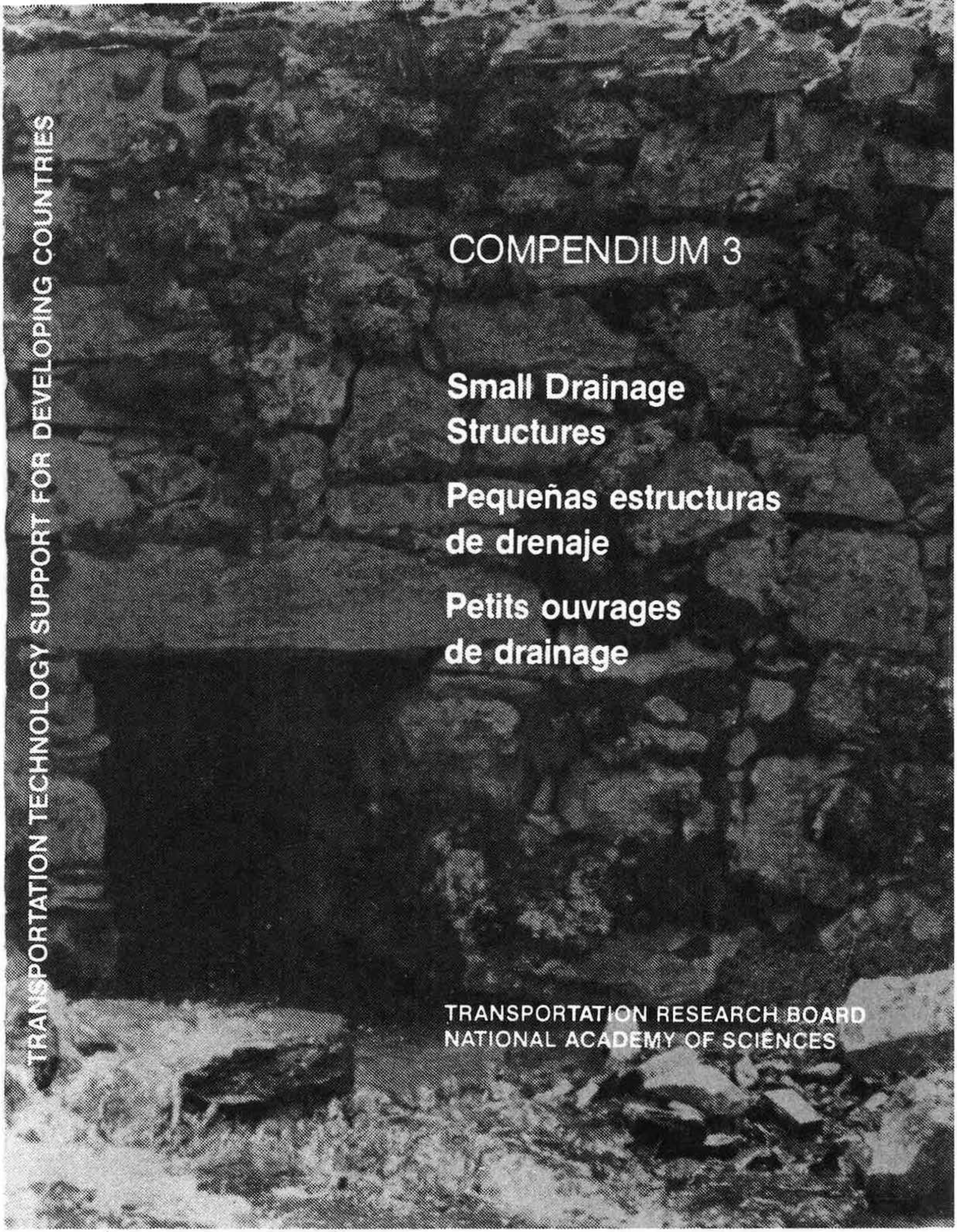

TRANSPORTATION TECHNOLOGY SUPPORT FOR DEVELOPING COUNTRIES

COMPENDIUM 3

Small Drainage Structures

Pequeñas estructuras de drenaje

Petits ouvrages de drainage

TRANSPORTATION RESEARCH BOARD
NATIONAL ACADEMY OF SCIENCES

Contents
Tabla de Materias
Table des Matières

Project Description

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 doc-

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Descripción del Proyecto

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 en las regiones rurales de países en desarrollo todos dependen en gran parte de los medios de transporte. Aunque en ciertas áreas los medios de ferrocarril y agua desempeñan una parte importante, una necesidad universal y dominante es para sistemas viales que proveen un medio asegurado pero relativamente poco costoso para el movimiento de gente y mercancías. La gran parte de esta necesidad es para caminos de bajo volúmen que generalmente mueven unicamente unos 5 a 10 vehículos por día y que pocas veces mueven tanto como 400 vehículos por día.

Con respecto a la economía, calidad, y rendimiento, el planeamiento, diseño, construcción y manutención de caminos de bajo volúmen para regiones rurales de países en desarrollo pueden ser mejorados en gran parte 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 mas desarrollados, y alguna se produce continuamente en los países menos y mas

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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 à l'information par l'intermédiaire de moyens éducatifs et d'autres moyens de communication, dépendent en grande partie des moyens de transport. Bien que les transports par voie férrée 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 en technologie, qui éxistent déjà et sont accéssibles dans beaucoup 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 déumented in papers, articles, and reports that have been written by experts in the field. But much of the technology is undocumented 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, conferences in the United States and abroad, and

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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 esta documentada y existe principalmente en las mentes de aquellos que han desarrollado y aplicado la tecnología por necesidad. En cualquier caso, los conocimientos en existencia sobre la tecnología de caminos de bajo volúmen están grandemente esparcidos geograficamente, varían bastante con respecto al idioma y su forma, y no se encuentran facilmente disponibles para su aplicación a las necesidades de los países en desarrollo.

En octubre de 1977 el Transportation Research Board (TRB) comenzó con 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 manutención 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 información técnica, se provee acciones recíprocas personales con los usuarios por

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veloppés, 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'éxiste que dans l'esprit de ceux qui ont développé et appliqué cette technologie par nécessité. 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 durée 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 éxistante 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. Généralement, l'aboutissement 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.

En plus de la dissémination de cette documentation technique, des visites, des conférences aux Etats Unis et à l'étranger, et 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

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medio de visitas de campaña, 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 asistir 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 tiene la responsabilidad para 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 relativamente limitados

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d'autres formes de communication permettent 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 de 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.

Le personnel attaché à ce projet 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 sera relativement limité, des synthèses de connaissances et de pratique sur des sujets beaucoup plus généraux, et finalement des comptes-rendus practice on somewhat broader 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 12 per year; consultants are employed to prepare syntheses at the rate of 2 per year. At least two conference proceedings will be published during the 3-year period. In summary, this project aims to produce and distribute between 40 and 50 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 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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temas, la síntesis del conocimiento y práctica sobre temas un poco mas ámplios, 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 12 por año; se utilizan consultores para preparar las síntesis a razón de 2 por año. Se publicarán por lo menos dos expedientes de conferencias durante el período de tres años. En breve, este proyecto pretende producir y distribuir entre 40 y 50 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 formularios de retroacción con los productos informativos 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 directo 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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de conférences sur les routes à faible capacité qui seront organisées complètement ou en partie par ce projet. Environ 12 recueils par an sont preparés par le personnel attaché au projet. Deux synthèses par an sont écrites par des experts. Les comptes-rendus d'au moins deux conférences seront écrits dans une période de 3 ans. En résumé, l'objet de ce projet est de produire et disséminer entre 40 et 50 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, le 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 le personnel et les usagers. Finalement, des colloques annuels sont organisés pour les étudiants des pays en voie de développement qui étudient dans les universités americaines.

