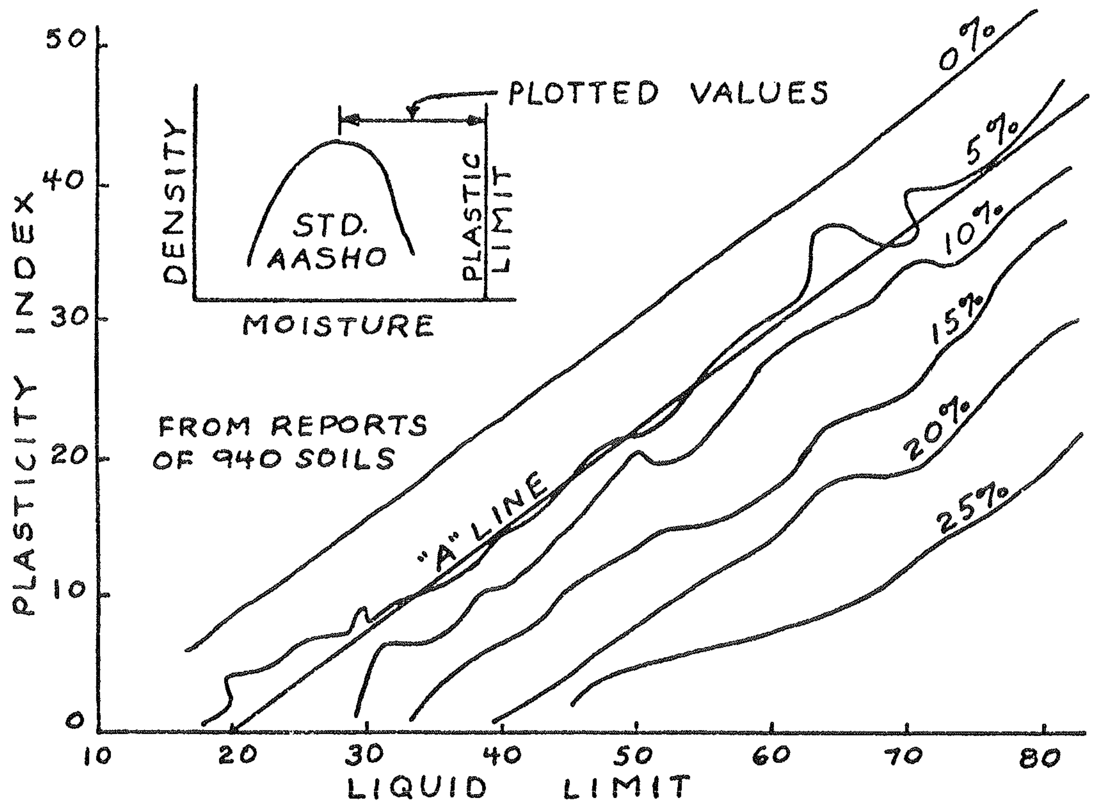

FIGURE 2. Soil P.L. Minus Std. AASHO Opt. Moisture

The Unified system is acknowledged to be the easiest to use in the field with hand tests (3), but it is still far beyond the capabilities of the average dirt foreman. The hand tests need to be shortened and simplified so that beginners can quickly get a feel for different soil types. In this regard, the fewer the soil groups the better.

The Swedish Geotechnical Institute proposed a compaction soil classification in 1968 (4). It had four main groups.

I. Rock fill and granular soils with large stones and boulders.
II. Sand and gravel.
III. Silt, silty soils, clayey sand and clayey gravel.
IV. Clay with low strength and clay with high strength.

To use this system, grain size curves are called for for the first three groups. For the last group the unconfined compressive strength at the compaction moisture content has to be known. Although this system is described in terms of laboratory tests, visual inspection would appear to be almost adequate. This then represents a good first approximation to a compaction soil classification.

PROPOSED SIMPLIFIED SOIL CLASSIFICATION FOR COMPACTION PURPOSES

This system is based on hand tests and on the recognition that soil types gradually blend into one another rather than fitting neatly into pigeonholes. There are no hard and fast limits to the soil groups and in that sense this system differs from the formal ones. The groups were chosen on the basis of either distinctive reactions to hand tests, recognizably different compaction moisture requirements, or a need for a particular type of compactor. The groups are: 1) Shot Rock or Boulders; 2) Clean Sands or Gravels; 3) Clay; 4) Silt; 5) Sands or Gravels with Silt Fines; and 6) Sands or Gravels with Clay Fines. Clues to the field identification of the various soils are described here. 207

Shot Rock or Boulders - Shot rock is jagged material and boulders are rounded. Generally, not enough smaller material is present to fill the "chinks" or void spaces. This material is recognizable at a glance.

Clean Sands or Gravels - Both rounded and angular materials are included. If the soil is dry, spread some out in the hand and look at it closely. Very little if any dust or crusty coatings on the particles should be visible. If the soil is wet, pick up a handful, knead it a few times and then shake it off the hand. No muddy residue should remain. If water is poured onto a clean sand or gravel it will sink in immediately without making any mud. If the soil is all sand, look at it closely to see whether it is all one size like beach sand or whether it is a mixture of large, medium and small-sized sand particles.

HAND TESTS TO SORT SILTS FROM CLAYS (5, 6, 7)

Shaking Test - This is also called the dilatancy test and it is a well-known test. Moisture is worked into a lump of soil until it gets shiny on the surface. Then the lump is alternately tapped and squeezed to see how the shiny surface moisture changes in appearance. The main problem is in deciding how rapidly the changes occur. Figure 3.a. gives some idea of the results that might be expected and shows how trouble in interpreting the results can occur in soils of either very low or very high plasticity. Usually clays give trouble in this test. It is hard to mix water into dry clay soils and they wind up messy and sticky whereas silts are neater and give more satisfying results.

Stiffness Test - This is another well-known test and it is also called the Plastic Thread test and the Toughness test. It is part of the standard Plastic Limit test. The test consists of kneading a lump of damp soil and then rolling it between the palms or on a smooth surface into a thread or worm. The object of the test is to gauge the stiffness of the thread when the soil gets so dry that it begins to crack apart when it is rolled down to 1/8 inch in diameter. Again, the main problem is in deciding how stiff the thread is at the end of the rolling procedure. Stiffness criteria are helpful.

A weak thread requires only a slight hand pressure to thin it down during the rolling procedure. The thread has little strength and will break if attempts are made to pick it up. After a weak thread cracks at 1/8-inch diameter, the remnants cannot be recombined for further kneading.

A medium stiff thread 2 to 3 inches long can be picked up and held horizontally by one end without breaking. The thread remnants can be recombined after cracking apart at 1/8-inch diameter but further kneading will crack and crumble them.

A very stiff thread requires considerable hand pressure toward the end of the rolling procedure. A piece of it several inches long will not droop when held horizontally by one end. The thread remnants can be recombined after the end point and can be kneaded and rolled out again.

In general, silts have an elastic, rubbery feel during the kneading while clays feel dull and dead. Figure 3.b. shows the trends of stiffness test results and again shows that confusion can come about in soils of low plasticity.

Drying Rate - Some idea of the nature of the fines can be gained by observing how long it takes for the soil to pass through the end point of the stiffness test. Figure 3.c. shows that the more plastic soils take longer to dry out. For example, a thread rolled from a soil that dries rapidly might roll down to 1/16 inch in diameter before breaking but the next thread might break when it gets down to 1/4-inch diameter. The soils that dry more slowly allow a better judgment of where the end point actually is. The main use of the drying rate observation is in pinpointing the low plastic soils that can give conflicting results in the other hand tests. It will be shown later that these low plastic soils behave similarly during compaction so it is not necessary to be too precise in trying to sort them into clays or silts.

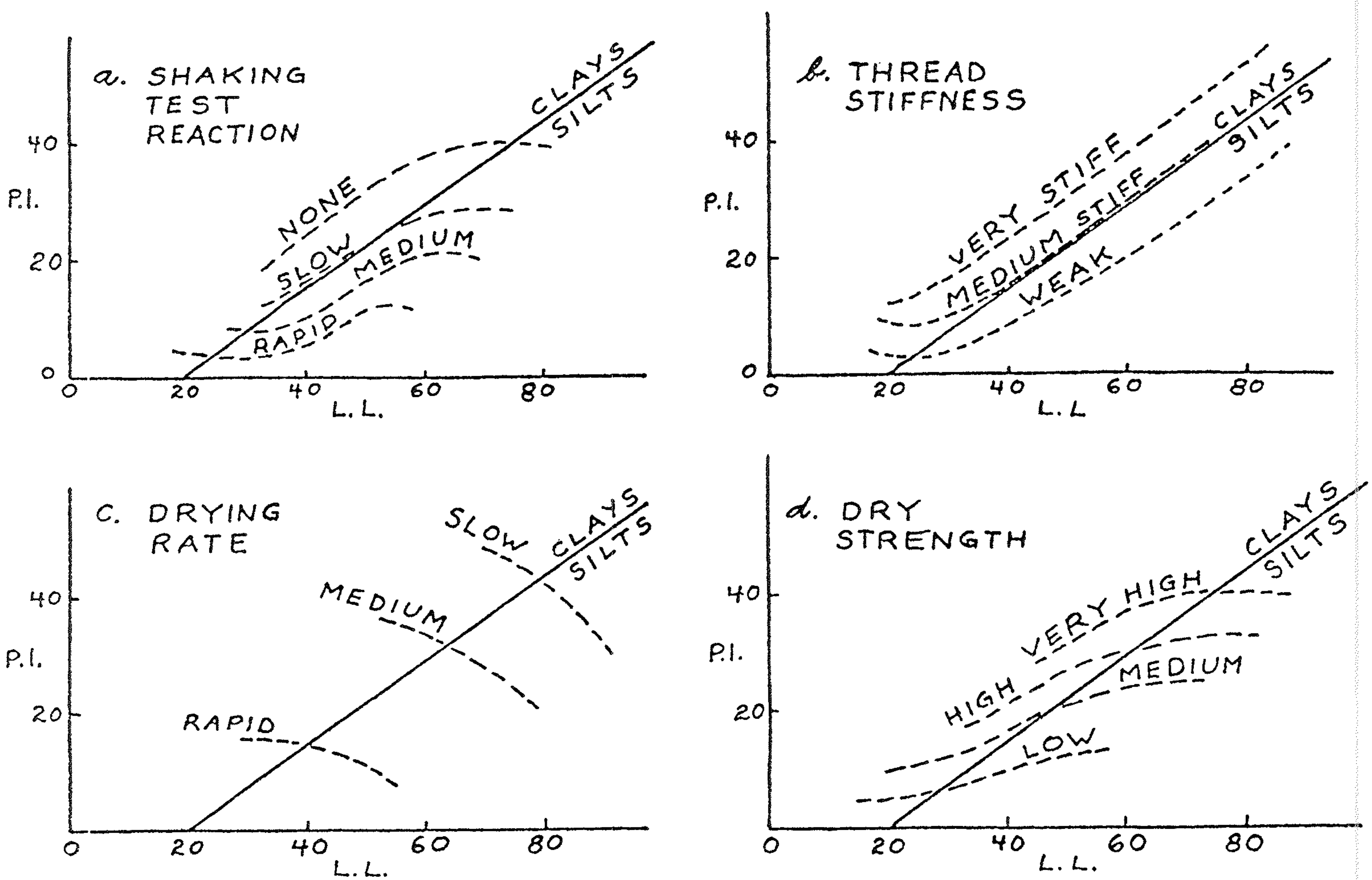

FIGURE 3. Estimated Trends Of Hand Test Reactions

Dry Strength Test - Most instructions for this test call for setting aside a soil lump to dry and then trying to crack it or rub dust off it. However, letting the soil dry and crust on the fingers gives much quicker results. If the dried crusts powder easily and feel slightly gritty, and if the powder can be blown off with a puff of breath, the dry strength is from very low to medium. The dried crusts from soils of very high dry strength will stick to the fingers for hours and simply cannot be rubbed off. Figure 3.d. shows about what can be expected from this test. Silts have low to medium strengths and clays have from medium to very high dry strength.

Hand Washing Test - Water flowing gently from a faucet will rinse silt soils off the hand by itself. Even swishing the hand in a puddle will clean it off. Clay cannot be rinsed off by water alone and the hands must be rubbed together under the water to cleanse them. A greasy or soap-like feeling is characteristic while washing clays off the hands.

TRANSITION FROM MIXED SOILS TO FINE SOILS

When small amounts of fines are present in sands and gravels, the fines tend to work themselves into the zones where the larger particles contact each other. The fines act as cushions or lubricants. After about one-fourth to one-third of a sand or gravel is composed of fines, the coarse particles lose contact with one another and the soil begins to act for compaction purposes as if it were all fines. Estimating the fines content is mostly guesswork but if most of the gravel and coarse sand particles can be worked out of a lump of damp soil that is held in the hand and enough fine material remains to form a ball that can be kneaded and played with, the soil should be treated as if it were all fines.

SANDS OR GRAVELS WITH SILT FINES

Here the sandy or gravelly nature of the soil is obvious to the eye. Pick up a handful of soil. Pour some water on it and knead it to form some mud. Then shake the larger particles of soil off the hand and check the dry strength and hand-washing characteristics of the mud that remains. Low dry strength and easy hand washing will be the clues to the silt fines.

SANDS OR GRAVELS WITH CLAY FINES

This group will be much the same as the preceding group except that the fines will have higher dry strength and will be harder to rub off or wash off the hands.

RULES OF THUMB FOR OPTIMUM COMPACTION MOISTURES

The most widely used compaction test is the Standard AASHO test. The rules to be described are based on optimum moisture for this test. For higher soil densities the rules have to be modified slightly, as explained later. In order to make the following explanations clearer, the plan is to first present the desired field appearances and hand test reactions for clean granular soils, clay soils, and silt soils, and then to show how these would change as the soil mixtures vary from one of these extremes to another.

Clean Granulars - In desert conditions, bone dry sands compact best. Otherwise the best moisture for sand, gravel, shot rock or boulders is sopping wet. Hard work and poor compaction will occur with clean granular soils that are merely damp. Nor is it enough to sprinkle and then come by with the roller a few minutes later. To do any good the water should be put on right in front of the roller. Shot rock or boulders should be sluiced with a hose during compaction to make them as wet as possible.

Clays - Figure 2 shows that clays should be at the Plastic Limit or not more than 5% dry of the Plastic Limit for Standard AASHO compaction. This is a very helpful finding because the Plastic Limit test can easily be done in the field. If a lump of clay is picked up, kneaded into a ball, and rolled into a thread between the palms of the hands it should start crumbling and cracking on the first rolling when it reaches a diameter of 1/4 to 1/8 inch. If it rolls to a tiny thread without breaking, it is too wet. If it is too crusty to allow rolling into a worm or thread, it is too dry. At optimum moisture clays are dark in color. Clay soils lighten in color when they are quite wet of optimum but by then they are greasy and sticky and obiously too wet. 211

Silts - As Figure 2 shows, silts have to be quite dry of the Plastic Limit for Standard AASHO compaction. At optimum moisture then, it should not be possible to begin rolling a thread from a lump of silt. It has been observed that soils become darker in color as they go through optimum moisture. This change in color or tone of the soil can be used as a guide. Silts before compaction should appear dry and light colored in the sense that if a little more water were added the color of the soil would just start to darken. In the field, if silts look dark after the first pass of the roller, the soil is too wet. Experience indicates that most of the trouble in compacting silts comes from treating them like clays by having them dark and damp looking before compaction.