Foreword and Acknowledgments

This compendium is the third 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 the drainage of low-volume roads. 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 (AASHTO), Washington, D.C.;
American Society of Photogrammetry, Falls Church, Virginia;
Butterworth and Co. (Publishers) Ltd., London, England;
U.S. Army Engineers Waterways Experiment Station, Vicksburg, Mississippi;
U.S. Federal Highway Administration, Washington, D.C.

Prefacio y Agradecimientos

Este compendio es el tercer 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 le drenaje de caminos de bajo volúmen. Se pedirá a los corresponsales en los países en desarrollo información sobre los resultados, para utilizarse en el asesoramiento del grado al cuál se ha obtenido ese objetivo y para influenciar la naturaleza de productos subsequentes.

Se reconoce a los siguientes editores por el permiso dado para re-imprimir las porciones de texto seleccionadas de este compendio:

American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.;
American Society of Photogrammetry, Falls Church, Virginia;
Butterworth and Co. (Publishers) Ltd., London, England;
U.S. Army Engineers Waterways Experiment Station, Vicksburg, Mississippi;
U.S. Federal Highway Administration, Washington, D.C.

Avant-propos et Remerciements

Ce recueil représente le troisième 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 responsables du drainage des routes à faible capacité. 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 des éditeurs qui ont gracieusement donné leur permission de reproduire les textes sélectionnés pour ce recueil:

American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.;
American Society of Photogrammetry, Falls Church, Virginia;
Butterworth and Co. (Publishers) Ltd., London, England;
U.S. Army Engineers Waterways Experiment Station, Vicksburg, Mississippi;
U.S. Federal Highway Administration, 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). 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 Lynn N. Irwin, Cornell University, George W. Ring III, Federal Highway Administration, and Eldon J. Yoder, Purdue University, 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 de la U.S. Department of Transportation y el Library and Information Service de la U.K. Transport and Road Research Laboratory (TRRL). Las fotografías proveídas por TRRL fueron reproducidas bajo permisión de Her Majesty's Stationery Office.

Finalmente, el Transportation Research Board agradece el consejo y dirección valiosos provisto por el Comité de Iniciativas, con especial reconocimiento a los señores Lynn N. Irwin, Cornell University, George W. Ring III, Federal Highway Administration y Eldon J. Yoder, Purdue University, 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 Lynn N. Irwin, Cornell University, George W. Ring III, Federal Highway Administration et Eldon J. Yoder, Purdue University, qui ont bien voulu prêter leur assistance à la préparation de ce recueil.

Overview

Background and Scope

Adequate drainage must always be provided if a road is to be usable in all seasons. Roadway drainage begins with the removal of surface runoff from the roadway itself. Proper drainage design must also include (a) the removal of excess water from under the roadway, (b) the provision of roadside ditches of correct size, shape and longitudinal slope, (c) the prevention of side slope and ditch erosion, and (d) the passage of water flowing in all natural and manmade drainage channels without undue damage to the roadway itself.

Low-volume roads, as defined in Compendium 1, include Class 1 roads with an average daily traffic (ADT) volume of less than 50, and Class 2 roads with 50 to 400 ADT. Many features of low-volume rural roads can be upgraded as traffic volume increases. The capacity of a small drainage structure (i.e., a commonly used size of culvert pipe or its equivalent) is determined by its size and slope and cannot be upgraded economically. Furthermore, the volume of water to be passed by the structure is unrelated to the volume of traffic on the

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Vista General

Antecedentes y Alcance

Caminos que son utilizados durante todo el año deben estar proveídos con drenaje apropiado. El drenaje del camino comienza con la eliminación de agua de la superficie del camino. El diseño correcto de sistemas de drenaje debe incluir: (a) la eliminación de agua excedente por debajo del camino, (b) la provisión de zanjas laterales de tamaño, forma y pendiente longitudinal correctos, (c) la prevención de erosión de las laderas laterales y zanjas del camino, y (d) el flujo de agua através de canales de drenaje naturales y artificiales sin causar daño al camino mismo.

Los caminos de bajo volúmen, como se definen en el Compendio 1, son de Clase 1 con un volúmen de tránsito de menos de TMDA 50, y caminos de Clase 2 con un TMDA de 50 a 400. Muchas características de los caminos rurales de bajo volúmen pueden ser mejoradas a medida que aumenta el volúmen de tránsito. La capacidad de una pequeña estructura de drenaje (es decir, un tamaño común de tubería de alcantarilla, o su equivalente) se determina por su tamaño y pendiente y no puede ser economicamente mejorada. Además, el volúmen de agua a drenar no se relaciona con el volúmen de tránsito sobre el camino. Las pequeñas estructuras de drenaje deben ser medidas e instaladas correctamente como primer paso en el desarrollo del camino.

Exposé

Historique et Objectif

Pour qu'une route soit utilisable en toutes saisons, il faut qu'un dispositif de drainage convenable soit assuré. Le drainage d'une route commence avec l'évacuation des eaux de ruissellement de la surface de la route elle-même. La conception d'un bon dispositif de drainage doit aussi comprendre: a) l'évacuation de tout excès d'eau des couches inférieures de la chaussée b) un ensemble de fossés de drainage de dimensions, formes et pentes longitudinales adequates c) la prévention de l'érosion des pentes des talus et des fossés et d) l'écoulement des eaux d'origine soit naturelle, soit artificielle, de façon à ce que l'évacuation se fasse en causant le moins de dommage possible au corps de la chaussée.

Les routes à faible capacité, selon la definition du Recueil No. 1, comprennent les routes de la Classe 1 avec un trafic moyen journalier de moins de 50 (ADT) et les routes de la Classe 2 avec un trafic moyen journalier de 50 à 400 (ADT). Beaucoup de caractéristiques de routes à faible capacité peuvent être ameliorées au fur et à mesure que le volume de trafic s'accroît. La capacité d'un petit ouvrage de drainage, (c'est à dire une conduite ou canalisation d'une taille ordinaire) est determinée par ses dimensions et sa pente, et ne peut pas être agrandie de façon économique. De plus, le volume d'eau qui doit être évacué par cet ouvrage n'a rien à roadway. Small drainage structures should be sized and installed correctly as the first step in roadway development. The sizing should anticipate future changes in land use which may affect runoff.

Initial costs of drainage may be reduced by stage construction. The first step in the stage construction of the drainage system for a low-volume road is to provide a gravel or stabilized soil all-weather surface. This step includes (a) some means of keeping the road foundation dry, (b) proper cross slopes to drain the rainwater from the road surface, and (c) a passage under the roadway at locations where water would otherwise pond and flood the roadway surface. These steps provide a road surface that can be used throughout the year. If an all-weather surface is combined with dips or fords at larger waterway crossings, the road will be usable in all but heavy rainfalls.

The second step in the stage construction of the drainage system for a low-volume road is the replacement of dips and fords with all-weather crossings such as large culverts or bridges. Only then can the roadway be called an all-weather road.

This compendium presents information about general drainage design, but it stresses data that are necessary to design and install culverts. Culverts are normally classified as structures

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Las medidas deben anticipar futuros cambios en el uso del terreno que podrían afectar el drenaje del agua.

El costo inicial de drenaje puede ser reducido por construcción en etapas. El primer paso en la construcción por etapas del drenaje para caminos de bajo volúmen es proveer una superficie para toda intemperie de grava o tierra estabilizada. Esto incluye (a) una forma de mantener seco la fundación del camino, (b) laderas laterales correctas para desaguar el agua de lluvia de la superficie del camino, y (c) aberturas debajo del camino en ubicaciones donde de otra manera el agua recolectaría e inundaría la superficie del camino. Estas aberturas proporcionan una superficie de camino utilizable através del año. Si se combina una superficie de toda intemperie con depresiones o vados en las travesías mayores de vías de agua, el camino sería utilizable en toda menos que las tormentas de lluvia más pesadas.

El segundo paso en la construcción por etapas del camino de bajo volúmen es el reemplazo de las depresiones y vados con travesías de agua para toda intemperie tales como alcantarillas grandes o puentes. Solo entonces puede llamársele al camino uno de toda intemperie.

Este compendio presenta información sobre el diseño general de drenaje pero le dá importancia a los datos necesarios para diseñar e instalar alcantarillas. Las alcantarillas normalmente se clasifican como estructuras que miden

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voir avec le volume de trafic de la route. La première phase de la construction de la route consiste en dimensionant et en installant correctement les petits ouvrages d'art. Le dimensionnement doit être calculé en prévoyant ou en anticipant les changements futurs de l'utilisation du sol qui seraient susceptibles d'affecter le débit.

On peut réduire le coût initial des ouvrages de drainage en utilisant la méthode d'aménagement progressif aussi appelée méthode de mise en oeuvre à plusieurs couches. Le premier stade de la construction d'un système de drainage pour routes à faible capacité, est de pourvoir à une surface de roulement soit en gravier soit en terrain amélioré qui soit praticable en tous temps. Ce stade comprend: a) trouver le moyen de garder le corps de la chaussée sec, b) une pente transversale convenable pour faciliter le drainage de la surface de roulement, et c) un passage aménagé sous la route aux endroits où l'eau autrement s'accumulerait et inonderait la surface de roulement. Ces stades fournissent une surface de roulement qui peut être utilisée toute l'année. Si on combine une surface de roulement qui peut être utilisée par tous temps, avec la construction de radiers aux passages de rivières, on aura une route qui sera praticable par tous temps, sauf en cas de chûtes de pluies très intenses.

Le deuxième stade de la construction du

measuring up to 6 m (20 ft) along the roadway centerline. It is possible to use the top of a large box culvert as a part of the road surface, but most of the information presented here concerns culverts that are covered with embankment. Other drainage features, such as major water crossings, open channel design, and erosion control, will be presented in future compendiums.

Rationale for This Compendium

This compendium includes (a) requirements for proper roadway drainage, (b) solutions to general problems that are encountered in the design and construction of road drainage, and (c) identification of major components that must be considered in the design of the total drainage system required for a road. Small drainage structures are identified as a specific component requiring special and individual attention by the design engineer.

The first step in the design of a culvert is the hydrological analysis of the area to be drained. This analysis supplies the designer with information on runoff and stream flow characteristics and is the basis for the hydraulic design of the culvert.

Many of the working principles of hydrology were developed for large flood-control projects and water supply problems that involve extremely large watershed areas. Variations that are insignificant in large watershed areas become very important in small drainage areas. Engineering judgment and approximate methods must therefore be applied to the basic principles of hydrology when small drainage areas are

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hasta 6 m (20 piés) a lo largo de la línea central del camino. Es posible utilizar la parte superior de una alcantarilla de cajón como parte de la superficie del camino pero casi toda la información aquí presentada se concierne con las alcantarillas cubiertas por el terraplén. Otras características de drenaje tales como travesías principales de vías de agua, diseño de canal abierto y control de la erosión se presentarán en futuros compendios.

Exposición Razonada para Este Compendio

Este compendio incluye (a) los requisitos para un buén drenaje del camino, (b) las soluciones para los problemas generales que se encuentran en el diseño y construcción del drenaje del camino, y (c) la identificación de los componentes principales que se deben considerar en el diseño del sistema total de drenaje requerido para un camino. Las pequeñas estructuras de drenaje se identifican como un componente específico que requiere atención especial e individual por parte del ingeniero de diseño.

El primer paso en el diseño de una alcantarilla es el análisis hidrológico de la área a ser desaguado. Este análisis le proporciona al diseñador con la información sobre características de agua de desagüe y flujo de corriente de agua y es la base para el diseño hidráulico de la alcantarilla.

Muchos de los principios fundamentales de hidrología fueron desarrollados para grandes

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système de drainage, consiste à remplacer les radiers ou les gués par de grands ponceaux ou des ponts. C'est seulement à ce moment là, qu'on peut parler d'une route praticable en toutes saisons.

Ce recueil contient des renseignements sur le concept du drainage en général, mais il met l'emphase sur les données nécessaires au dimensionnement et à l'installation des ponceaux. Les ponceaux sont généralement des ouvrages d'art qui mesurent jusqu'a 6 m (20 pieds), installés le long de la ligne médiane de la route. Il est possible d'utiliser le haut d'un dalot comme partie intégrante de la chaussée, mais la plupart des données présentées ici concernent les ponceaux couverts par un remblai. D'autres ouvrages de drainage, tels que ceux utilisés pour le franchissement de cours d'eau importants, les fossés à ciel ouvert, et les ouvrages assurant le contrôle de l'érosion, seront traités dans de futurs recueils.

Objet de ce recueil

Ce recueil comprend: a) les conditions requises pour un système convenable de drainage des routes, b) la solution de problèmes généraux que l'on peut avoir lors du dimensionnement et de la construction des systèmes de drainage, et c) l'identification des composants les plus importants qui doivent être considérés lors de la conception de l'infrabeing analyzed. The design engineer must be aware that hydrological analysis is approximate and must expect that variable results will occur from time to time. The engineer should learn from these errors and then should use the additional knowledge in future analyses. Furthermore, no drainage system is designed to carry the maximum possible flow. The possibility of an occasional failure should be expected and accepted.

This compendium does not present a universal formula for the determination of runoff from drainage areas, since no "best" method for estimating runoff has been developed. Comparisons among the more commonly used formulas show that they all have shortcomings. Among the more notable drawbacks of these formulas are (a) lack of agreement with frequency curves developed from observed streamflow data, (b) inability to evaluate short downpours (thunderstorms) if the input is derived from widespread general storms, and (c) strong dependence on the judgment of the designer to estimate coefficients with arbitrary ranges that can double or quadruple the estimated runoff.

After the anticipated quantity of water to be passed has been determined, the culvert is designed according to the principles of hydraulics. The engineer must be able to

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proyectos de control de inundaciones y problemas de provisión de agua que involucran grandes áreas de cuencas. Las variaciones que son insignificantes en grandes áreas de cuencas se vuelven muy importantes en pequeñas áreas de drenaje. Por lo tanto al analizar áreas pequeñas de drenaje se deben aplicar juicios ingenieriles y métodos de aproximación a los principios básicos de hidrología. El ingeniero de diseño debe estar al tanto de que el análisis hidrológico es aproximado y debe anticipar que ocurrirán resultados inexactos de vez en cuando. El ingeniero debe aprender de estos errores y utilizar el conocimiento adicional en futuros análisis. Además, no se ha diseñado ningún sistema de drenaje para soportar el flujo máximo posible. Se debe esperar y aceptar que a veces habrá un fracaso.