Soil color can be checked by putting a bit of the silt soil into the corner of an envelope in a pocket where it will dry quickly. Then some soil can be picked up from the ground, pinched hard between the thumb and forefinger, and compared with the dry soil from the envelope. The pinched soil should be slightly darker than the dry soil.

If silt soils are too wet they will be springy under the compactor or underfoot. However, some silts are springy by nature because they contain an excess of tiny mica flakes. These mica flakes reflect light and can be seen if the soil is spread out in the hand and examined in the sunshine. Springiness during compaction is generally a warning of unsatisfactory soil performance.

Silt and Clay Mixtures - If a soil is judged from the hand identification tests to be halfway between a clay and a silt, then the moisture to shoot for is halfway between that recommended for a clay and that for a silt. Probably more soils will seem to be borderline varieties than definite clays or silts. Figures 2 and 3 can be used together to make a better moisture estimate. These figures also make it clear that for the low plastic soils with Liquid Limits less than 30 where the hand tests can give confusing results the optimum moistures are at or slightly less than the Plastic Limit. These are the soils that are best identified by considering the relatively short time it takes for them to dry out and reach the end point in the stiffness test.

Mixed Coarse and Fine Soils - Coarse soils should be sopping wet but fine soils have definite moisture requirements. When these soils are mixed together, the moisture content of the fines is the thing to pay attention to. As the fines get fewer they should get wetter. If the fines are clay and there are not enough fines present to readily sort out enough to make a stiffness test thread with, then the clay that is visible in the soil should be almost greasy and sticky. If the entire random soil lump can be kneaded somewhat, there is enough clay present that the moisture rules for pure clay should be followed. Sands or gravels with silt fines cause more trouble in compaction that any other soil group. Experimentation is usually necessary, but in no case should silt fines be drier than they would be if no sand or gravel were present.

OPTIMUM MOISTURES FOR HIGHER SOIL DENSITIES

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Figure 4 shows the reduction in moisture content that is necessary when going from Standard AASHO to Modified AASHO specification. It is better to first estimate the optimum Standard AASHO moisture and then to go a little drier for Modified AASHO than it is to try to set up new rules of thumb for directly estimating the optimum Modified AASHO moisture. For the clean granular soils the rules of bone dry or sopping wet hold regardless of the compaction energy level.

HAND TESTS IN THE FIELD VERSUS LABORATORY COMPACTION TESTS FOR OPTIMUM MOISTURES

Judging optimum moistures from hand tests may seem rather crude but laboratory tests have their problems too. In 1964 the American Council of Independent Laboratories sent identical samples of three types of soil to 99 commercial laboratories for compaction tests (8). The ranges of optimum moistures that resulted were: 6.1% for an ML silt; 7.7% for a CL clay; and 11.8% for a CH clay. More recent results of comparisons from 40 laboratories in England have been reported by the British Road Research Laboratories (9). Their ranges were: 4% for a CL clay; 8% for a CH clay; and 20% for another CH clay. It can be seen that specifications that call for field moistures to be within, for example, 2% of the laboratory optimum value are quite unrealistic.

These comparisons were run with soils that passed the #4 sieve. If a soil contains gravel it has to be screened out and its effect on total density estimated mathematically. No help is available for adjusting moistures. If the soil consists of more than one-third gravel the usual compaction tests are practically useless. To further complicate matters, it is not unusual to find that each scraper load of soil on a project has a different compaction curve.

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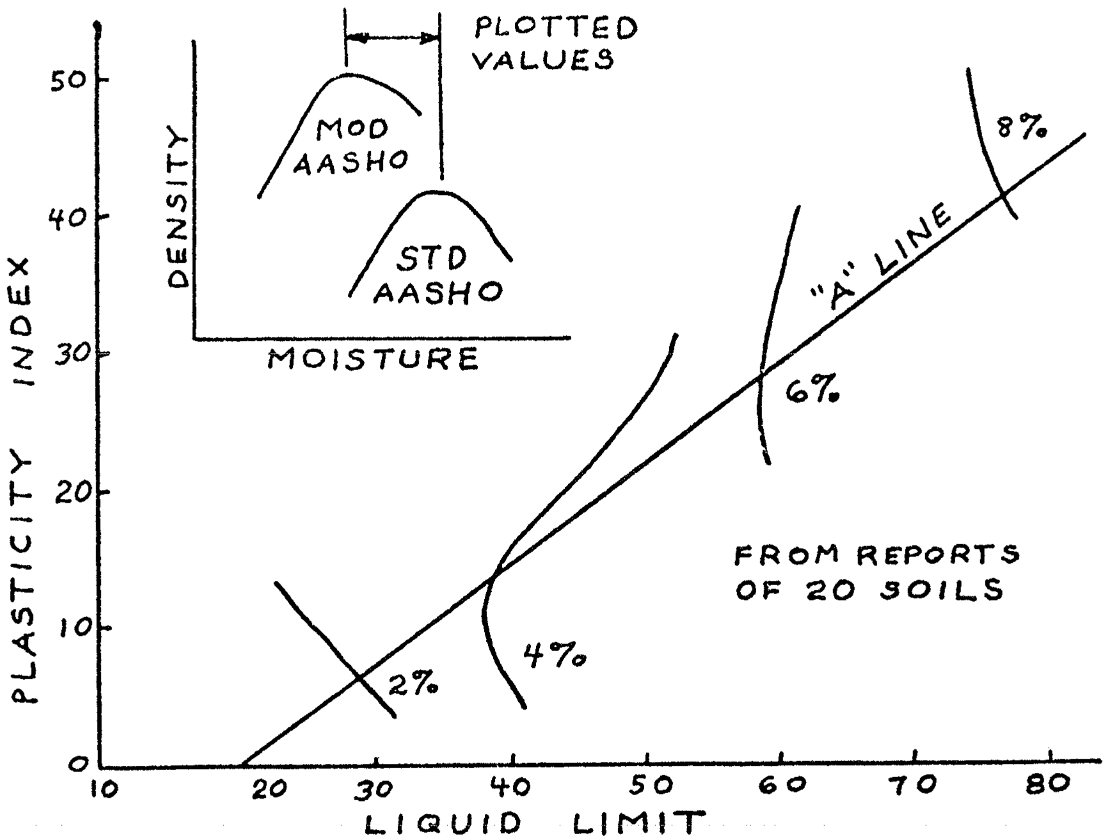

FIGURE 4. Reduction In Moisture For Higher Density

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What all this means is that laboratory compaction tests often have to be interpreted in the light of actual field experience with the soils in question. It also means that a person who practices hand tests and observes the soil during compaction is oftentimes a better judge of the best compaction moisture than a random routine laboratory test is. Laboratory tests still have to be run for density control but their limitations for moisture control should be recognized. The wiggles and ripples in Figure 2 may well be due to errors in compaction tests.

COMPACTOR RECOMMENDATIONS FOR INDIVIDUAL SOIL GROUPS

The following recommendations are based on numerous personal observations as well as several years of collecting field reports and magazine articles on compactor performance.

Clays and Mixtures of Clay and Silt - The large-footed or segmented roller is the automatic choice here because of the few passes it requires and the high speed at which it can operate. With loose soil lifts of 8 inches or less, and proper moisture, Standard AASHO densities are usually achieved in four single-drum passes. For Modified AASHO requirements, soil lifts should be as thin as possible.

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The sheepsfoot has been the traditional compactor for clays but Standard AASHO density is about all that can be expected without very great effort. For higher densities the traditional approach has been to use the heavy pneumatic. The pneumatic has low productivity and for clays on the wet side of optimum it tends to produce laminations or pancakes all through the lift of soil. The tires of heavy earth carriers produce this same effect particularly where they use part of the embankment as a haul road.

Dozer blades on compactors should be used only when the clays are dry of optimum and then only on the first pass. Clays respond to compressive stresses, not sliding stresses.

Silts - Many reports have indicated the difficulties in compacting silt soils. Sheepsfoot rollers are often tried and they often fail to get the job done because the soil rarely gets strong enough to withstand the high-foot pressures and the consequently high sliding stresses exerted by the sheepsfoot. The pneumatic is usually the next machine tried and it has been more successful. The smooth drum vibrating roller has also been used to advantage.

The way to compact silt soils is with machines that furnish low sliding stresses but adequate compressive stresses. The large-footed or segmented rollers, particularly those with flat feet, have these characteristics. If the roller has pointed or chisel-shaped feet the roller should be very light in weight. In this soil the thinner the lift the better. Dozer blades should not be used after compaction has commenced.

Sands or Gravels with Silt Fines - These soils give more trouble in compaction that any other group. Trials of different moisture contents and different compactors are reported frequently. Large-footed rollers generally do well. Smooth drum vibrating rollers and footed vibrators are also popular with these soils.

Sands or Gravels with Clay Fines - This group is very easy to compact and oftentimes the haul units alone are sufficient. In these cases, thin soil lifts and careful routing of traffic are mandatory to insure uniformity of compaction. The large-footed or segmented roller is recommended where higher densities are required but for ordinary densities a wide variety of compactors have been used successfully.

Clean Sands and Gravels - This is where the smooth drum vibrating roller comes into its own. In using this roller on sands, it should be kept in mind that the upper part of the sand lift does not get compacted to any extent. If lifts are too thin, much of the compactive effort is wasted in repeated over-vibration of the surface material. A good compromise between lift thickness and required number of passes that leads to high production of compacted soil is when the lift thickness is slightly less than the depth at which maximum density would occur in a single very thick lift. This means that for large heavy vibrators the sand lifts should be around two feet thick.

The Grid-roller works well on sands and gravels and can handle lifts of one foot of sand. When pneumatic or smooth steel wheel rollers are used on poorly graded sands the soil lifts should be very thin and the roller should start out as lightly loaded as possible. The use of the compactor dozer blade is recommended on the first couple of passes. 215

Shot Rock or Boulders - Rearrangement and breakdown of the rocks is the main avenue to compaction of these materials. Precompaction blading and dozing are very important and the compactor should have a crushing ability. When the rocks are two feet in size or more the most effective compactor is the heavy smooth drum vibrating roller. For smaller sized pieces the large-footed roller or the Grid-roller are recommended. The Grid-roller pulverizes the top surface of the rocks and makes a smooth surface that reduces wear in the rubber tires of the haul units. The large-footed roller has good crushing action and in addition it can easily handle any pockets of finer soils that crop up in the fill.

FIELD ESTIMATION OF COMPACTION PROGRESS

While hand tests can help judge optimum moistures there are no hand tests that can tell when a certain density has been reached. Therefore, laboratory compaction tests and field density are indispensable. However, as far as the contractor is concerned, it would be very helpful to know when a compactor has done all that it is going to do. In addition, compaction is often done when all parties are aware that the soil is far above optimum in moisture content. Here the compaction curves go out the window and the engineer merely wants all the density he can get without turning the soil into mud with excessive efforts at compaction.

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It has been found that as compactor drums roll along in soft soil they rotate either more or less than would seem necessary to move forward a given distance. In other words, towed rollers tend to be dragged along and driven rollers slip as they move forward. This dragging and slipping takes place within the upper part of the soil rather than between the drum and the soil surface. Vibrating rollers jump into the air and turn slightly before striking the soil surface again. On succeeding passes the soil is tighter and there is less of this slipping or dragging. When the compactor has done all the useful work it is going to do, the amount of slipping or dragging stabilizes.

To use this concept, the distance between footprints is measured on successive passes and when the measurements are equal for two or three passes the roller has done all the compacting it is going to do. This concept is most useful with towed machines and those where one axle does all the driving. All-wheel-drive compactors do not do much slipping or dragging except in the first pass or so where the front wheels or drums have more rolling resistance than the rear drums. In any event, this simple measurement is worth trying and it can be quite informative. The main thing it tells is when it is a waste of time to run the compactor any more. When soils are uniform over a large area more useful information can be gained by running the compactor over a section that has passed the density test, measuring its rolling circumference there and using this measurement as a guide.

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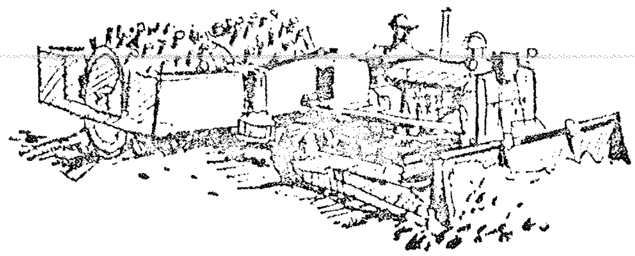
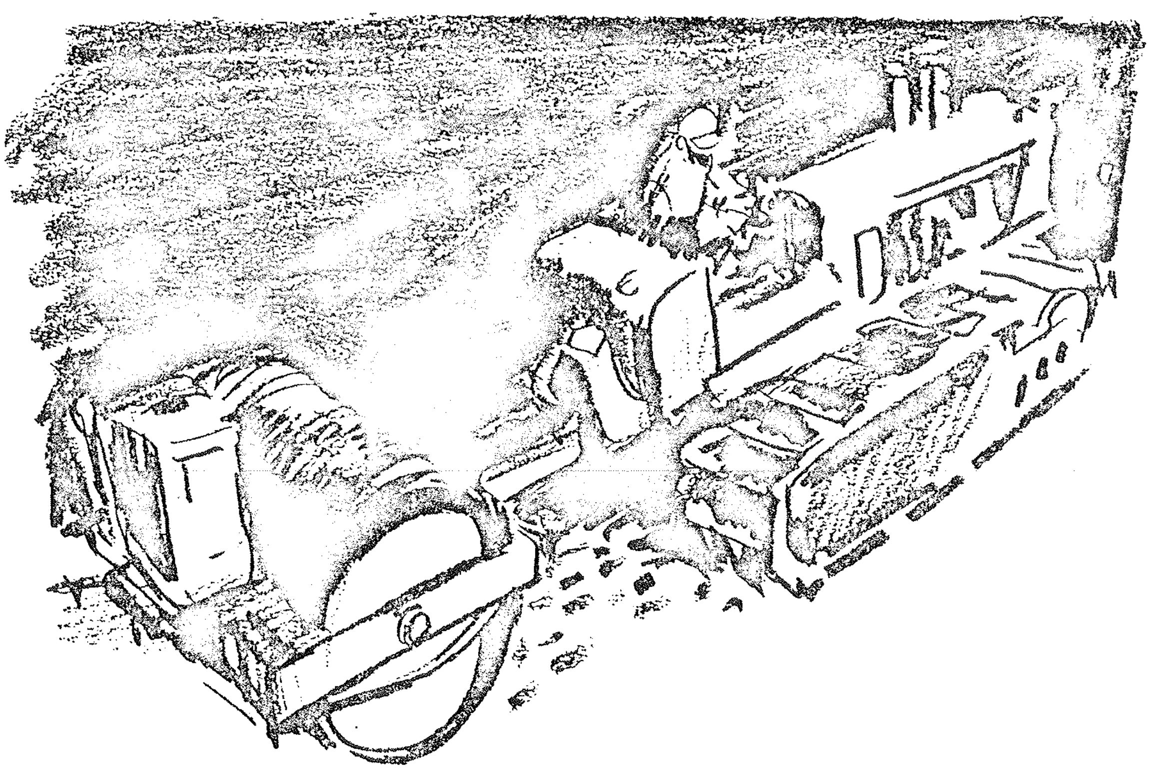
218 A self-propelled rubber-tired roller is used in construction work (Bolivia).



by M. D. MORRIS

EARTH COMPACTION



A REPRINT FROM

Construction Methods

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3. COMPACTION EQUIPMENT

IT IS UNREALISTIC to attempt to compact an embankment properly without adequate compaction equipment. But there are many dreamers who hold that special compaction machinery would not be necessary if the fill-hauling vehicles were to follow a proper pattern across the embankment.