No se presenta en este compendio un fórmula universal para la determinación del agua de desagüe en áreas de drenaje. Esto es porque todavía no se ha desarrollado un "mejor" método para estimar el desagüe. Una comparación entre las fòrmulas más utilizadas muestra que todas tienen varias faltas. Entre las desventajas más notables de estas fórmulas hay (a) una falta de acuerdo con las curvas de frecuencia desarrolladas de datos observados de flujo de agua, (b) una inhabilidad para evaluar aguaceros cortos (tormentas eléctricas) si los datos se derivan de tormentas generales diseminadas, y (c) una fuerte dependencia sobre el juicio del diseñador para estimar los coeficientes que tienen alcances arbitrarios que pueden duplicar ó cuadruplicar el agua de desagüe estimado.

Después de que se haya determinado la cantidad de agua que pasará, se diseña la

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structure de drainage. Les petits ouvrages de drainage sont identifiés comme un composant spécifique qui requiert l'attention particulière de l'ingénieur.

La première phase du dimensionnement d'un ponceau consiste à faire l'analyse hydrologique de la surface à drainer. Cette analyse fournit les données sur le débit et les caractéristiques de l'écoulement du cours d'eau, et forme la base sur laquelle on calcule le dimensionnement hydraulique de l'ouvrage.

La plupart des principes de l'hydrologie ont été développé pour les grands barrages et les problèmes d'alimentation en eau qui comportent des bassins versants très étendus. Des variations, qui sont insignifiantes dans les bassins versants très étendus, deviènnent très importantes quand il s'agit de drainer des étendues plus modestes. Le jugement proféssionel de l'ingénieur, et l'utilisation de procédés approximatifs, doivent intervenir et être appliqués aux principes de base de l'hydrologie quand il s'agit d'analyser le drainage de petites surfaces. L'ingénieur donc, doit se souvenir que l'analyse hydrologique est approximative, et qu'il doit s'attendre à obtenir, de temps à autre, des résultats variables.

Il devra se rappeler qu'"à quelque chose malheur est bon" et pourra appliquer les connaissances acquises de cette façon au calcul de futurs ouvrages. De plus, aucun système de drainage n'est conçu pour évacuer un débit maximum. Il faut s'attendre à la possibilité d'un fiasco occasionnel et l'accepter.

identify the conditions under which the culvert will operate. The Selected Texts include nomographs that permit the engineer to choose proper culvert sizes for the appropriate conditions.

The engineer should evaluate the performance of each culvert over a range of flow values. Performance curves will aid in the selection of the culvert size, shape, material, and inlet geometry that meet site requirements at the lowest annual cost. The curves may show opportunities for increasing the factor of safety and for improving hydraulic capacity at little or no increase in cost. The designer should consider the use of multiple culverts. The reduced size of the individual pipes may permit their installation without the use of heavy equipment. However, the designer must recognize that culverts smaller than 1 m (3.3 ft) in diameter are difficult to clean and repair.

A culvert is designed as a part of a continuous channel. It should alter the natural flow conditions as little as possible. If the grade of the culvert is flatter than the channel, the culvert inlet may fill with sediment. If the grade of the culvert is steeper than the channel grade, the culvert outlet may cause erosion or fill with sediment. If the culvert is too small to pass debris carried by the water course, the entrance area may be obstructed. If the culvert alters the direction of stream flow, erosion can occur at either the entrance or the exit of the culvert. Solutions of these problems will directly reduce maintenance costs.

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alcantarilla de acuerdo con principios hidráulicos. El ingeniero debe tener la habilidad de identificar las condiciones bajo las cuales operará la alcantarilla. Los Textos Seleccionados incluyen nomografías que le permiten al ingeniero escoger los tamaños correctos de alcantarilla para las condiciones apropiadas.

El ingeniero debe evaluar el funcionamiento de cada alcantarilla sobre un rango de valores de flujo. Curvas de rendimiento ayudarán en la selección del tamaño, forma, y material de la alcantarilla y la geometría de la boca de entrada que satisfacen los requisitos del sitio al menor costo anual. Las curvas podrán indicar oportunidades para aumentar el factor de seguridad y para mejorar la capacidad hidráulica con poco o ningún aumento del costo. El diseñador debería considerar el uso de alcantarillas múltiples. El tamaño reducido de los tubos individuales permitirían su instalación sin equipo pesado. Sin embargo, el diseñador deberá reconocer que las alcantarillas con un diámetro de menos de un metro son difíciles de limpiar ó reparar.

Ce recueil n'a pas la prétention de donner une formule universelle pour calculer le débit des surfaces à drainer, puisqu'aussi bien, il n'éxiste pas de méthode qui soit "la meilleure" Une comparaison des formules généralement utilisées, montre qu'elles ont toutes des défauts. Parmi leurs inconvénients les plus insignes, on peut compter: a) un manque de conformité des courbes de fréquence developpées en analysant des données de l'écoulement du cours d'eau b) l'incapacité d'évaluer les averses locales (orages) si les données sont derivées d'analyses de tempêtes généralisées et c) le fait que ces formules dépendent en grande partie du jugement de l'ingénieur pour estimer les coéfficients variables qui peuvent doubler ou quadrupler l'évaluation du débit.

Après que la quantité d'eau à évacuer est determinée, l'ouvrage est dimensionné selon les principes de l'hydraulique. L'ingénieur doit être capable d'identifier les conditions sous lesquelles le ponceau doit opérer. Les Textes Choisis comprennent des abaques qui permettront à l'ingénieur de calculer les dimensions convenables de l'ouvrage par rapport aux conditions d'utilisation.

L'ingénieur devrait évaluer le rendement de chaque ponceau pour des débits variables. Les courbes de rendement l'aideront pour faire le choix des dimensions de l'ouvrage, de sa forme, du matériau dont il est construit, et de la géométrie du canal d'amenée, choix qui doit réunir à la fois toutes les conditions requises, et un coût annuel peu onéreux. Ces courbes peuvent lui indiquer les moyens d'augmenter le facteur de sécurité, et d'améliorer la capacité hydraulique, pour un prix équivalent ou légèrement plus élevé. L'ingénieur devrait prendre en considération la construction de ponceaux multiples. Les dimensions réduites

A culvert must be able to withstand the structural load placed upon it by the embankment that covers it and by construction machinery and traffic that pass over it. A design method for determining the allowable structural load for corrugated metal pipe is given. Corrugated metal pipe is quite useful in the construction of drainage structures for low-volume rural roads because it is (a) quite light, (b) less subject to damage by handling, and (c) easy to assemble by unskilled or semiskilled laborers.

Criteria for the structural design and installation of reinforced concrete pipe culverts are also included. Concrete pipe culverts may in some cases cost less than metal pipe because they can be fabricated on the site with local materials, and they may be more resistant to some chemicals. The ability of a concrete pipe to withstand structural loading is a function of the strength of the pipe wall. The strength of the pipe wall is in turn a function of the control of the manufacturing process. This control can vary greatly, especially during on-site pipe construction. Any tabulation of allowable embankment heights for various thicknesses of concrete pipe walls, with or without reinforcement, might not apply to concrete pipe culverts made on-site.

Una alcantarilla está diseñada como parte de un canal contínuo. Deberá cambiar lo menos posible las condiciones naturales de flujo. Si el pendiente de la alcantarilla es menos que el del canal, la boca de la alcantarilla puede llenarse de sedimento. Si el pendiente es más que el del canal, la salida de la alcantarilla puede causar erosión ó llenarse con sedimento. Si la alcantarilla es demasiada pequeña para permitir el paso de desechos llevados por el agua, la área de entrada puede obstruirse. Si la alcantarilla cambia la dirección de flujo del agua, puede ocurrir erosión en su entrada o salida. Las soluciones a estos problemas directamente reducen el costo de mantenimiento.