"This is a ridiculous piece of thinking," says Nomer Gray, partner in the consulting engineering firm of Ammann & Whitney, whose current Chantilly Airport job near Washington, D.C., includes a vast amount of compaction. "To begin with, a truck driver couldn't care less. Unless someone constantly stands over him, he's naturally going to drive in the ruts of the vehicle before him. Instruction to drive 1 ft to the left or right of these tracks is a waste of time and effort and has gotten meager results, in my experience."

In any event, it is not necessary to depend on the action of earth-hauling equipment to achieve proper compaction. Today there are well over 100 different items of commercially available machines or tools especially designed for compacting earth. There is some device built to suit nearly any job condition. And there is a trend toward units that are self-propelled.

Regardless of how it is powered or propelled, compaction equipment does its job in any one of four principal ways, or combinations of them:

1. Static weight
2. Kneading action
3. Vibration
4. Impact

Static-weight compactors are surface rollers of either the smooth-steel-wheel or pneumatic-tired type. The steel units may be two- or three-axle tandems or of the three-wheel configuration. Pneumatic-tired rollers come in a variety of sizes and weights; the differentiating point is the tire size.

Kneading-action compactors are primarily tamping rollers of the sheepsfoot type. But this category also includes grid rollers and steel rollers with segmental-pad drums.

Vibration compactors are vibratory steel or rubber-tired rollers, or vibratory plates or shoes.

Impact compactors are primarily hand-held pneumatic tampers. But some are self-contained gasoline-powered units that jump up and down. Still others resemble drop hammers.

Finally, there are combinations that are neither bird nor mouse and must be classified as bats.

Steel Rollers

Tandem rollers are those that have two or three rolls in line. The rolls are actually steel drums that can be filled with ballast to increase their weight. If a roller is described as "14-20-tons," it means that the minimum dead-load weight of the machine is 14 tons and that the rolls can be ballasted with material such as water or wet sand to give a maximum total weight of 20 tons. It should be kept in mind that although total weights of tandem rollers can be greater than three-wheel rollers, their unit pressures tend to be less because the greater contact surface of the rolls will spread the load over a larger area.

Three-wheel rollers have two rear wheels and a front steering wheel. The narrow rear wheels and the wide front wheel may be either spoked or ballastable. The three-wheel roller is quite maneuverable but tends to leave deep ruts in granular soils due to the concentration of load in the narrow wheels.

Both types have rather slow running speeds and have questionable safety for operation near the edges of high, steep-sided fills.

Steel rollers of the tandem or three-wheel type are effective most generally on soils of a more granular nature where the crushing effect of their static weight can be best employed. However, loose sands may not support the heavier rollers.

A steel roller's compactive effort is lessened in material of granular-plastic or plastic-granular nature. That's because the heavy rollers create crusting at the top of the layer, with diminishing effectiveness down to the lower parts of the lift, even in shallow thicknesses.

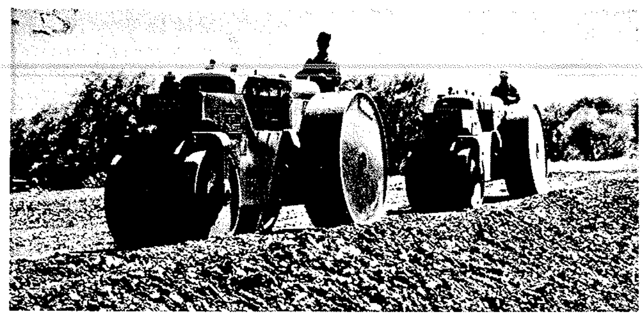
For very plastic material, steel rollers tend to have a bridging effect. This means that the roller will squeeze material from the high spots to the lows, but the material moved will not be compacted. Steel rollers also have a plowing effect. This creates plastic waves ahead of the rolls and also results in a springing up of material behind them.

Steel-wheel rollers can be used effectively to level off high spots after sheepsfoot or pneumatic-tired rollers have done their work.

E. Miller Smith, a compaction expert with S. J. Groves & Sons Co., one of the country's foremost earthmoving contractors, considers steel rollers as "possibly the oldest in design of all classes in use today. To use a common expression, the 'bugs' have been pretty well removed so that they contribute very little to downtime and maintenance costs.

"Very definitely they have their place in some locations. But they are losing favor on the largest of the earthmoving projects on embankment rolling, although they are used in many places for sealing-off the fill surfaces."

Twenty-nine states and the District of Columbia permit steel-wheel rollers for compaction (see table, page 7). Only two



STEEL ROLLER—Tandem or three-wheel type (here) compact granular soil.

states require that three-wheel rollers be used. New York specifies that steel-wheel rollers shall be used for compacting slag, coarse gravel, rock, or layers composed of soil and rock.

Weight requirements for steel-wheel rollers vary from 8 to 10 tons for tandems and from 10 to 14 tons for three-wheel rollers. Required compression produced under the driving wheels runs from 275 to 330 lb per in. of roller width—a 20% variation.

One state requires that the roller combination include a three-wheel roller. This may mean that the department is paying the cost of having idle equipment on the job.

Pneumatic-Tired Rollers

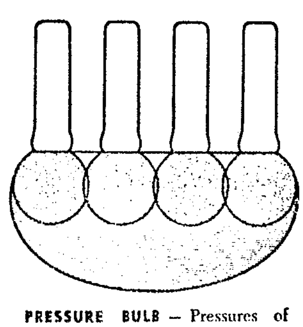
Pneumatic-tired rollers are surface rollers, yet they also apply the principle of kneading action. They are either self propelled or drawn, and are of two types: Those with small tires and those with large ones.

Small-rubber-tired rollers generally have two tandem axles with four to nine 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 chassis of the piece is also a container for solid or liquid ballast. The weights carried may be varied to suit the material being compacted.

The individual wheels may be on knee-action type mountings to avoid omissions of low spots or bridging on highs. Wheels may also be mounted slightly out of line with the axle, giving them a weaving action and the name "wobbly-wheel." This condition improves the kneading action. It gives better routine compaction in all but very plastic materials, whereas the standard small-wheel compactor is suggested for finishing operations.

Pneumatic-tired rollers should not be overloaded with ballast or moved at excessive speeds. Such faulty operation will give more coverage but will result in extra tire and bearing wear, thus increasing maintenance costs.

Small-tire compactors provide the same unit surface pressure as large-tire units, with less over-all weight on the material being compacted. They provide more crushing of lumps, do not push whole masses before them, or cause lateral displacement. They also offer more maneuverability with less motive power. Disadvantage lies in poor flotation in loose materials, slipping of self-propelled units in very wet soil, and about a 6-in. maximum depth of compactive effectiveness.



PRESSURE BULB — Pressures of individual tires inter-react to form larger bulb deeper in the lift.

Large - tire rollers, generally towed, get into the realm of super-compactors or proof rollers. Most are from 15 to 50 tons in weight and are on large-diameter, large-section, rubber-tired wheels. Some heavy airport units having five or six wheels go up to 200 tons.

Large-rubber-tire rollers will work on all types of soils. They cover a bigger unit-pressure area and have a deeper effect on soil movement (due to less lateral support, percentagewise) than do small-tire units. Large-tire compactors can handle higher lifts and get deeper penetration of compressive force. But there is a definite variation, from the surface down, in resulting density. The expense is in their operation since they require the right type tractor to pull them, and they must make a greater number of passes to get complete coverage of the spaces between the wheels. Their best use is in test or "proof" rolling.

When considering a large-tire roller for general compaction work, the contractor should check the economics of getting the same unit loadings (in psi) with small-tire units used in more passes on shallower lifts. Advisable also is the consideration of gaining maximum results by using pneumatic units in concert with steel units where the soil types warrant.

There are at least four ways used to express the compacting ability of pneumatic rollers. They are: (1) gross weight of vehicle; (2) wheel or tire load; (3) weight per inch of tire width; and (4) tire inflation pressure. The problem is complex because rubber, unlike the steel roller, is flexible. And low tire air pressure allows an oval surface contact area to

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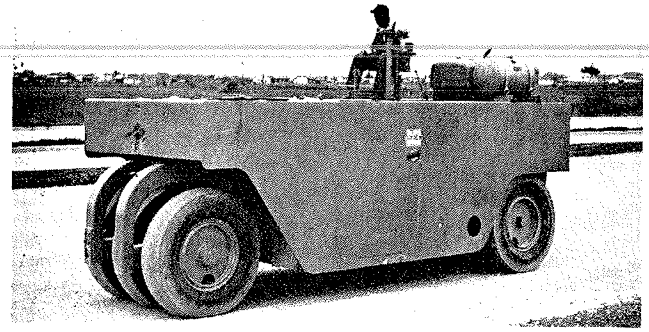


PNEUMATIC-TIRED ROLLER—Trend with these units is self-propulsion.

enlarge. This diminishes the effect of total load by giving larger weight distribution and, consequently, lower unit ground pressure.

Accordingly, gross-weight ratings mean nothing unless the number of wheels, tire size, and inflation pressures are known. Yet five states rate pneumatic rollers on the basis of gross weight only (CM&E, April, p. 226), with weight required ranging from 5 to 30 tons.

Seven states specify minimum or ranges of wheel loads that vary from 1,000 to 6,000 lb. But wheel loads do not express compacting ability unless the tire size and inflation pressure are also given. Wheel loads within a range of 1,000 to 2,500 lb, even when used with the smallest compactor tire now available, will not produce the compactive effort needed in modern highway construction.

Seventeen states rate the required pneumatic rollers by "weight per inch of tire width." Yet the ranges specified vary from a low of 43-300 lb to a high of 400-600 lb.

While the "weight per inch of tire width" applied to steel-wheel rollers gives a representative measure of compacting ability, the same criterion loses much of its meaning when applied to pneumatic tires. In the case of the steel-wheel rollers, a uniform rectangular ground contact pattern exists under most conditions. On the other hand, the oval contact pattern produced by a tire changes as wheel loads and inflation pressures vary. In general, the tire contact length changes to a greater degree than the tire contact width. If the weights per inch of tire width are converted into wheel loads at various inflation pressures, a sizeable range of contact pressures results.

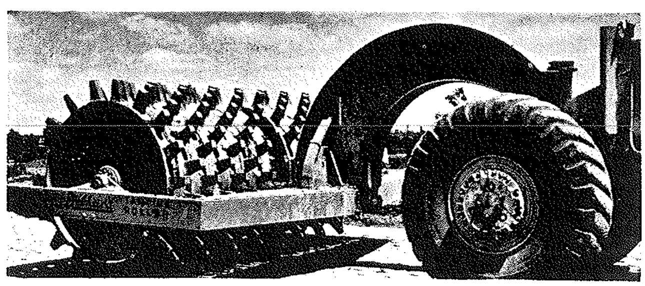
Tire sizes also have an influence. For example, a requirement of 600 lb per inch of tire width could be converted into a contact pressure of 62.0 psi for one tire size and a contact pressure of 81.6 psi for another. This means a differential of more than 30% in compacting effort.

Only three highway departments' standard specifications have no provisions which allow use of pneumatic rollers for compacted embankment work.

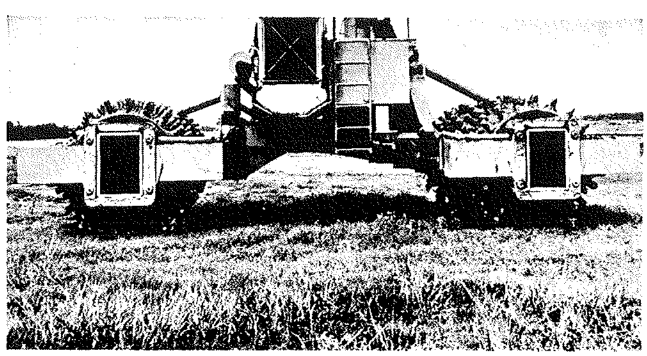
Several states specify the tire inflation pressure. Again, this is of no significance without tire size and wheel load. Contrary to popular belief, inflation pressure and contact pressure are not necessarily synonymous.

Sheepsfoot Rollers

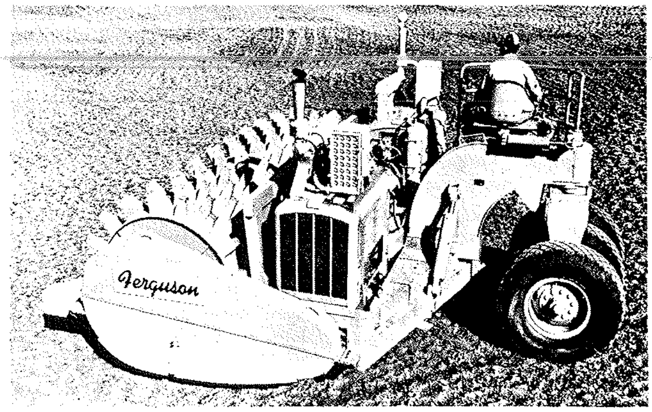
For cohesive materials (clays and silty clays) the proper compaction equipment is the sheepsfoot roller: This is the tamping unit that produces kneading action in the soil. Sheepsfoot rollers range from about 2 to 20 tons in weight. The average unit is about 6 ft wide. The drum, close to 5



SHEEPSFOOT ROLLER—Steel feet "walk out" when lift is compacted.



SELF-PROPELLED SHEEPSFOOT—Outsize unit covers large area fast.



TRICYCLE—Separate engine drives each drum of this unit.

ft in dia, is hollow and ballastable to increase the load. The rollers can be towed in pairs, or four-block pairs, or other arrangements. Feet, about 7 to 12 in. in length, have various shapes: round, pie-segment, diamond, or elliptical. There has been no research on which is best, but the compacting surface should be 5 to 10 sq in. in order to meet various state specifications for contact pressures.

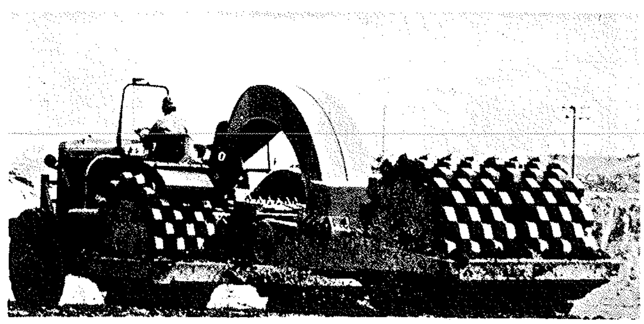
The sheepsfoot roller usually can handle loose lifts up to 10 in. The theory is that the feet will compact the lower layers in successive passes until, when completely packed, the soil will yield no further and cause the roller to "walk out." Because the feet penetrate the lift and compact within it, they affect the soil particles in all directions. It is not necessary that the roller drum touch the surface, since the total load is transmitted to the soil by the feet in small areas at high concentrations.

Disadvantage here is the comparatively shallow depth to which the feet can effect compaction. Also, they have no effect in granular materials. The Corps of Engineers, for its large scale work, has developed some huge machines with larger feet. But they apparently increased the motive power required without any significant increase in effective compaction depth.

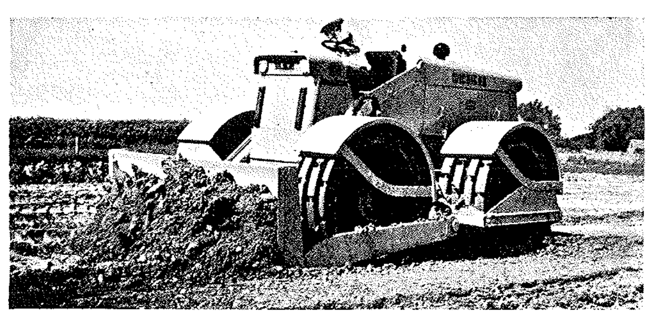
In some cases sheepsfoot rollers will bridge over the soil at the outset, but this bridging stops after several passes. They do expose more soil surface to the air for evaporation of moisture, causing crusting. Their penetrating feet pulverize lumps in the soil. And they work well by causing lateral particle movement beneath the surface, thus blending coarse and fines more thoroughly. On the other hand, sheepsfoot rollers should not be used in graded aggregate or stone bases, since there they will cause segregation.

Twenty-nine states have specific requirements for sheepsfoot or tamping rollers: They spell out the general type, area of tamping feet, and/or psi of contact area needed, (see table, page 16). Another six states permit tamping rollers if approved by the engineer. In one state tamping rollers are permitted as a contractor's choice, and another twelve states are also on an end-result requirement. Seven states require dual-drum rollers. Five states require self-cleaning features for tamping rollers.

Thirty-three awarding agencies have ground-pressure requirements expressed in minimum psi



SHEEPSFOOT VARIATION – Sloped-pad tamping feet do the compacting here.



TRACTOR COMPACTOR—The segmented steel rolls replace rubber tires.

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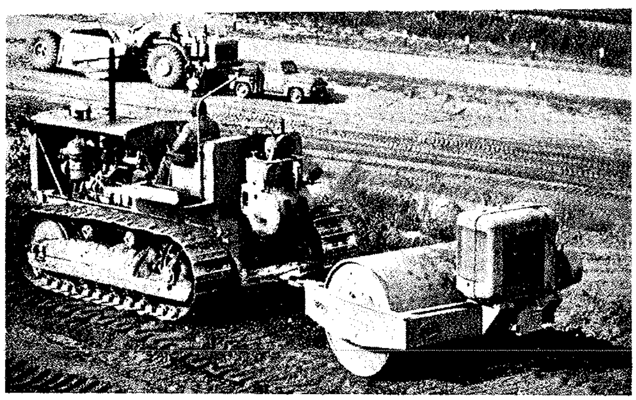
or minimum weight per foot when the entire weight of the roller is supported by one row of feet. The range of minimum contact pressures on the tamping feet varies from 100 to 500 psi. But 16 states call for a minimum 200 psi. Where the maximum contact pressure is specified, the range is from 175 to 550 psi.

There is a trend toward self propulsion of sheepsfoot rollers. Both R. G. LeTourneau Inc. (Longview, Tex.) and Ferguson (Shovel Supply Co., Dallas) have built large four-drum units for heavy compaction work. The latter also has produced self-propelled two-drum rollers of various configurations.

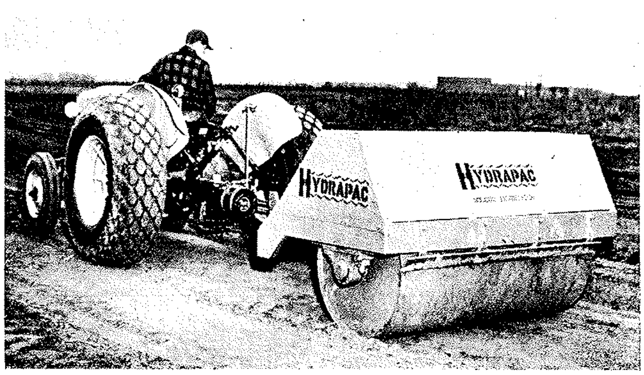
A somewhat different variation of the sheepsfoot roller is made by Hyster Co. (Peoria, Ill.). They replace the rear wheels of a Cat DW20 prime mover with steel-drum wheels fitted with sloped-pad tamping feet. Also, hooked to the unit by a gooseneck are two trailing compaction drums.

Another Hyster variation is the "grid" roller. Here the DW20's rear wheels are replaced by wheels whose perimeter is made of an open mesh of 1½-in.-wide steel bars on a 5-in. square spacing. Similar grid drums can be hooked behind the DW20 or towed separately. Eskridge Eqpt. Co. (Tulsa) makes a somewhat similar towed "grate" roller. The grid roller pulverizes subsurface lumps and works well in gravel or rocky fill.

Buffalo-Springfield Co. (Springfield, Ohio) manufactures a 16-ton self-propelled segmented-wheel roller that combines the features of sheepsfoot and steel wheel rollers. They also make a segmented roll that can be substituted for the guide roll of a regular steel roller. And Clark Equipment Co. (Benton Harbor, Mich.) and Wagner Tractor Co. (Portland, Ore.) have segmented rolls that can be substituted for the rubber-tired wheels of their large tractors.



TOWED VIBRATORY ROLLERS—Gasoline engines usually power the vibrators of these units, but the one below is hydraulically driven by power take-off.



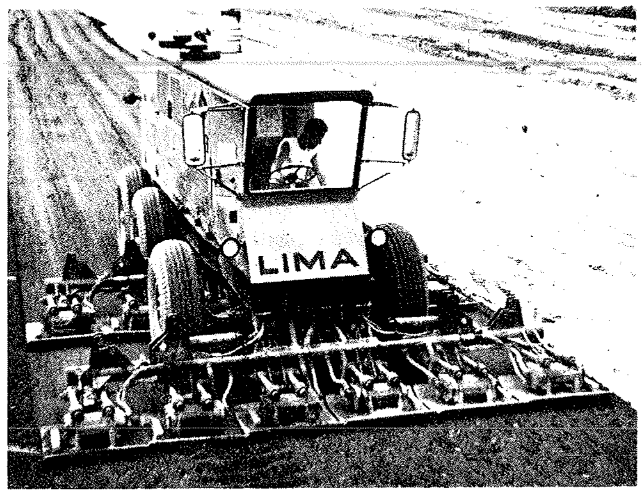
Vibratory Compactors

Sands and sandy silts are granular materials, and their soil particles stack by nesting in the void spaces between other grains. When shaken or vibrated they will shift themselves into the tightest or closest arrangement. This is maximum density. If these particles are dry, friction may prevent their shifting readily. If they are too wet, they will flow in solution since water is not compressible. The correct amount of water will lubricate the particle movement but will not displace the tightest grain arrangement. This is optimum moisture.

Maximum density at optimum moisture content can be achieved in some granular soils by some steel-wheel or rubber-tired static-weight units, but is best gotten by vibratory (dynamic) compactors.

There are two principal classes of vibratory compacting machines: rollers and plates. Rollers impart vibrations to the soil through a steel drum or rubber-tired wheels and thus, in effect, serve a double function. Plates generally are wheel mounted, but they apply their vibrations directly to the ground through skids or shoes.

In compacting granular material, some number (frequency) of blows in a given period gets it down faster and tighter than fewer blows of a heavier order (amplitude.) The combination of frequency and amplitude of vibra-



MULTIPLE-SHOE VIBRATOR—Big unit carries 12 shoes, covers 15-ft width.

tions gets varying results with varying sands, depending on grain size distribution and moisture, since every material has its own frequency rate (or resonance). Vibrations vary from 1,000 per min in some compaction machines to 5,000 per min in others, and it is more economical to use a machine that will vibrate at the soil's resonant frequency.

There are two schools of thought regarding resonant frequency. One claims that since the resonance factor of various granular materials determines their compactability and the depths to which compaction will be effective, then before going ahead with the selection of a dynamic compactor it is best to have some competent laboratory run a set of vibratory compaction tests. The other viewpoint is that only in compacted fills for plant foundations where the structure will contain vibrating machinery are such tests necessary. On highways or airports where the embankment is subjected to impact loads, or in dams where the stress is static, no laboratory vibration tests are needed. However, a full-scale test-trial on the job is always in order before any equipment is finally chosen.

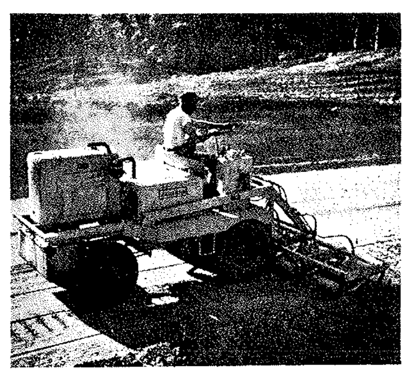
Low frequencies (1,000 to 3,000 vpm), usually are generated in vibrating rollers. A small engine of some sort, through a transmission system, spins a shaft with a single or double eccentric-weight element inside the roll. Single-element vibrators require more input power, and their vibrations are of a higher amplitude, than the double-element type due to the size of weights.

High frequencies (3,000 to 5,000 vpm) handle less eccentric weight and, consequently, have lower amplitudes. (A high-frequency high-amplitude machine would require an inefficient amount of power for work done, and also would require too much strength in the structural frame.) High-frequency, low-amplitude machines can be either roll or plate type.

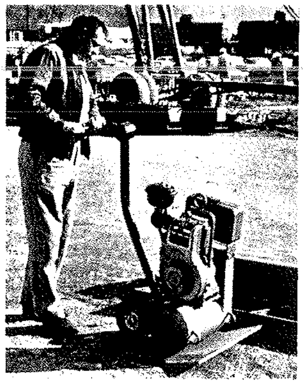
Plates, or shoes, or skids (as they are variously called) are individually vibrated by eccentric devices driven electrically, hydraulically, or mechanically. The eccentrics shake the plates both ways in all three directions (a "six-way"), or just up and down, or crosswise. Shoes are generally about 2x3 ft in contact surface, and are mounted on some self-propelled frame in rows of two to six. Smaller vibrating-plate type units of various sizes, guided individually by hand, are suited for compacting bottoms of trenches, confined areas, and steep slopes. Maintenance on the many moving parts of vibratory units is comparatively high.

One advantage of vibratory compaction is that it keeps the compacted surface fairly well sealed against evaporation of internal moisture and also against entrance of new water. This crusting effect permits rapid resumption of work after rain. Due to the penetrating effect of the vibrations, this crusting does not prevent uniform compaction through the entire lift, as is the case with crusting under a static roller.

Tampo Manufacturing Co. (San Antonio, Tex.) has recently introduced a self-propelled vibratory roller of a size that is better used on base and flexible pavement construction but also is practical on fill work. The driving roll is 42½ in. wide with vibrating frequency variable up to 2,200 vpm. Although it weighs only 4 tons,



VIBRATORY COMPACTOR – Self-propelled rig gives 4,200 3-ton blows per min, generated electrically.



SINGLE-SHOE VIBRATOR—It gives up to 7,500-lb blows 2,500 per min.

VIBRATORY ROLLER—A 1-ton static, it's equivalent to an 8-tonner.

the compacting power of this machine is reported to be equal to that furnished by a static roller of 16 to 20-ton capacity. Tampo has also introduced a meter for measuring the resonance frequency of materials being compacted. Called a Vibra-Meter, this dashboard-mounted device allows the roller operator to read the resonance frequency point on a dial and make roller adjustments accordingly for alleged full compaction efficiency.

Littleford Bros. (Cincinnati, Ohio) also makes a self-propelled vibrating roller. Weighing 1 ton, its compactive ability is listed as equal to that of an 8-ton static-weight roller.

Combining both static and vibratory compaction principles in the same unit, Buffalo-Springfield developed a three-axle roller with vibration on the middle roll. The vibratory roll is retractable, which allows the roller to be used as a two-axle tandem unit. Last year this roller was used successfully in several states for compacting base courses in lifts up to 9 in.

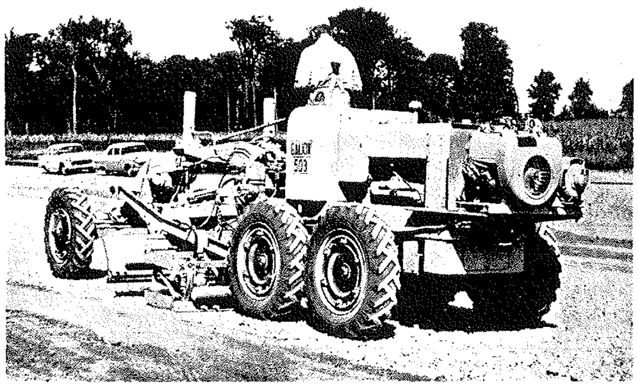
Another approach to combining static and vibratory compaction is taken by Austin-Western (Aurora, Ill.) and Galion Iron Works (Galion, Ohio). They attach vibrating plates to their regular steel-wheel rollers.

Along a somewhat different line, Iowa Manufacturing Co. (Cedar Rapids) has combined vibratory compaction with large-rubber-tire rolling. Their larger towed unit weighs 30 tons, and a smaller version 12½. The machines push the soil straight down, with a minimum of lateral displacement.

And Vibro-Plus Products (Stanhope, N. J.) has added vibrators to towed sheepsfoot rollers to make units that are effective in clays with high sand or silt content. Vibration frequency is variable from 1,400 to 1,600 vpm, and at maximum frequency the unit delivers a 10-ton impact.

Seaman-Gunnison (Milwaukee) has taken yet a different tack in multiple-type compaction equipment. They combine small-rubber-tire rolling with steel-wheel rolling—and vibration can be applied to the steel roll if desired. This self-propelled machine can be used on almost any type of soil under almost any conditions.

Its axles for tires and steel roll are in tandem and are hydraulically mounted so tires and roll can be used separately or together. In normal mixed-grade soils the machine makes two or three passes on the lift with the tires down. This applies kneading action through high unit loads, leaving compacted ruts and somewhat looser ridges. Next, the steel roll is lowered until the weight is carried by both axles. Then the roll compacts the ridges, and a uniform high density results.



VIBRATORY GRADER—Separate engine and generator power four 26-in. shoes.

Impact Compactors

When a compaction unit has a very low frequency and a very high amplitude it is classed as an impact compactor even though it does have a somewhat vibratory effect. Generally, these are hand-held tampers or rammers and are

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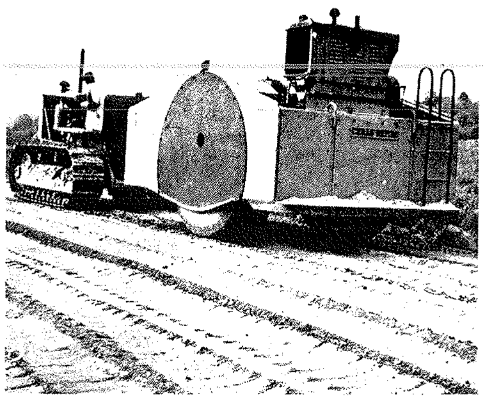


VIBRATORY ON RUBBER—On two large tires, 30-ton ballasted roller vibrates 600 to 1,400 cycles per min.

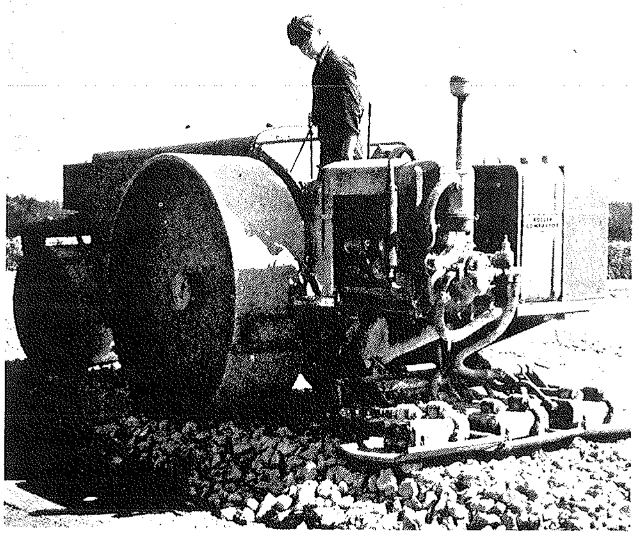
MULTIPLE COMPACTOR—Versatile self-propelled rig combines rubber and steel rolling. Vibration can be added.

used in small areas and confined spaces. Most familiar are the air-powered tampers, used singly or with a fitting that holds three.