Una alcantarilla deberá poder resistir el peso estructural colocado sobre ella por el terraplén y la carga de la maquinaria de construcción y tránsito que pasa. Se dá un método de diseño para determinar el peso estructural permisible para tubería de metal corrugado. La tubería de metal corrugado es muy útil en la construcción de estructuras de drenaje para caminos de bajo volúmen porque es (a) bastante liviano, (b) no tan sujeto a daño al manipularla, y (c) fácil de armar por los obreros no especializados.

También se incluyen criterios para el diseño estructural e instalación de Alcantarillas de Tubo de Hormigón Reforzado. Las alcantarillas de tubo de hormigón pueden (a) costar menos, (b) estar más disponibles, (c) ser fabricados en *situ* de materiales locales, y (d) ser más resistentes a algunos químicos. La capacidad de un tubo de hormigón de resistir cargas estructurales es una función del aguante de la pared de hormigón del tubo. A su vez la resistencia ó aguante de la pared de hormigón del tubo es una función del control del proceso

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des canalisations peuvent permettre leur installation, sans l'utilisation de matériel lourd. Cependant, l'ingénieur doit se souvenir que les buses ou dalots d'un diamètre inférieur à 1 m (3.3 pieds), sont difficiles à nettoyer et à entretenir.

Un ponceau est conçu comme faisant partie intégrante d'un cours d'eau ininterrompu et il devrait modifier le moins possible les conditions naturelles d'écoulement du cours d'eau. Si la pente longitudinale du ponceau est moindre que celle du cours d'eau, l'amenée du ponceau peut se remplir de sédiments. Si, au contraire, la pente du ponceau est plus grande, l'exutoire risque de provoquer l'érosion, ou de se remplir de sédiments. Si le ponceau est trop petit pour laisser passer les débris flottants, son canal d'amenée peut être obstrué. Si le ponceau change la direction du cours d'eau, l'érosion peut être provoquée, soit à son amenée, soit à son exutoire. La bonne solution de ces problèmes réduit immédiatement les frais d'entretien. Un ponceau doit être capable de subir la change structurale du remblai qui le recouvre, la charge du matériel de construction, et celle du trafic routier. Une méthode pour le calcul de la charge structurale admissible pour les buses métalliques en tôle ondulée est donnée. Les canalisations en tôle ondulée sont très indiquées pour la construction des ouvrages d'art des routes à faible capacité parce qu'elles sont: a) très légères, b) moins sujettes à être endommagées par manipulation et c) sont faciles à assembler par la main d'oeuvre non spécialisée.

Des critères pour le calcul et l'installation des buses, ou dalots, en béton armé, sont aussi inclus. Les canalisations en béton, peuvent, en certains cas, coûter moins que celles en métal, car elles peuvent être coulées sur place

Discussion of Selected Texts

The first text, ***Chapter 5, Road Drainage*** from ***Low Cost Roads; Design, Construction and Maintenance*** (UNESCO,1967; translated into English, 1971), is reprinted in full. The first part of this chapter discusses roadway drainage, control of erosion, and the stability of embankments and cuttings. Recommendations for the drainage of the road structure are given. Defensive measures to prevent delays and inefficient construction operation during wet weather are noted. The second part of the chapter discusses location and waterway requirements for bridges and culverts.

The second text, ***Guidelines for the Hydraulic Design of Culverts*** (AASHO, 1975), is reprinted in full. Comprehensive guidelines are presented for the hydraulic aspects of culvert design. The function of a culvert is to convey surface water under or from the road. In addition to this hydraulic function, a culvert must also carry construction and highway traffic and earth loads. Therefore, culvert design involves both hydraulic and structural design. This text refers to the structural aspects of culvert

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de manufactura. Este control puede variar mucho, especialmente en la construcción en *situ* de la tubería. Cualquier tabulación de alturas permisibles de terraplén para varios espesores de paredes de tubería de hormigón, con ó sin refuerzo, quizás no sean aplicables a las alcantarillas de tubo de hormigón hechas en *situ*.

Presentación de los Textos Seleccionados

El primer texto es ***Chapter 5, Road Drainage*** (capítulo 5, Drenaje de caminos) de ***Low-Cost Roads, Design, Construction and Maintenance*** (Caminos de bajo costo, su diseño, construcción y manutención), (UNESCO 1967; traducido al inglés, 1971). Se reproduce el capítulo totalmente.

La primera parte de este capítulo trata sobre el drenaje del camino, control de la erosión, y la estabilidad de terraplenes y cortes. Se dán recomendaciones para el drenaje de la estructura del camino. Se señalan medidas de prevención para demoras y operaciones ineficientes de construcción durante temporadas de lluvia. La segunda parte del capítulo trata sobre ubicación y requerimientos de vías de agua para puentes y alcantarillas.

El segundo texto ***Guidelines for the Hydraulic Design of Culverts*** (Pautas para el diseño hidráulico de alcantarillas (AASHO, 1975) se reproduce totalmente.

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avec des matériaux locaux, et elles peuvent être plus résistantes à certains agents chimiques. La capacité d'une conduite en béton à subir une charge structurale, est fonction de la résistance de ses parois. La résistance des parois de la conduite, est, à son tour, fonction du contrôle de sa manufacture. Ce contrôle peut varier beaucoup, spécialement pour les tuyaux fabriqués sur place. Les tables qui indiquent les hauteurs de remblai admissibles pour différentes épaisseurs de parois de tuyaux en béton, armé ou non, ne sont pas nécessairement applicables aux tuyaux en béton fabriqués sur place.

Discussion des Textes Choisis

Le premier texte, chapitre 5, le drainage, (Road Drainage) du livre ***Low Cost Roads, Design Construction and Maintenance,*** (Routes dans les pays en voie de développement, conception, construction, entretien) publié par l'Unesco en 1967 et traduit en anglais en 1971. Ce chapitre est reproduit en entier.

Le début du chapitre discute du drainage de la route, de la protection contre l'érosion, et de la stabilité des remblais et des déblais. Des conseils sur le drainage des couches de la chaussée, et sur les précautions à prendre si on veut éviter les dégâts et les retards dûs à la pluie pendant la construction, sont inclus. Le reste du chapitre concerne l'emplacement des ouvrages et les infrastructures.

Le deuxième texte, ***Guidelines for the Hydraulic design of Culverts (Guide pour le dimensionnement hydraulique des ponceaux)*** (AASHO, 1975) est reproduit en entier.

Un guide complet du dimensionnement

design only as they are related to the hydraulic design of culverts.

The text indicates that the cost of individual culverts is usually relatively small, but that the total cost of culvert construction can constitute a substantial share of the total construction costs of a low-volume rural road. The total cost of properly maintaining highway drainage systems is substantial. Culvert maintenance can account for a large share of these costs. Improved traffic service and a sizable reduction in the total cost of road construction and maintenance can be gained by a reasonable choice of design criteria and careful attention to the hydraulic design of each culvert.

The third text, ***Drainage Studies from Aerial Surveys,*** was published in ***Photogrammetric Engineering*** in September 1961. Compendium 2 introduced the concept of using aerial photographs as an engineering tool. This text further describes the use of stereoscopic viewing of aerial photographs for drainage design. It discusses the use of aerial photographs to determine drainage areas. It includes an illustration of the method used to correct the plotting of drainage areas taken stereoscopically from aerial photographs of terrain with major differences in elevation.

Se presentan pautas comprensivas para los aspectos hidráulicos del diseño de alcantarillas. La función de una alcantarilla es la de trasladar agua de desagüe a través ó desde el camino. Además de esta función hidráulica una alcantarilla también deberá soportar tránsito de construcción y vial y cargas de tierra. Por lo tanto el diseño de alcantarillas involucra diseño hidráulico y estructural. Este texto se refiere a los aspectos estructurales del diseño de alcantarillas únicamente como se relacionan al diseño hidráulico.

El texto indica que el costo de alcantarillas individuales generalmente es relativamente poco pero que el costo total de su construcción puede constituir una parte importante de los costos totales de construcción de un camino de bajo volúmen. También es grande el costo total de una correcta manutención de los sistemas viales de desagüe. La manutención de las alcantarillas forman una gran parte de estos costos. Se puede obtener un mejor servicio de tránsito y una considerable reducción en el costo total de construcción y manutención del camino por una selección razonable de criterios de diseño y especial atención al diseño hidráulico de cada alcantarilla.

El tercer texto, *Drainage Studies from Aerial Surveys* (Estudios de drenaje de reconocimientos aéreos), fué publicado en *Photogrammetric Engineering* (Ingeniería Fotogramétrica) en septiembre de 1961. El Compendio 2

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hydraulique des ponceaux est présenté. La fonction d'un ponceau est de conduire les eaux de ruissellement sous la route, ou au-delà de la route. En outre de cette fonction hydraulique, un ponceau doit aussi supporter les charges du matériel de construction, du trafic routier et des remblais de terre. Le dimensionnement des ponceaux comprend donc le dimensionnement hydraulique et le calcul des ouvrages. Le texte s'adresse au calcul des ouvrages seulement quand celui-ci se rapporte au dimensionnement hydraulique.

Le livre indique que le coût de chaque ponceau en lui-même, est relativement peu élevé, mais le coût total de la construction de l'ensemble des ponceaux constitue une part substantielle du coût total de la construction d'une route à faible capacité. Le coût total de l'entretien d'un dispositif de drainage est élevé. L'entretien des ponceaux y compte pour une large part. Un choix raisonnable de critères pour le calcul des ponceaux et leur dimensionnement hydraulique, résultera en une réduction notable du prix de revient total de la construction et de l'entretien de la route, et en une amélioration de la circulation routière.

Le troisième texte, *Drainage Studies from Aerial Surveys* (Etudes de drainage à l'aide de levés aériens) est extrait de *Photogrammetric Engineering,* de Septembre, 1961—le recueil numéro 2 de notre série a déjà introduit le concept de l'utilisation des photos aériennes, Various other methods of determining drainage areas are discussed. Comparisons are made of the amount of labor required and of the accuracy of each method. Other drainage data obtainable from aerial photographs are listed.

This text describes methods for using aerial photographs to position culverts. This technique requires the use of large-scale (1:3000) photographs that would not normally be available in rural areas requiring low-volume roads.

The fourth text, ***Hydraulic Charts for the Selection of Highway Culverts,*** was issued by the U.S. Department of Transportation as Hydraulic Engineering Circular No. 5. This publication has been reproduced in full from the April 1977 reprint of the original 1965 publication. This text is also available in Spanish (see Bibliography).

It assumes that the engineer has determined the quantity of water to be passed through the culvert, and discusses culverts flowing with both inlet and outlet control. It explains the hydraulics of culverts flowing partially full and with various depths of headwater and tailwater. Variations of culvert capacity due to different inlet shapes are also discussed. The text includes a series of nomographs for use in the design of culverts; explanations are given for using the nomographs to select culvert size.

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presentó el concepto del uso de fotografías aéreas como una herramienta ingenieril. Este texto lo amplía con su descripción de la observación estereoscópica de fotografías aéreas para el diseño de desagües.

Habla sobre el uso de fotografías aéreas para la determinación de áreas de drenaje. Incluye una ilustración del método utilizado para corregir la diagramación de áreas de drenaje tomadas estereoscopicamente de fotografías aéreas de terrenos con grandes diferencias de elevación.

Presenta varios otros métodos para la determinación de áreas de drenaje. Se hacen comparaciones de la cantidad de trabajo requerido y de la precisión de cada método. También se presentan otros datos de drenaje que son obtenibles de fotografías aéreas.

El texto describe métodos de utilización de fotografías aéreas para situar alcantarillas. Esta técnica requiere el uso de fotografías en gran escala (1:3000) que no son normalmente obtenibles en áreas rurales que requieren caminos de bajo volúmen.

El cuarto texto, ***Hydraulic Charts for the Selection of Highway Culverts*** (Mapas hidráulicas para la selección de alcantarillas viales), fué publicado por el U.S. Department of Transportation como Engineering Circular No. 5 (Circular No. 5 de Ingeniería Hidráulica.) Esta publicación ha sido reproducida en toto de la reimpresión de abril 1977 de la publicación original de 1965. Este texto también se puede obtener en español (ver Bibliografía).

Presume que el ingeniero ha determinado el volúmen de agua que pasará por la alcan-

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et ce texte décrit de plus l'emploi du stéréoscope avec ces photos aériennes pour l'étude du système de drainage.

Les photos aériennes sont utilisées pour décider la localisation du drainage. Un exemple de la méthode employée pour corriger le tracé des zones de drainage determinées stéréoscopiquement d'après des photos aériennes de terrains qui ont de grandes différences d'élévation, est inclus.

Plusieurs autres procédés pour déterminer les zones de drainage sont discutés. On compare la valeur du temps nécessaire pour la complétion des travaux et la précision de chaque méthode. D'autres données sur le drainage, obtenues d'après les photos aériennes, sont présentées.

Ce texte décrit aussi différentes façons de situer les ponceaux en utilisant les photos aériennes. Cependant, cette technique éxige des photos à grande échelle (1:3000) qui ne sont pas normalement disponibles dans les régions rurales ayant besoin de routes à faible capacité.

La quatrième publication, ***Hydraulic Charts for the Selection of Highway Culverts*** (Graphiques hydrauliques pour la sélection des ponceaux) a été publiée par le U.S. Department of Transportation sous le titre ***Hydraulic Engineering Circular No. 5*** (Circulaire de travaux hydrauliques No. 5). Ce texte, reproduit ici en entier, est celui de la réimpression en Avril 1977, de la circulaire originale publiée en 1965. Cette circulaire est aussi publiée en espagnol (Voir la bibliographie).

Il est supposé que l'ingénieur a déterminé

The same organization also published *Capacity Charts for the Hydraulic Design of Highway Culverts,* Hydraulic Engineering Circular No. 10 (see Bibliography). These charts permit the direct selection of a culvert size without making detailed computations, but do not replace the nomographs in the selected text. The charts are not as comprehensive as the nomographs, nor do they cover as wide a range of conditions as are presented in the selected text.

The fifth text is a reproduction of *Debris-Control Structures,* Hydraulic Engineering Circular No. 9, issued by the U.S. Department of Transportation in March 1971. It discusses water-borne debris problems and structures used for controlling that debris. An accumulation of debris at inlets of highway drainage structures is a frequent cause of unsatisfactory performance or malfunction. This is especially true in low-volume rural roads where maintenance of waterways is neglected because of money limitations.