But Barco Mfg. Co. (Barrington, Ill.) makes a rammer with a self-contained gasoline engine that makes the entire unit jump up and down. Guided by an operator, the 210-lb machine jumps 13 or 14 in. off the ground.

Wacker Corp. (Hartford, Wis.) also makes a jumping rammer driven by a gasoline engine. The percussion rate of this 115-lb tool's 3-in. blows can be varied from 450 to 650 per min by throttle control. And Complete Machinery Co. (Long Island City 6, N.Y.) has a gasoline-powered 220-lb rammer that jumps 18 in. 80 times a minute.

Schield Bantam Co. (Waverly, Iowa) makes a crane-handled impact compactor. A concrete-filled steel box, with a total weight of 1,375 lb, operates as a drop hammer within a set of steel box



DUAL COMPACTOR—Static weight compaction is augmented by vibrating pads.

IMPACT COMPACTOR—Powered by gas engine, it jumps up and down.

SOIL CLASSIFICATION SYSTEM and COMPACTION EQUIPMENT SELECTION *

	1			2	3	4
	Major divisions			Soil groups and typical names	Comparable groups in BPR classification	Value as foundation when not subject to frost action
1.	Coarse-grained soils	Gravel and gravelly soils		Well-graded gravel and gravel-sand mixtures, little or no fines	A-3	Excellent
2.				Well-graded gravel-sand mixtures with excellent clay binder	A-1	Excellent
3.				Poorly graded gravel and gravel-sand mixtures, little or no fines	A-3	Excellent
4.				Gravel with fines, silty gravel, clayey gravel, poorly graded gravel-sand-clay mixtures	A-2	Good to excellent
5.		Sands and sandy soils		Well-graded sands and gravelly sands, little or no fines	A-3	Excellent
6.				Well-graded sand with excellent clay binder	A-1	Excellent
7.				Poorly graded sands, little or no fines	A-3	Good
8.				Sand with fines, silty sands, clayey sands, poorly graded sand-clay mixtures	A-2	Fair to good
9.	Fine-grained soils containing little or no coarse grained material.	Fine-grained soils having low to medium compressibility; liquid limit < 50		Silts (inorganic) and very fine sands, mo, rock flour, silty or clayey fine sands with slight plasticity	A-4	Fair to poor
10.				Clays (inorganic) of low to medium plasticity, sandy clays, silty clays, lean clays	A-4 A-6 A-7	Fair to poor
11.				Organic silts and organic silt-clays of low plasticity	A-4 A-7	Poor
12.		Fine-grained soils having high compressibility; liquid limit < 50		Micaceous or diatomaceous fine sandy and silty soils, elastic silts	A-5	Poor to very poor
13.				Clays (inorganic) of high plasticity, fat clays	A-6 A-7	Poor to very poor
14.				Organic clays of medium to high plasticity	A-7 A-8	Very poor
15.	Fibrous organic soils with very high compressibility			Peat and other highly organic swamp soils	A-8	Extremely poor

* Suggestions based on Table 4 in "Classification and Identification of Soils" by Dr. Arthur Casagrande, American Society of Civil Engineers' Transactions paper No. 2351.

COMPACTION EQUIPMENT USE

COMPACTOR TYPE	SOIL BEST SUITED FOR	Max. Effect In Loose LIFT (in.)	DENSITY GAINED IN LIFT	MAX. WEIGHT (TONS)
STEEL TANDEM TWO AXLE	Sandy silts, most granular material with some clay binder	4 to 8	Average*	16
STEEL TANDEM THREE AXLE	Same as above	4 to 8	Average*	20
STEEL THREE WHEEL	Granular or granular-plastic material	4 to 8	Average* to Uniform	20
PNEUMATIC SMALL TIRE	Sandy silts, sandy clays, gravelly sands & clays with few fines	4 to 8	Average* to Uniform	12
PNEUMATIC LARGE TIRE	All (if economical)	to 24	Average*	50
SHEEPSFOOT	Clays, clay silts, silty clays, gravels with clay binder	7 to 12	Nearly Uniform	20
VIBRATORY	Sands, sandy silts, silty sands	3 to 6	Uniform	30
COMBINATIONS	All	3 to 6	Uniform	20

* *Density diminishes with depth*

leads. Hammer impact area is about 470 sq in., and maximum hammer drop is 10 ft.

Equipment Selection

The ultimate goal is to build a stable, acceptable embankment in the shortest time at least cost. And the contractor gains or loses depending upon the equipment he selects to do the job. Unfortunately, there are no pat rules to make this choice a routine matter. The variations in specifications, soil types, machines available, and operational methods make this even more complex. There is no panacea that will work magic under all conditions.

But there are some bases for reasonable judgment in selecting a piece of equipment. Two accompanying tables give simplified general outlines of what to take

5		6	7	8	9	10	
Value as Wearing Surface for Stage or Emergency Construction		Potential frost action	Compressibility and expansion	Drainage characteristics **	Field compaction characteristics	Equipment Best Suited	
With dust palliative	With bituminous surface treatment						
Fair to poor	Excellent	None to very slight	Almost none	Excellent	Excellent	Crawler tractor, rubber tired equipment ***	1.
Excellent	Excellent	Medium	Very slight	Practically impervious	Excellent	Tamping roller, rubber tired equipment ***	2.
Poor	Poor to fair	None to very slight	Almost none	Excellent	Good to excellent	Crawler tractor, rubber tired equipment ***	3.
Poor to good	Fair to good	Slight to medium	Almost none to slight	Fair to practically impervious	Good to excellent	Crawler tractor, rubber tired equipment, tamping roller***	4.
Poor	Good	None to very slight	Almost none	Excellent	Excellent	Crawler tractor, rubber tired equipment ***	5.
Excellent	Excellent	Medium	Very slight	Practically impervious	Excellent	Tamping roller, rubber tired equipment ***	6.
Poor	Poor	None to very slight	Almost none	Excellent	Good to excellent	Crawler tractor, rubber tired equipment ***	7.
Poor to good	Poor to good	Slight to high	Almost none to medium	Fair to practically impervious	Good to excellent	Crawler tractor, rubber tired equipment, tamping roller ***	8.
Poor		Medium to very high	Slight to medium	Fair to poor	Good to poor; close control essential	Rubber tired roller	9.
Poor		Medium to high	Medium	Practically impervious	Fair to good	Tamping roller	10.
Very poor		Medium to high	Medium to high	Poor	Fair to poor	Tamping roller	11.
Very poor		Medium to very high	High	Fair to poor	Poor to very poor		12.
Very poor		Medium	High	Practically impervious	Fair to poor	Tamping roller	13.
Useless		Medium	High	Practically impervious	Poor to very poor		14.
Useless		Slight	Very high	Fair to poor	Compaction not practical		15.

** Do not apply to undisturbed materials having fissures and root holes, such as most surface soils.
*** Ed. Note: Vibratory compactors also suited.

into account. By combining information from these tables with suggestions given previously, the range of field equipment can be narrowed down for consideration. Full-scale field trials should then be made under conditions expected to prevail throughout the job and, on the basis of comparative results, final choice should be made. We shall learn in the next installment about methods, but no method can be successful without the proper machines.

VIBRATORY ROLLERS

Bros Inc., Minneapolis 14, Minn.
Browning Mfg. Co., San Antonio 6, Tex.
Buffalo-Springfield Co., Springfield, Ohio
Essick Mfg. Co., Los Angeles 21, Calif.
Iowa Mfg. Co., Cedar Rapids, Iowa
Littleford Bros., Inc., Cincinnati 2, Ohio
Rolcor Industries, Minneapolis 3, Minn.
Rosco Mfg. Co., Minneapolis 6, Minn.
Seaman-Gunnison Corp., Milwaukee 15, Wis.
Shovel Supply Co., Dallas 21, Tex.
Tampo Mfg. Co., San Antonio 6, Tex.
Vibro Plus Products, Inc., Stanhope, N.J.
Western Equipment Div. (Douglas Motors Corp.), Milwaukee, Wis.

VIBRATORY PLATES

Austin-Western, Aurora, Ill.
Baldwin-Lima-Hamilton Corp., Lima, Ohio
Galion Iron Works & Mfg. Co., Galion, Ohio
International Vibration Co., Cleveland 10, Ohio
Jackson Vibrators, Inc., Ludington, Mich.
Jay Co., Columbus 7, Ohio
Kelley Machine Div. (Wiesner-Rapp Co.), Buffalo 23, N.Y..
Maginniss Power Tool Co., Mansfield, Ohio
Master Vibrator Co., Dayton, Ohio
Muller Machinery Co., Inc., Metuchen, N.J.
Racine Hydraulics & Machinery, Inc., Racine, Wis.
Shovel Supply Co., Dallas 21, Tex.
Stow Mfg. Co., Binghamton, N.Y.
Vibro Plus Products, Inc., Stanhope, N.J.
Wacker Corp., Hartford, Wis.

PNEUMATIC TAMPERS

Chicago Pneumatic Tool Co., New York 17, N.Y.
Davey Compressor Co., Kent, Ohio
Gardner-Denver Co., Quincy, Ill.
Ingersoll-Rand Co., New York 4, N.Y.
Joy Mfg. Co., Michigan City, Ind.
Schramm, Inc., West Chester, Pa.
Thor Power Tool Co., Chicago 1, Ill.
Worthington Corp., Holyoke, Mass.

SELF-PROPELLED SHEEPSFOOT

Bros Inc., Minneapolis 14, Minn.
R. G. Letourneau Inc., Longview, Tex.
Shovel Supply Co., Dallas 21, Tex.

TOWED SHEEPSFOOT ROLLERS

American Steel Works, Kansas City 8, Mo.
Bros Inc., Minneapolis 14, Minn.
Browning Mfg. Co., San Antonio 6, Tex.
W. E. Grace Mfg. Co., Dallas 15, Tex.
Koehring California Co., Stockton 4, Calif.
LeTourneau-Westinghouse Co., Peoria, Ill.
Littleford Bros., Inc., Cincinnati 2, Ohio
McCoy Co., Denver, Colo.
Shovel Supply Co., Dallas 21, Tex.
Southwest Welding & Mfg. Co., Alhambra, Calif.
The Slusser-McLean Scraper Co., Sidney, Ohio
Tampo Mfg. Co., San Antonio 6, Tex.

4. COMPACTION METHODS

TO A CONTRACTOR, success in embankment construction means making a fair profit from finishing the job to the satisfaction of the owner on or before the required completion date.

To the owning agency, a successful embankment must be at maximum density, be stable and impervious, and perform its intended function—all at moderate cost.

To the supervising engineer, success in such a venture means arbitrating both sides' desired ends as economically and efficiently as reasonably possible.

Economy without sacrifice of quality is the one basic thing all three parties have in common, and methods are the keys to economy. Even in an ideal situation where the specifications are just right for the contractor, and soil conditions are the best, and the equipment he selected is correct and in top mechanical shape, the job still could be a physical and financial fiasco if he does not use all three basic elements in the most advantageous way.

Knowing what types of soil are to be compacted helps the contractor to determine what piece of equipment he might select to do the job. The accompanying "Quick Soil-Typing Guide" is useful for making rough field checks without any apparatus. But it should not eliminate the standard tests for positive confirmation.

Moisture Control

Regardless of the type of soil, proper control of moisture is vital to compaction success. Too little moisture means there will be insufficient lubrication for the soil particles and, therefore, no maximum density. Too much moisture makes the material unsafe and unworkable. With proper moisture control, light compaction equipment may give results superior to those obtained from much heavier equipment where moisture control is poor.

Optimum moisture content for the fill material is determined in the laboratory. The difference between it and the moisture in the borrow will tell whether the material must be dried or wetted.

If it is to be dried, scarifying or disk harrowing or rotary tilling of the loose lifts on the fill will reduce moisture by aeration.

If the material is to be wetted, water can be added either on the fill or at the borrow pit. The amount to add is the difference, in percent by weight, between actual and optimum. This readily can be converted into gallons of water per cubic yard of soil.

At the borrow pit, the necessary water can be added either by sprinkling or by "ponding"—building a series of low earth dikes to make shallow ponds into which water is pumped. In either case, enough time must be allowed before excavation for the water to penetrate and moisten the borrow material uniformly. And usually the material must be over-wetted to compensate for evaporative losses when it is dug, hauled, and spread on the fill.

When water is added to the borrow material after it is placed on the fill, it is usually done with tank trucks fitted with spray bars. The applied water must be worked into the soil by cultivators or harrows before actual compaction begins. As with any fill where there is a possibility of night rains, at the end of the day the fill surface should be left smooth and slightly sloped to aid run-off, and then be sealed by steel rollers.

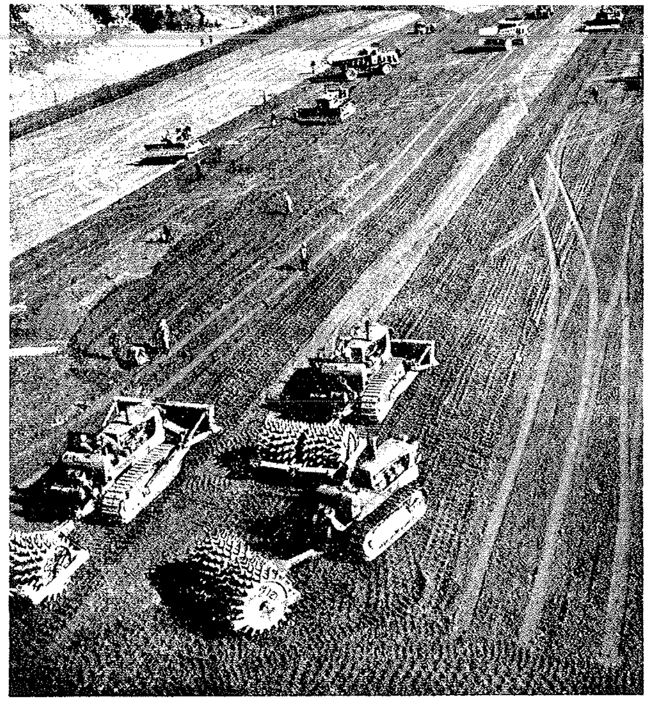
Soil Mixing

Mixing soils at the borrow or on the job is closely allied with moisture. It is the key step that makes subsequent operations easy or difficult. Best results come not from soil of any one predominant type, but from good sensible mixtures of two or more classifications of soils if they are readily available. Here the contractor and the engineer can work closely together in a cooperative effort to develop a superior end product.

In a coarse grained sand, for example, fine grain sand should be added to improve the density since the smaller grains will distribute themselves among the spaces between the larger grains and thereby reduce the amount of voids. If possible, clay should be added as binder and to make it more workable.