The text describes three methods of controlling debris. It lists the advantages of debris-control structures and the various classifications of debris. A guide is included for selecting the type of structures that are suitable for various debris classifications. Photographs of various debris-control structures and design drawings for some of the most common structures are included.

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tarilla, y habla sobre alcantarillas con control de flujo en su boca de entrada y de salida. Explica la hidráulica de las alcantarillas fluyendo parcialmente llenas y con varias profundidas de agua de cabecera y cola. También trata con las variaciones de la capacidad de alcantarilla debidas a distintas formas de boca de entrada.

El texto incluye una serie de nomografías para uso en el diseño de alcantarillas; se dán explicaciones para utilizar las nomografías en la selección de tamaño de alcantarilla.

La misma organización también publica *Capacity Charts for the Hydraulic Design of Highway Culverts* (Diagramas de capacidad para el diseño hidráulico de alcantarillas viales) (Circular No. 10 de Ingeniería Hidráulica (Ver Bibliografía). Estos diagramas permiten una selección directa de tamaño de alcantarilla sin hacer computaciones detalladas, pero no reemplazan las nomografías en el Texto Seleccionado. Los diagramas no son tan comprensivas como las nomografías ni abarcan tan gran variedad de condiciones como en el Texto Seleccionado.

El quinto texto es una reproducción de *Debris-Control Structures* (Estructuras para el control de desechos), Hydraulic Engineering Circular No. 9 de Ingeniería Hidráulica, publicada por el U.S. Department of Transportation en marzo de 1971.

Habla sobre los problemas causados por desechos llevados por el agua, y las estructuras que se utilizan para controlar esos desechos. Repetidas veces una acumulación de desechos en las bocas de entrada de estructuras de drenaje es la causa de rendimiento insatisfactorio ó malfuncionamiento. Esto es especialmente cierto en los caminos rurales de bajo volúmen donde el mantenimiento de

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le volume d'eau qui doit être évacué, et les ponceaux controlés à l'amenée et à l'exutoire sont discutés. L'hydraulique des ponceaux, partiellement pleins, et avec des niveaux d'eau de différentes profondeurs en amont et en aval, est expliquée. Les différences de capacité des ponceaux, attribuées aux variations des canaux d'amenée, sont aussi discutées. De plus, une série d'abaque pour le calcul des ponceaux, avec des explications sur leur utilisation pour déterminer les dimensions de ceux-ci, est incluse.

Le même organisme a publié *Capacity Charts for the Hydraulic Design of Highway Culverts, Hydraulic Engineering Circular No. 10* (Cartes de capacité pour le dimensionnement hydraulique des ponceaux, Circulaire de travaux hydrauliques No. 10. (voir bibliographie). Ces cartes permettent de choisir la taille des ponceaux sans se livrer à des calculs détaillés, mais elles ne remplacent pas les abaques mentionnées dans les Textes Choisis. Les cartes ne sont pas aussi complètes que les abaques, et ne s'appliquent pas à une aussi grande gamme de conditions.

Le cinquième texte est reproduit de *Debris-Control Structures, Hydraulic Engineering Circular No. 9* (Ouvrages d'art pour le contrôle des corps flottants, Circulaire de travaux hydrauliques No. 9) publié par le U.S. Department of Transportation en mars 1971. Il discute des problèmes posés par les corps flottants, et les ouvrages d'art nécessaires pour les contrôler. L'accumulation de corps flottants à l'amenée des ouvrages d'art cause fréquemment leur mauvais fonctionnement ou, tout du moins, The sixth text consists of excerpts from a report, ***Practical Guidance for Design of Lined Channel Expansions at Culvert Outlets,*** published by the Hydraulics Laboratory of the U.S. Army Engineers Waterways Experiment Station in October 1974. The excerpts summarize and demonstrate the application of research results to the design of lined channel expansions at culvert outlets. Empirical equations and charts are presented for estimating anticipated localized scour at culvert outlets. The size and shape of revetments and energy dissipators to control localized scour are described.

The design engineer can select appropriate and alternative schemes of protection for controlling erosion at culvert outlets using the data included in this text.

The seventh text is a reproduction of ***Corrugated Metal Pipe Culverts; Structural Design Criteria and Recommended Installation Practices,*** published by the U.S. Department of Commerce, Bureau of Public Roads, in 1966. A design method is given for determining the allowable structural load for corrugated steel and corrugated aluminum pipe culverts. Included are pipes of riveted, resistance spot-welded, helical, and bolted fabrication. The design charts provide for a rapid determination of the maximum allowable fill height for given pipe diameters.

The text also gives recommended installation

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vías de agua es descuidado por limitaciones monetarias.

El texto describe tres métodos para el control de desechos. Numera las ventajas de estructuras para el control de desechos y las varias clasificaciones de desechos. Se incluye una guía para seleccionar el tipo de estructura apropiado para las varias clasificaciones de desechos. Se incluyen fotografías de varias de las estructuras y dibujos de diseño de algunas de las estructuras más comunes.

El sexto texto consiste en extractos de un informe titulado *Practical Guidance for Design of Lined Channel Expansions at Culvert Outlets* (Una guía práctica para el diseño de extensiones de canal revestidas en bocas de salida de alcantarillas), publicado por el Hydraulics Laboratory of the U.S. Army Engineers Waterways Experiment Station en octubre de 1974.

La parte seleccionada resume y demuestra la aplicación de resultados de investigación al diseño de extensiones de canal revestidas en bocas de salida de alcantarillas. Se presentan ecuaciones y diagramas empíricas para estimar el estrego situado en las bocas de salida. Se describen el tamaño y forma de revestimiento y disipadores de energía para controlar el estrego.

El ingeniero de diseño puede seleccionar planes apropiados y alternativos de protección para el control de la erosión en las bocas de salida de alcantarillas utilizando los datos incluídos en este texto.

El séptimo texto es una reproducción de *Corrugated Metal Pipe; Structural Design Criteria and Recommended Installation Practice* (Tubería de metal corrugado; criterios de diseño y prácticas recomendadas de instalación), publicado por el U.S. Department

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un fonctionnement qui ne donne pas complète satisfaction. Ceci est particulièrement vrai des routes rurales à faible capacité, où l'entretien des cours d'eau est négligé par manque de fonds.

Le texte décrit trois méthodes de contrôle des corps flottants. Il énumère les qualités des différents ouvrages d'art, et les diverses classifications des corps flottants. Un guide pour la sélection des ouvrages d'art, selon la classification des corps flottants, est presenté. Des photographies de divers ouvrages d'art, et les plans des plus communs sont inclus.

Le sixième texte consiste en des extraits du rapport *Practical Guidance for the Design of Lined Channel Expansions at Culvert Outlets* (Guide pratique pour le dimensionnement des ouvrages d'extrémité) publié par le Hydraulics Laboratory of the U.S. Army Engineers Waterways Experiment Station, en Octobre 1974.

Les extraits résument et démontrent comment appliquer les résultats de recherches au dimensionnement des ouvrages d'extrémité. Des équations empiriques, des graphiques et des tables sont présentés pour évaluer l'effet érosif anticipé à l'exutoire des buses et des dalots. Les dimensions et formes des revêtements et ouvrages qui dissipent l'énergie et contrôlent l'afouillement local sont décrites.

En utilisant les données incluses dans ces extraits, l'ingénieur peut faire le choix entre plusieurs méthodes de protection contre l'erosion des exutoires.