Quick Soil-Typing Guide

WHAT TO LOOK FOR	GRANULAR SOILS, FINE SANDS, SILTS	PLASTIC (COHESIVE) SOILS, CLAYS
Visual appearance and feel.	Coarse grains can be seen. Feels gritty when rubbed between fingers.	Grains cannot be seen by naked eye. Feels smooth and greasy when rubbed between fingers.
Movement of water in the spaces.	When a small quantity is shaken in the palm of the hand, water will appear on the surface of the sample. When shaking is stopped, water gradually disappears.	When a small quantity is shaken in the palm of the hand, it will show no signs of water moving out of the voids.
Plasticity when moist.	Very little or no plasticity.	Plastic and sticky. Can be rolled.
Cohesion in dry state.	Little or no cohesive strength in dry state. Will crumble and slake readily.	Has a high dried strength. Crumbles with difficulty and slakes slowly in water.
Settlement in water.	Will settle out of suspension within an hour.	Will stay in suspension in water for several days unless it flocculates.



EFFICIENT COMPACTION – When well graded, at optimum moisture content, and spread evenly in workable lifts, soil compacts well.

In every clayey material, granular soil should be added to provide internal friction, prevent slides, and make possible a better choice of compaction equipment. Gravel and stones bear up well, but do not compact well, are unstable, and may injure some compacting equipment. In general, plastic materials are more workable but have less bearing capacity, while granular materials lend stability due to internal friction and good strength.

What to mix in what proportions is decided by knowing what combination of soils and water is wanted and then using trial and error for refinements.

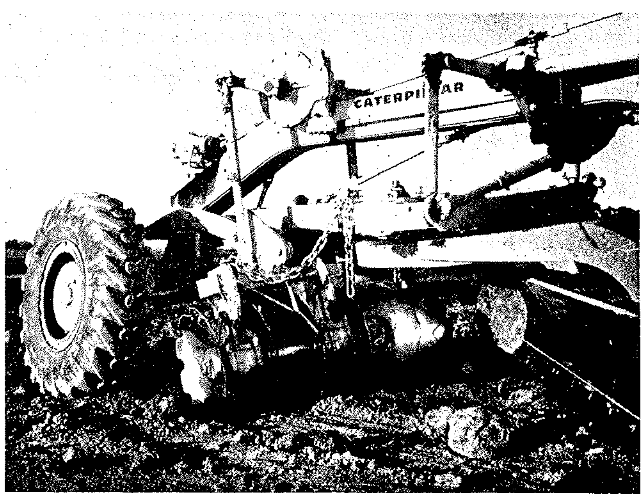
If the soils to be mixed appear together in the same borrow pit in different layers, they often can be handled economically by shovel or belt loader. The machines, working against a mixed face, mingle the different materials directly as they dig and load them into the hauling units.

More generally, the different soils will come from separate sources of borrow and must be mixed thoroughly on the fill before compacting. It is poor practice to make alternate lifts of the different materials. They should be dumped out and mixed together long and well, generally by harrows.

An hour's time spent in processing is worth 3 to 5 hr of random rolling. Dozing serves to level out and spread the loose material; back-dozing provides a pulverizing effect. Grader blading for evenness of layer thickness is important, for then the compaction equipment can give the entire area the same number of passes to reach uniform density throughout. With difficult, lumpy soil, other equipment often must be brought into play: heavy disk harrows, field cultivators, or rotary tillers.

Lift Heights

The question of deep or shallow lifts leaves something to be said on both sides. Here again, other variable conditions have bearing on the issue. Deep lifts (12 to 24 in.) might appear to be the best way to make a fill of 40 to 70 ft. However, not all compaction equipment can handle such lifts economically. And, unless the equipment selected can obtain uniform density throughout the lift, the top may be crusted and the bottom may remain loose. 231



AERATION – Disks on grader can help dry soil that is too wet.

WETTING – Soil that is too dry is soaked by sprinkler truck.

If 12 to 24-in. lifts are chosen, the compaction equipment will have to be the 50-ton large-pneumatic-tired type since nothing smaller can handle the job. Sometimes such lifts are dictated by the desire to incorporate chunks of blasted rock in the fill. Then a sheepsfoot roller should work over the material first to break down any sharp edges near the surface that might injure the super-compactor's tires.

In any deep lift (12 to 24 in.) constant testing should be carried out to be sure that uniform densities are obtained through its entire depth, to avoid later settlement of the embankment.

Even in cases where uniform densities can be obtained throughout deep lifts, the contractor would do well to consider the relative economics of spreading and compacting shallow ones. The additional testing that deep lifts require is added expense. So, too, is the frequent necessity for push-tractors to help the earthmoving units move through the loose, deep lift to unload, even at low speeds.

Shallow lifts, say 3 to 5 in., have much to recommend them. Hauling units can dump at high speed without extra help, more volume of material can be placed per unit of time, better pulverization of fill material is achieved, and lighter compaction equipment can get complete penetration for more uniform density at greater speeds. But the costs of labor, fuel, and equipment ownership and maintenance must be studied on each individual job and for each different soil to determine the most economical height of lift.

Ballasting Machines

Load on a compaction machine can vary from the empty dead weight of the piece itself to a total ballasted load at some maximum capacity. Ballastable machines can be loaded with water, wet sand, or special metallic or concrete weights. Each machine's operating manual should carry a chart of ballasting suggestions for making it work most efficiently in various materials.

If such a chart is not available, here are general suggestions, subject to check on the actual job:

Empty – Working in sands or silts.

Light Ballast–In light gravels and coarse sands.

Heavy Ballast–For wet clays and coarse gravels.

Full Ballast–In dry clays and for proof rolling.

One important word of caution: Do not overballast. This will only break down the material being compacted, and cause soil displacements, subgrade deflections, and pumping. Wheel loads that exceed the bearing capacity of the soil being worked on cannot compact or stabilize that material.

Compaction Speeds and Passes

Some engineers and contractors believe that the first pass of a compacting machine is the most effective, subsequent ones progressively less so, and the effect of any more than eight negligible. Others say each additional pass helps that much more. In the case of plastic material and the sheepsfoot roller, it is automatic; under proper conditions the piece will "walk out" when the job is done. If good compaction isn't achieved in a reasonable time, continued coverage becomes uneconomical and a reason should be sought. It may be too much or too little moisture, too high a lift, inadequate processing, or just plain wrong choice of equipment.

Rolling speeds are closely allied with the number of passes. Only field testing can determine the combination that gives the best results. Slower speeds consume more fuel and time, but they get a deeper effect in plastic materials. Faster speeds are recommended on yielding subbases and on sand in thin lifts. In some deep lifts, higher speed also may be helpful in keeping the loose material from flowing laterally.

The general method of dumping a complete loose lift, processing it, then compacting it is often referred to as the project method. For many cases, two other methods may be considered: the progressive method, and so-called stage compaction.

The progressive method is one where the lifts are really thin (up to 5 in.). A dozer and a grader follow the dumper, then the compactor packs the lift as the job moves along. When dumping has ceased, the first compaction pass will have been completed. Two or three more compacting passes by a light, fast machine, and the next lift can be applied.

Stage compaction is used when a complete loose high lift (over 12 in.) has been dumped, but will not support the weight of the large machine chosen to compact it. A light, unballasted machine must first go over the lift once or twice to form a working crust for the heavier equipment to run on. The heavier rigs, then, must make one or two passes before loading-up ballast completely to attain required density, which in cases like this is usually 100% of AASHO.

Determining the best compaction method is not always simple. One prominent Southern roadbuilder says, "At the present it is not uncommon at all to find, on different sizeable contracts, that the contractor has spent from $50,000 to $150,000 in his efforts to find the equipment to produce the required compaction results. We personally have devoted considerable experimental effort to this and feel that we have been fairly successful in arriving at an economical method of gaining the high percentage of compaction required today.

"Our method has been to take the large 60-in. sheepsfoot rollers which were formerly crawler-tractor drawn and substitute a wearing or compaction surface on the feet to where it is increased from approximately 6 sq in. of bearing surface to 12 to 16 sq in. By substituting large, high-powered rubber-tired tractors for the track-type units, thereby increasing the speed from 3 mph to 10 and 12 mph, we have developed a medium combining the principles of the sheepsfoot roller, the vibrating roller, and the impact roller.

"By varying the size of our bearing plates, and by varying the speed of our rubber-tired tractors (depending on the types of soil involved), we have been very well satisfied with our compaction results. As you can see, this method could be highly controversial with different manufacturers and, possibly, engineering research."

Weather

Because of the delicate balance of optimum moisture content, compaction seldom should be done in the rain. If sufficient water is available, hot and dry weather is no obstacle. Snow, like rain, should stop a job, and there is a limit to what can be done in extremely cold temperatures. Work on frost susceptible materials is uneconomical. That's because the effort required to compact granular soils properly at temperatures below freezing is several times that necessary to do the same job when the soil is thawed. Proper compaction of cohesive soils that freeze into clods is practically impossible.

However, in colder climates, many contractors on large jobs find it economical to stockpile fill material near the job sites during the winter months when they would not ordinarily be working. While the ground is frozen, they can make heavy hauls across lots, swamps, and streams, and make the trips shorter and direct. This way, too, they keep force and plant working during the slow months to free some haul rigs for other jobs during busy times.

PROGRESSIVE METHOD—On thin lifts, smaller machines get good results while working at high speeds. Above, vibratory rollers are drawn by rubber-tired tractor.

Test Embankments

Within the limits of the specifications, a contractor can use the information given in this series of articles in the selection of some types of equipment and the elimination of others. The final choice of one or two from the narrowed range must be made in actual field test in combination with testing the other variables: moisture, proper soil mixture, height of lift, ballast load, speed and number of passes.

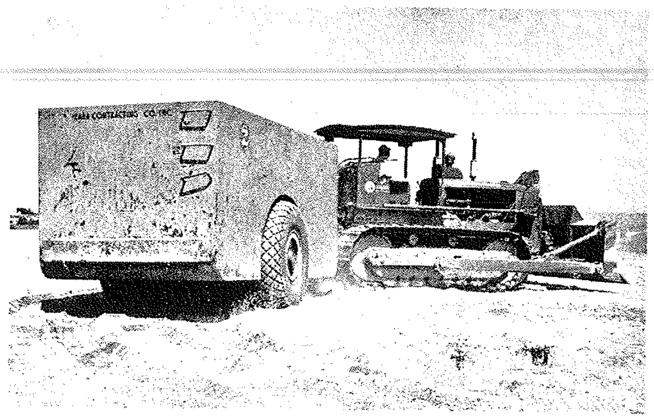
Earthwork specifications often require the contractor first to build some part of the final embankment as a "test embankment." Even when not specified, a test embankment often can save money in the long run. 233

Laboratory testing can establish specified densities and indicate corresponding optimum moisture ranges. The contractor can then indicate what materials will be used where, and what equipment he has available, as dictated by his proposed construction schedule. Then, by varying factors such as moisture content, thickness of layer, placement methods, compaction equipment, and compactive effort, the best

STAGE COMPACTION—An unballasted machine often makes the first pass on loose, high lifts to form crust for heavier rigs that will bring soil to specifications.

combination to achieve these desirable densities and conditions may be determined by field testing. Of course, the contractor's costs of equipment operation and manpower per unit volume of embankment are a necessary consideration.

This obviously leads to more efficient inspection by the engineer. He should know what moistures and field operations will produce desired results without excessive and delaying field testing. It also provides the contractor with knowledge of what is his most efficient operation, and allows for his scheduling of equipment with minimum delay. For instance, there is no need to require eight full passes with a sheepsfoot roller when four will do. Nor is there need to shut down a grading operation while the engineer tests the entire embankment layer, only to order more rolling.



PROOF ROLLING—After normal compaction of subgrades is completed, a few passes with a large-tired roller will help correct deficiencies, check on moisture.

Proof Rolling

Subgrades often are tested by giving them a few passes of a large-tired heavy proof roller after completion of normal compaction. When properly specified and when the soil's moisture is in the optimum range, such rolling will correct compaction deficiencies. When the soil is too wet, proof rolling will indicate it so the condition can be corrected.

If the material is too dry, however, there is danger that the roller will give a false indication of firmness. Then, as moisture increases later, the fill will weaken.

Contractors who do not have large-tired rollers need not give up the idea of proof-rolling, since it is the unit pressure and not the total load that counts. Small-tired rollers may be utilized if ballast is properly calculated. They have the added advantage of finding smaller wet spots often bridged by the larger machines.

Conclusion

Compaction is a subject so vast, and complex, and which depends upon so many variables, that one would be a fool to pretend to offer a solution to all its problems. This series of articles will be successful if it helps the contractor to understand some of these dependent functions and how to attempt to cope with many of the problems that occur in the course of normal job operations. The utmost gratitude is expressed to the great number of experienced men in all corners of the field who cheerfully gave of their time and knowledge in order that this series could be produced accurately and informatively. They, like I, want to promote one objective; better earth compaction results at less cost.

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M. D. MORRIS

Bibliography

The following bibliography contains two sets of references. The first set consists of a reference for each selected text that appeared in the preceding part of this compendium. The second set consists of references to additional publications that either were cited in the selected texts or are closely associated with material that was presented in the overview and selected texts. Each reference has five parts that are explained and illustrated below.

(a) Reference number: This number gives the position of the reference within this particular bibliography. It is used in the compendium index but should ***not*** be used when ordering publications.

(b) Title: This is either the title of the complete publication or the title of an article or section within a journal, report, or book.

(c) Bibliographic data: This paragraph gives names of personal or organizational authors (if any), the publisher's name and location, the date of publication, and the number of pages represented by the title as given above. In some references, the paragraph ends with an order number for the publication in parentheses.

Bibliografía

La siguiente bibliografía contiene dos series de referencias. La primera serie consiste en una referencia para cada texto seleccionado que apareció en la parte anterior de este compendio. La segunda serie consiste en referencias a publicaciones adicionales que fueron mencionadas en los textos seleccionados o que se asocian íntimamente con el material que se presentó en la vista general y los textos seleccionados. Cada referencia tiene cinco partes que se explican y se ilustran abajo.

(a) Número de referencia: este número indica la posición de la referencia dentro de esta bibliografía en particular. Se utiliza en el índice del compendio pero ***no*** deberá utilizarse al pedir publicaciones.

(b) Título: el título de la publicación completa o el título de un artículo o sección dentro de una revista, informe, o libro.

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(a) Reference number
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(a) Numéro de la référence

(b) Title
(b) Título → A REVIEW OF HIGHWAY DESIGN PRACTICES IN DEVELOPING COUNTRIES
(b) Titre

(c) Bibliographic data
(c) Datos bibliográficos → Cron, Frederick W. Washington, DC: International Bank for Reconstruction and Development; 1975 May. 57 p.
(c) Données bibliographiques

(d) Availability information
(d) Disponibilidad de la información → Order from: International Bank for Reconstruction and Development, 1818 H Street, N.W., Washington, DC 20433.
(d) Disponibilité des documents

(e) Abstract
(e) Resumen → The design standards of some 150 highway projects financed by the International Bank for Reconstruction and Development between 1960 and 1970 are reviewed, and areas of agreement between the standards of the 63 countries studied are identified; practical highway standards based on these areas of agreement are sketched for the guidance of planners in developing countries. The roads discussed here, fall into three functional categories: a small group of expressways, freeways and toll roads carrying large volumes of traffic; a very large group of 2-lane highways carrying a wide range of traffic volumes serving both local and long distance traffic; and a smaller group of low-traffic tertiary or special purpose roads existing primarily for land service. Comments are made on the problem of classifying highway standards, and on the comparison of standards. Conclusions regarding standards for the capacity- related elements of design and standards for the velocity-related elements of design (radius of curvature, stopping sight distance, passing sight distance) are discussed, as well as the horizontal and vertical clearances for bridges. The standard live loadings for bridges, the structural capacity of pavements and legal load limits are covered, and conclusions relating to pavement design, design standards for 2-lane highways, incremental development of highways, and levels of service are presented.
(e) Analyse

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El pedido deberá incluir toda la información dada en las partes (b) y (c).

L'ordre de commande doit inclure toutes les informations données dans les parties (b) et (c).

SELECTED TEXT REFERENCES

Reference 1
SIGNIFICANCE OF QUALITY CONTROL

Kerr, Byron T.; Hénault, Giles G. Proceedings of the Golden Jubilee Convention, Canadian Good Roads Association, held at The Queen Elizabeth, Montreal, Quebec, October 19–22, 1964. Ottawa: Canadian Good Roads Association; 1964; pp. 233–237.

Order from: Roads and Transportation Association of Canada, Technical Information Service, 1765 St. Laurent Boulevard, Ottawa, Ontario, Canada K1G 3V4.

This paper focuses on two main topics: inspection and testing in highway construction work and specifications. Comments are made on the purpose of quality control, and a description is given of the procedures that ensure the quality of workmanship and materials. The value of inspection and testing is discussed in relation to the nature of the samples and the application of test results. The need for specifications to be realistic is noted and ways of achieving this goal are discussed. Some suggestions are made on how inspection and testing can be effective in spite of normal variations in test results and in spite of delays due to the time required for some tests. It is noted that with established sampling and testing procedures experienced personnel can do representative sampling and obtain reliable results, that quality control must include adequate preengineering, and that specifications should be written in collaboration with the materials or quality control engineer.

Reference 2
HIGHWAY MATERIALS - CHAPTER 7: COMPACTION

Krebs, Robert D.; Walker, Richard D. New York, New York: McGraw-Hill Book Company; 1971; pp. 175–223.

Order from: McGraw-Hill Distribution Center, Book Order Department, Princeton Road, Hightstown, New Jersey 08580.

This chapter, which was excerpted from a text prepared for undergraduate civil engineering students, analyzes the compactive effort upon various types of soils and the tests that indicate the degree of compaction of the soils. The chapter provides a general review of the techniques of densification of subgrades, embankments, subbases, bases, and gravel surfaces. Fundamental concepts such as moisture-density relations and saturation moisture content are discussed as well as compaction and compactive effort in relation to soil type. A general guide to the selection of soils on the basis of anticipated embankment performance is included. The properties (stability indices, stability and structure of compacted clay, family of curves) of compacted fine-grained soils and field compaction procedures and requirements are detailed. Quality-control test procedures reviewed include conventional field-density tests and nuclear field-density tests. The field impact compaction tests reviewed include the Ohio typical moisture-density curves method, the Hilf method, and the constant dry weight method. The control-strip technique is also reviewed.

Reference 3
SOILS MANUAL FOR THE DESIGN OF ASPHALT PAVEMENT STRUCTURES

Asphalt Institute. College Park, Maryland; March 1978. 238 p. (Manual Series No. 10; MS–10).

Order from: The Asphalt Institute, Executive Offices and Research Center, Asphalt Institute Building, College Park, Maryland 20740.

The origin, composition and properties of soil, and the significance of tests for soil materials are discussed briefly, and soil investigation and sampling are discussed in some detail because of their importance in obtaining accurate test results. Three soil classification systems (The American Association of State Highway and Transportation Officials Classification, the Unified Soil Classification, and the Pedological Classification of the U.S. Department of Agriculture's Soil Conservation Service), and four principal testing methods used in selecting pavement thicknesses are included. These tests are (a) bearing ratio of laboratory-compacted soils - ASTM Designation D 1883, (b) plate bearing test - ASTM Designation D 1195, (c) resistance value (R) method - AASHTO Designation T 190 - ASTM Designation D 2844, and (d) resilient modulus of soil - the Asphalt Institute. The importance and use of aerial photographs for highway location, drainage, soil studies and design are also reviewed. Chapters IV and VII are of special significance. Chapter IV (Significance of Tests on Soil Materials) stresses the importance of thorough familiarity with the test methods and the significance and interpretation of the test results. Four basic laboratory soil tests are described, namely, mechanical analysis, specific gravity, consistency tests and indices, and the moisture-density tests. Chapter VII gives a detailed description of the California bearing ratio (CBR) test procedure. The equipment used, the preparation of the sample, and the calculation and correction of the stress-penetration curve of the ASTM (American Society for Testing and Materials) procedure for CBR determination are detailed.

Reference 4
MECHANICAL DURABILITY OF LATERITIC GRAVELS FROM SOUTHEAST ASIA; SUGGESTED TESTS AND TEST STANDARDS FOR HIGHWAY USES

Shuster, J.A. Australian Road Research, Volume 4, No. 5. Kew, Victoria: Australian Road Research Board; 1970 September; pp. 32-44. (Journal of the Australian Road Research Board).

Order from: Australian Road Research Board, P.O. Box 156 (Bag 4), Nunawading, Victoria 3131, Australia.

An investigation to determine the tests most suitable for evaluating the mechanical durability of lateritic gravels for use in road construction is reported. The relative durability of these materials was also investigated and compared with past performance and durability of lateritic gravels in existing pavement sections in Thailand. The probable range of durability as well as durability test techniques and tentative test standards for these mateirals, when used in pavement sections, was established.

Reference 5
LATERITE AND LATERITIC SOILS AND OTHER PROBLEM SOILS OF THE TROPICS; AN ENGINEERING EVALUATION AND HIGHWAY DESIGN STUDY FOR UNITED STATES AGENCY FOR INTERNATIONAL DEVELOPMENT, VOLUME II, INSTRUCTIONAL MANUAL

Morin, W.J.; Todor, Peter C. Baltimore, Maryland; Lyon Associates, Inc.; 1975. 92 p. (Performed jointly with the Brazilian National Highway Department Road Research Institute; Report # PB-267 263).

Order from: National Technical Information Service 5285 Port Royal Road, Springfield, Virginia 22161.

This is an instructional manual for field inspectors and laboratory technicians who work on engineering and construction projects that utilize tropical soils. The engineering descriptions, procedures, and specifications described are a consolidation of information obtained from Volume I: Laterite and Lateritic Soils and Other Problem Soils of the Tropics (see Reference 14). Both volumes are the final report of a worldwide tropical soil study. This manual summarizes background information (on soils classifications, physical and engineering properties, red tropical soils, and volcanic soils) and reviews test procedures for the evaluation of tropical soil properties (preparation of soils samples, particle size analysis, liquid limit, plastic limit and plasticity index, moisture density relations, specific gravity, California bearing ratio, sand equivalent value, and test for durability of aggregates). Details are given of flexible pavement design (an appendix describes the determination of the coefficient of variation) and the stabilization of selected tropical soils. Design considerations for roads over tropical black clays and a recommended design procedure are set forth. Specifications for subbase, base- and surface-course materials, for excavation of borrow areas, compaction equipment and compaction requirements, and for materials and construction in tropical climates are also presented.

Reference 6
STANDARD SPECIFICATIONS FOR TRANSPORTATION MATERIALS AND METHODS OF SAMPLING AND TESTING

American Association of State Highway and Transportation Officials. Washington, DC. 1978 July. 998 p. (Twelfth Edition).

Order from: American Association of State Highway and Transportation Officials, Suite 225, 444 North Capitol Street, NW, Washington, DC 20001.

This book covers methods of testing and specifications for testing equipment and includes 195 test methods. A complete repertoire of test procedures that have been standardized by the American Association of State Highway and Transportation Officials (AASHTO) is given. These tests cover the areas of hydraulic cement, bituminous cement, soils, aggregates, concrete, brick, joint filler and asphalt plank, culvert pipe and drain tile, metallic materials, and other miscellaneous fields. A numerical sequence of test by test number showing equivalencies between AASHTO and ASTM (American Society for Testing and Materials) is also included.

Reference 7
THE CONSTANT DRY WEIGHT METHOD — A NO-WEIGHING FIELD COMPACTION TEST

Schonfeld, R. Ontario, Canada: Department of Highways; 1968 September. 22 p. (Presented at the 48th Annual Meeting of the Canadian Good Roads Association, Toronto. D.H.O. Report No. RR 141).

Order from: Editor, Research and Development Division, Ministry of Transportation and Communications, Downsview, Ontario, Canada M3M 1J8.

The problem of making quick reliable decisions about the state of compaction of subgrades, embankments, and pavement courses is discussed. Several field compaction test methods are described and a new procedure, the constant dry weight field compaction test, is detailed. In this test, which is an adaptation of the Proctor test (ASTM Designation D698-58T), the percentage compaction is ascertained by comparing the in situ volume of soil sample with the volume of the same sample in the Proctor mold after standard compaction at approximately optimum moisture content. The accuracy and precision of the results obtained from the constant dry weight compaction tests are analyzed and its role in statistical quality control is evaluated. The use of the method to supplement nuclear probe testing in the determination of percentage compaction is also discussed.

Reference 8
TECHNICAL COMMITTEE REPORT ON TESTING OF ROAD MATERIALS

Permanent International Association of Road Congresses. Paris, France; 1979. 52 p. (XVIth World Road Congress, Vienna, September 16-21, 1979).

Order from: Permanent International Association of Road Congresses, Secretariat, British National Committee, St. Christopher House, Southwark Street, London, SE1 07E.

This report makes recommendations to ensure uniformity in certain test methods and terminology. The report consists of three sections. The first section presents recommendations on methods used for testing aggregates. The tests included are particle-size distribution by sieve, Los Angeles test, sand equivalent, polished-stone-value test, quality of fine material passing a 0.075 mm sieve, density measurements (three tests), aggregate shape (two tests), sample reduction to provide the test sample, and sensitivity to freezing test. The second section of the report summarizes the data collected from a questionnaire on the Marshall test and its applications that was circulated to 46 countries. Conclusions derived from the survey are presented. These conclusions relate to the need for extreme care in the preparation of test specimens, the reduction of the variability of the stability and flow measurements, methods of defining flow, the method of assessing the degree of compaction, and data on the precision of the various procedures. The third section of the report presents a draft terminology on the treatment, improvement, and stabilization of soil and materials used for road foundations. See Reference 16.

Reference 9
PROCEEDINGS OF THE 10TH ANNUAL ENGINEERING GEOLOGY AND SOILS ENGINEERING SYMPOSIUM

Idaho Department of Highways; University of Idaho; Idaho State University. Boise, Idaho: Idaho Department of Highways; 1972. 339 p. (Proceedings of the 10th Annual Symposium Held at Moscow, Idaho, April 5, 6, 7, 1972).

Order from: Idaho Department of Transportation, Annual Symposium on Engineering Geology and Soils Engineering, 3311 W. State Street, P.O. Box 7129, Boise, Idaho 83707.

The following papers are included in this publication: The Need for Expanding the Scope of Geotechnical Investigations; Contributions of Engineering Geology and Land Use; Geological Hazards and Cities; Engineering Geology of the Proposed Snake River Bridge Site Near Twin Falls, Idaho; Subsurface Openings in Soil Fields; Electrical Earth Resistivity Surveying in Landfill Investigations; A Preliminary Evaluation of the Centralia Strip-Mine Area for Sanitary Landfill Purpose; Soil Classification for Compaction; Experiences with Compaction of Hydraulic Fills; Use of Engineering Geology for Planning New Roads; Correction of an Embankment Foundation Failure at Thania Rocks, Idaho; Reinforced Earth as a Highway Structure; Structural-Environmental Characteristics of Tailings Ponds; Administration of Ground Water as a Nonrenewable Resource; Some Geologic Criteria for Evaluating Engineering Properties of Rock Masses; Recent Developments in Hard Rock Tunneling; Some Characteristics of Gouge Material as Related to Stability Underground; Factors in Determining Seismic Risks with Application to Southeast Idaho; Earthquake History of Bear River Valley, Idaho; Applied Geophysics for Engineering Problems; The Development and Application of Tiebacks and Soil Anchors in Open Foundations; Modeling Failure of Cohesive Slopes; Floor Foundation Stabilization in Permafrost at Barrow, Alaska; New Techniques in Foam Drilling; Exploration and Sampling '72; and Overcoring Stress Relief Experience in Belt Rocks.

Reference 10
EARTH COMPACTION

Morris, M.D. New York, New York: McGraw-Hill Publishing Company, Incorporated; 1961, 32 p. (Reprint from Construction Methods and Equipment).

Order from: Construction Contracting, Reader Services Department, 2500 Artesia Boulevard, Redondo Beach, California 90278.

This reprint from the periodical Construction Methods and Equipment, discusses construction specifications, reconnaissance materials and tests, compaction equipment and compaction methods. Four basic types of standard specifications and their use and the comparison of specifications are discussed. Comments are made on soil reconnaissance, soil types, and soil tests, including liquid tests, sand tests, and nuclear tests. Steel rollers, pneumatic-tired rollers, sheepsfoot rollers, vibratory compactors, and impact compactors are discussed, and the importance of selecting the right type of equipment is emphasized. Tables are included that give simplified outlines of the various characteristics to be considered in selecting compacting equipment. Knowledge of the type of soil to be compacted will help in selecting the type of equipment to be used. A quick soil-typing guide for rough field checks is included. The control of moisture, soil mixing, the question of lift heights, the ballasting of machines, compaction speeds and passes, the influence of the weather, the building of the test embankment, and the proof rolling of subgrades are also covered.

ADDITIONAL REFERENCES

Reference 11
FACTORS THAT INFLUENCE FIELD COMPACTION OF SOILS: COMPACTION CHARACTERISTICS OF FIELD EQUIPMENT.

Johnson, A.W.; Sallberg, J.R. Washington, DC: Highway Research Board; 1960; 206 p. (Bulletin 272. National Academy of Sciences, National Research Council publication 810).

Order from: University Microfilms International, 300 North Zeeb Road, Ann Arbor, Michigan 48106.

This bulletin, which summarizes the results of researches with full-scale equipment and provides other useful data, is intended for use by construction engineers, project engineers, and their technical assistants. The introductory portions of this bulletin present some historical highlights and state some of the principles that govern compaction in the field as well as in the laboratory. The text focuses on data that illustrate the compaction and operation characteristics of the several types of compactors on different types of soils. Brief statements are made on methods used as aids in the control of moisture content and unit weight in construction. Data on current (as of March 1960) state highway department practices as indicated by specifications governing compaction requirements and compaction equipment are tabulated. Tables of manufacturers' specifications for compaction equipment and data on permissible loads and the inflation pressures for tires used on pneumatic-tired rollers are also included.

Reference 12
SYMPOSIUM ON COMPACTION OF EARTHWORK AND GRANULAR BASES. 17 REPORTS.

Highway Research Board. Washington, DC; 1967, 279 p. (Highway Research Record Number 177. National Research Council, National Academy of Sciences - National Academy of Engineering Publication 1508).

Order from: University Microfilms International, 300 North Zeeb Road, Ann Arbor, Michigan 48106.

The papers in this publication that deal with various aspects of the compaction problem will be of interest to researchers and practicing engineers concerned with the design and construction of compacted earth structures. They include reports on specification trends and major compaction problems, available information on the structural properties of compacted soil, and a laboratory investigation of the geological properties of compacted soil that leads to a general program to determine the optimum type and amount of compaction energy. A large field study to evaluate typical compactors and rapid control methods presents data indicating the major effect of moisture on the compaction of soils. These conclusions have a direct application to construction practice. A new laboratory compaction test for granular material is presented. Papers on rapid nondestructive control tests and methods discuss many aspects in which management of inspection and utilization of modern equipment and methods increase productivity in the compaction of granular base materials and soils. Proposed new testing techniques and evaluation of current compaction and controls methods are also included.

Reference 13
THE COMPACTION OF SOIL AND ROCK MATERIALS FOR HIGHWAY PURPOSES

Wahls, H.E.; Fisher, C.P.; Langfelder, L.J. Raleigh, North Carolina: North Carolina State University, Department of Civil Engineering; 1966 August. 468 p. (Record # PB 227 931/3).

Order from: National Technical Information Service, Springfield, Virginia 22151.

This review of the current state of the art of the compaction of soil and rock materials for highway purposes, evaluates current (1966) state highway department specifications and field construction practices, and recommends methods for improving earthwork construction. The report reviews the mechanics of compaction, moisture-density relations, properties of compacted materials and the factors that affect them, behavioral requirements of pavement elements, variability and reliability of testing procedures, and statistical quality control methodology. Specifications and field practices are evaluated and recommendations are made for their modification. An annotated bibliography on compaction is included.

Reference 14
RAPID TEST METHODS FOR FIELD CONTROL OF HIGHWAY CONSTRUCTION

Antrim, J.D.; Brown, F.B.; Busching, H.W.; Chisman, J.A.; Moore, J.H.; Rostron, J.P.; Schwartz, A.E. Washington, DC: Transportation Research Board, 1970. 89 p. (National Cooperative Highway Research Program Report 103).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, NW, Washington, DC 20418.

This two-phase research project determined the state of the art in the development, need and use of rapid test methods, and developed and evaluated new test methods for field control of construction. In the first phase the literature on new rapid methods and current practices in quality control and acceptance testing was surveyed, and a statistical study was made to determine time limits for rapid tests. It was found that the broad area of compaction control, which includes the determination of standard densities and field densities for base and earthwork construction as well as asphalt pavements, had the greatest need for rapid test methods. Next in order of priority were determinations of concrete strength and base course gradation. In the second phase, development and evaluation studies were conducted in the areas of asphalt content and compaction control of base course materials, and the density and moisture content of soils. Among the methods applicable to asphalt content determination were the pat-stain method, the Wyoming flask method, and the ignition method. A sampling technique that uses a thermoplastic cup for easy removal of bituminous concrete density specimens was developed, and a laboratory study was conducted to determine the feasibility of using an incomplete series of sieves to estimate the gradation of several types of aggregate. Methods for determining the density of base course materials and soils were also studied. The feasibility of the technique for measuring the density of soils using ultrasonic ceramic crystal driver and pickup transducers was studied. Two rapid test methods of determining moisture content were evaluated: the alcohol burning method and the calcium carbide gas pressure method. An extensive annotated bibliography is included.

Reference 15
THE INVESTIGATION OF PRESENT AGGREGATE GRADATION CONTROL PRACTICES AND THE DEVELOPMENT OF SHORT-CUT OR ALTERNATIVE TEST METHODS

Richardson, E.S.; McCleland, R.L.; Rosenbaum, R.E.; Barger, R.J. Vancouver, Washington. Washington, DC: U.S. Federal Highway Administration, Offices of Research and Development; 1977 April. 108 p. (Report # PB 274 154/4ST).

Order from National Technical Information Service, Springfield, Virginia 22161.

This report covers the first phase of an investigation into current aggregate gradation control practices. It presents the results of a literature search and a survey of current testing procedures and sampling and testing frequencies for various aggregate uses. Summary information is presented on length and method of sample drying; size and number of sieves; length and method of shaking; source of the sample (i.e. stockpile, truck, roadway, etc.); and who (producer or consumer) performs the test. Also included are recommendations as to testing and sampling frequencies based on the information gathered. Short-cut and alternative test methods for gradation determination are described, and those that offer promise of being a good substitute for the standard method will be further investigated and tested in the second phase of the study.

Référence 16
COMITE TECHNIQUE DES ESSAIS DE MATERIAUX ROUTIERS

Association Internationale Permanente des Congrès de la Route, Paris, France, 1979. 61 p. (XVIe Congrès Mondial de la Route, Vienne, 16–21 Septembre 1979).

Commandez à: L'Association Internationale Permanente des Congrès de la Route, Secrétariat Général, 43 Avenue du Président Wilson, Paris XVIe.

Ceci est l'édition française du texte choisi no. 8.

Reference 17
EFFECTS OF DIFFERENT METHODS OF STOCKPILING AND HANDLING AGGREGATES

Miller-Warden Associates. Raleigh, North Carolina. Washington, DC: Highway Research Board, 1967. 102 p. (National Cooperative Highway Research Program Report 46).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, NW, Washington, DC 20418.

This report contains the findings and recommendations on aggregate gradation variation resulting from field investigations of stockpiling and base course construction procedures. A segregation index has also been developed for rating the different stockpiling methods. This study involved both uncrushed gravel and a crushed limestone gradation in the evaluation of segregation as related to various stockpiling techniques, plus a measure of degradation caused by handling, spreading, and compaction methods for base courses. Six full-scale stockpiles were built and the degree of segregation was determined. To measure aggregate degradation, six dense-graded aggregate base courses were constructed using crushed limestone from two sources with significantly different Los Angeles abrasion loss histories. Methods of minimizing segregation during stockpiling are discussed. The amount of degradation of the particular aggregates used and base course construction procedures investigated was much lower than anticipated. The report also contains considerable background information on related factors such as sample size, reliability, and statistical concepts.

Reference 18
DENSITY STANDARDS FOR FIELD COMPACTION OF GRANULAR BASES AND SUBBASES

Roston, J.P.; Roberts, F.L.; Baron, W. Washington, DC: Transportation Research Board, 1976. 73 p. (National Cooperative Highway Research Program Report 172).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, NW, Washington, DC 20418.

This report presents the findings of a study to evaluate current procedures and criteria for the setting of density standards to control compaction during construction of granular base and subbase courses and to develop more appropriate procedures and criteria. An extensive laboratory test program, comprising seven conditions and gradations, was carried out. The aggregates tested were a granite-gneiss, a crushed gravel, a dolomitic limestone, and a basalt. A prototype field compaction testing program was conducted using the same four aggregates. Procedures and criteria are proposed for use by highway agencies.

Reference 19
QUALITY ASSURANCE THROUGH PROCESS CONTROL AND ACCEPTANCE SAMPLING

U.S. Department of Transportation, Statistical Quality Control Group, Office of Research and Development, Washington, DC: U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads; 1967, April; 79 p. (Reprinted May 1974). (Report # PB 190 671).

Order from: National Technical Information Service, Springfield, Virginia 22151.

This publication summarizes the status of the studies relating to quality assurance, defines the significant aspects of the problem and presents the statistical definitions and concepts needed for application in specification writing. The basic philosophy of statistical specifications is expressed in general terms, and the problem of fitting statistics to the highway problem is discussed. Problems involved in determining quality requirements and expressing the requirements by specifications are also covered. Writing specifications based on statistical concepts, the normal distribution curve, risks, acceptance sampling plans, acceptance sampling for attributes, the operating characteristics curve, the average outgoing quality curve, and acceptance sampling for variables are discussed in some detail. The use of control charts for variables and control charts for attributes is explained.

Reference 20
STATISTICALLY ORIENTED END-RESULT SPECIFICATIONS

Transportation Research Board. Washington, DC: 1976. 40 p. (National Cooperative Highway Research Program Synthesis of Highway Practice 38).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, NW, Washington, DC 20418.

This report, which discusses the advantages and disadvantages of statistically defensible acceptance plans, extends and amplifies the concepts of an earlier HRB Special Report (#118) with respect to specifications for highway materials and construction, and shows how they have been applied in those instances where the current information is available. There are two types of statistical acceptance plans: the attributes sampling plan (useful when the attribute can be accepted or rejected by visual inspection); and the variables acceptance plan (uses both average values and variability measurements to determine acceptance). Problems associated with such specifications (buyers and sellers risk, defining good and poor material, defining lot size and testing frequency, determining equitable prediction in price, administrative problems, human, legal and economic factors, and cost-effectiveness) are discussed, and contractors and producers quality control systsms are reviewed. Contracting agency aceptance procedures are described, and current practices in state highway agencies (33 states have used, are using or planning to use such specifications) and foreign countries are outlined. Comments from trade associations and producers are also presented.

Reference 21
PROCEEDINGS NATIONAL CONFERENCE ON STATISTICAL QUALITY CONTROL METHODOLOGY IN HIGHWAY AND AIRFIELD CONSTRUCTION (MAY 3–5, 1966)

University of Virginia, School of General Studies.

Charlottesville, Virginia; 1966 November. 664 p. (Report # AD 742270).

Order from: National Technical Information Service, Springfield, Virginia 22151.

In this conference, which summarized existing (1966) knowledge and provided a forum for planning and discussing the techniques and implications of statistical procedures in highway and airfield construction, 33 papers were presented in five sessions. The papers in the first session presented statistical concepts and methods that can provide a basis for inferences on attributes or measured characteristics of a variable product. Papers discussed control charts and acceptance sampling plans as useful techniques for relating the science of statistical inference to the problems of quality control. The papers at the second session noted the use of the statistical approach by many industries, discussed the contractual relationships between government and contractors in quality assurance, as well as the need for experience in the application of statistical methods in road construction, and the problem of selecting appropriate levels of quality. The third session covered the research being done in the measurement of construction. The papers pointed out the magnitude of the variations found in construction and presented proof of the deficiencies in the present methods of quality control. A substantial portion of these variations may be attributed to variations in the sampling and testing procedures. A knowledge of material, sampling, and testing variation is essential to the establishing of realistic, enforceable limits and performance quality criteria. The papers of the fourth session summarized and explained statistical techniques that can be used in the control of highway and airfield construction, assessed these techniques, and gave examples of their application to process control and acceptance testing. The fifth session considered implications of increasing use of statistical quality control methods in highway and airfield construction. Points of view were expressed by various elements of the pavement industry including research management, auditing, highway engineering, materials supply, equipment manufacturing, and contracting.

Index

The following index is an alphabetical list of subject terms, names of people, and names of organizations that appear in one or another of the previous parts of this compendium, i.e., in the overview, selected texts, or bibliography. The subject terms listed are those that are most basic to the understanding of the topic of the compendium.

Subject terms that are not proper nouns are shown in lower case. Personal names that are listed generally represent the authors of selected texts and other references given in the bibliography, but they also represent people who are otherwise identified with the compendium subjects. Personal names are listed as surname followed by initials. Organizations listed are those that have produced information on the topic of the compendium and that continue to be a source of information on the topic. For this reason, postal addresses are given for each organization listed.

Numbers that follow a subject term, personal name, or organization name are the page numbers of this compendium on which the term

Indice

El siguiente índice es una lista alfabética del vocablo del tema, nombres de personas, y nombres de organizaciones que aparecen en una u otra de las partes previas de este compendio, es decir, en la vista general, textos seleccionados, o bibliografía. Los vocablos del tema que aparecen en el índice son aquellos que son necesarios para el entendimiento de la materia del compendio.

Los vocablos del tema que no son nombres propios aparecen en letras minúsculas. Los nombres personales que aparecen representan los autores de los textos seleccionados y otras referencias dadas en la bibliografía, pero también pueden representar a personas que de otra manera están conectadas a los temas del compendio. Los nombres personales aparecen con el apellido seguido por las iniciales. Las organizaciones nombradas son las que han producido información sobre la materia del compendio y que siguen siendo fuentes de información sobre la materia. Por esta razón se dan las direcciones postales de cada organización que aparece en el índice.

Los números que siguen a un vocablo del tema, nombre personal, o nombre de organización son los números de página del compendio donde el vocablo o nombre aparecen. Los números romanos se refieren a las páginas en la vista general, los números arábigos se refieren a páginas en los textos seleccionados, y los números de referencia (por ejemplo, Ref. 5) indican referencias en la bibliografía.

Algunos vocablos del tema y nombres de organizaciones están seguidos por la palabra ***see.*** En tales casos los números de página del com-

Index

Cet index se compose d'une liste alphabétique de mots-clés, noms d'auteurs, et noms d'organisations qui paraissent dans une section ou une autre de ce recueil, c'est à dire dans l'exposé, les textes choisis, ou la bibliographie. Les mots-clés sont ceux qui sont le plus élémentaires à la compréhension de ce recueil.

Les mots-clés qui ne sont pas des noms propres sont imprimés en minuscules. Les noms propres cités sont les noms des auteurs des textes choisis ou de textes de référence cités dans la bibliographie, ou alors les noms d'experts en la matière de ce recueil. Le nom de famille est suivi des initiales des prénoms. Les organisations citées sont celles qui ont fait des recherches sur le sujet de ce recueil et qui continueront à être une source de documentation. Les adresses de toutes ces organisations sont incluses.

Le numéro qui suit chaque mot-clé, nom d'auteur, ou nom d'organisation est le numéro de la page où ce nom ou mot-clé parait. Les numéros or name appears. Roman numerals refer to pages in the overview, Arabic numerals refer to pages in the selected texts, and reference numbers (e.g., Ref. 5) refer to references in the bibliography.

Some subject terms and organization names are followed by the word *see.* In such cases, the compendium page numbers should be sought under the alternative term or name that follows the word *see.* Some subject terms and organization names are followed by the words *see also.* In such cases, relevant references should be sought among the page numbers listed under the terms that follow the words *see also.*

The foregoing explanation is illustrated below.

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pendio se encontrarán bajo el término o nombre alternativo que sigue a la palabra *see.* Algunos vocablos del tema y nombres de organizaciones están seguidos por las palabras *see also.* En tales casos las referencias pertinentes se encontrarán entre los números de página indicados bajo los términos que siguen a las palabras *see also.*

La explicación anterior está subsiguientemente ilustrada.

écrits en chiffres romains se rapportent aux pages de l'exposé et les numéros écrits en chiffres arabes se rapportent aux pages des textes choisis. Les numéros de référence (par exemple, Ref. 5) indiquent les numéros des références de la bibliographie.

Certains mots-clés et noms d'organisations sont suivis du terme *see.* Dans ces cas, le numéro des pages du recueil se trouvera après le mot-clé ou le nom d'organisation qui suit le terme *see.* D'autres mots-clés ou noms d'organisations sont suivis des mots *see also.* Dans ce cas, leurs références se trouveront citées après les mots-clés qui suivent la notation *see also.*

Ces explications sont illustrées ci-dessous.

Illustration (from Comp. 1) **Ilustración** (del Comp. 1) **Illustration** (du Recueil 1)

Selected Text page numbers
Números de página en los Textos Seleccionados
Numéros des pages des Textes Choisis

Subject term and *see also* terms
Vocablo del tema y términos *see also* (ver también)
Mot-clé et *see also*

Organization name and address
Nombre y dirección de la organización
Nom et adresse de l'organisation

Overview page numbers and reference number
Número de página en la Vista General y números de referencia
Numéro des pages de l'Expose et numéros des références

Subject term and *see* term
Vocablo del tema y término *see (ver)*
Mot-clé et *see*

Personal names
Nombres personales
Noms propres

mountainous terrain (see also degree of curvature; design speed; maximum gradient; radius of curvature; shoulder width): 11, 17, 34, 35, 38, 173, 175, 217, 234, 238

National Association of Australian State Road Authorities(P.O. Box J141, Brickfield Hill, N.SW. 2000, Australia):
publications, xxiii, Ref. 8, Ref. 9

no-passing markings and signs: 31, 95, 132

non-passing sight distance, see stopping sight distance

Odier, L.: Ref.3

Oglesby, C.H.: 231, 233, 234, 239, 240, 241, 242, Ref.14

Selected Text page numbers and reference number
Números de página en los Textos Seleccionados y número de referencia
Numéros des pages des Textes Choisis et numéros des références