Le septième texte choisi est reproduit de *Corrugated Metal Pipe Culverts; Structural Design Criteria and Recommended Installation Practices* (Buses en métal ondulé; critères pour le calcul des ouvrages et recommandapractices. These practices ensure that the flexible pipe will perform structurally as designed.

The eighth text is a reproduction of ***Reinforced Concrete Pipe Culverts; Criteria for Structural Design and Installation,*** published by the U.S. Department of Commerce, Bureau of Public Roads, in 1963. It discusses and defines the factors affecting the strength of rigid types of pipe. It presents formulas for the determination of loads on pipes under various types of embankment construction. A method for evaluating the different classes of pipe and bedding required for various heights of fill is described. Charts are included to simplify the design procedures. Also, a section on the installation of concrete pipe discusses (a) construction of the bedding, (b) laying the pipe, and (c) backfilling around and over the pipe.

Bibliography

The Selected Texts are followed by a brief bibliography containing reference data and abstracts for 20 publications. The first eight describe the Selected Texts. The other 12 describe publications that are closely related to the Selected Texts.

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of Commerce, Bureau of Public Roads, en 1966.

Se dá un método de diseño para determinar la carga estructural permisible para alcantarillas de tubo de acero y aluminio corrugado. Están incluídos tubería remachada, soldada electricamente por puntos de resistencia, helicoidal, y empernada. Los diagramas de diseño permiten una rápida determinación de la altura máxima permisible de terraplén para los diámetros dados de tubería.

El texto también dá las prácticas recomendadas de instalación. Estas prácticas aseguran el cumplimiento estructural de diseño del tubo flexible.

El octavo texto es una reproducción de *Reinforced Concrete Pipe Culverts: Criteria for Structural Design and Installation* (Alcantarillas de tubo de hormigón reforzado: criterios para su diseño estructural e instalación), publicado por el U.S. Department of Commerce, Bureau of Public Roads en 1963. Trata sobre y define los factores que afectan la resistencia de varios tipos de tubería rígida. Presenta fórmulas para la determinación de cargas sobre los tubos bajo varios tipos de construcción de terraplén. Se describe un método para evaluar los distintos tipos de tubería y fundamento que se requieren para varias alturas de terraplén. Se incluyen diagramas para simplificar los procedimientos de diseño.

Una sección sobre la instalación de tubería de hormigón abarca (a) la construcción del fundamento, (b) la colocación de la tubería y (c) rellenando alrededor y sobre la tubería.

Bibliografía

Después de los Textos Seleccionados hay una breve bibliografía que contiene datos de referencia y abstractos para 20 publicaciones. Los primeros 8 describen los Textos Seleccio-

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tions pratiques pour leur installation), publié par le U.S. Department of Commerce, Bureau of Public Roads, en 1966.

Une méthode de calcul des charges structurales admissibles pour les conduites en tôle ou en aluminium ondulé est presentée. Les conduites soudées par points, rivées, hélicoïdales, et boulonnées sont incluses. Des graphiques et des tables permettent de calculer rapidement la hauteur maximale admissible des remblais selon le diamètre des conduites.

Ce texte donne aussi des recommandations pour l'installation des conduites. Ces méthodes garantissent le fonctionnement structural des conduites flexibles.

Le huitième texte, reproduit de *Reinforced Concrete Pipe Culverts; Criteria for Structural Design and Installation* (Buses en béton armé; critères pour le calcul des ouvrages et leur installation), publié par le U.S. Department of Commerce, Bureau of Public Roads, 1963. Les facteurs qui influencent la résistance des conduites rigides sont définis et discutés. Des formules pour déterminer les charges auxquelles sont soumises les conduites selon differents types de remblais sont presentées. Une méthode pour déterminer les différentes sortes de tuyaux et de berceaux nécessaires pour différentes hauteurs de remblais est proposée. Des tables sont incluses pour simplifier le calcul. Un chapitre sur l'installation des conduites en béton discute de: a) la construction du berceau, b) l'installation des tuyaux et c) le remblai qui entoure les tuyaux.

Bibliographie

Les Textes Choisis sont suivis d'une bréve bibliographie contenant les référence et les résumés de 20 publications. Les huit premiers

Although there are many other articles, reports, and 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 text has been selected or basic publications that would have been selected had there been no limit on the number of pages in this compendium.

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nados. Los otros 12 describen publicaciones que se relacionan intimamente con los Textos Seleccionados.

Aunque hay muchos artículos, informes, y libros que podrían haber sido nombrados en la bibliografía, no es el propósito de éste contener todas las referencias posibles sobre el tema. La bibliografía contiene únicamente aquellas publicaciones de las cuales se seleccionó el texto ó publicaciones básicas que se hubieran seleccionado si no hubiera límite al número de páginas en este compendio.

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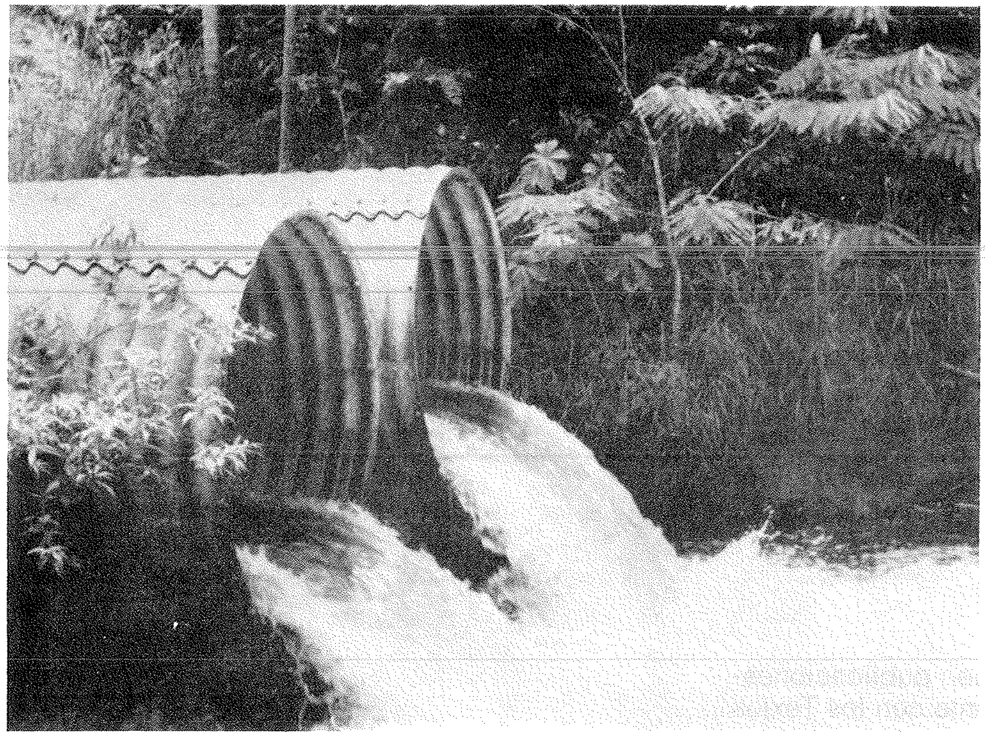
se rapportent aux Textes Choisis. Les autres douze à des publications qui sont étroitement apparentées aux Textes Choisis.

Bien qu'il y ait beaucoup d'autres articles, rapports et livres, qui pourraient être inclus, l'objectif de cette bibliographie n'est pas d'énumérer toutes les références possibles ayant rapport au sujet de ce recueil. Donc, notre bibliographie, telle quelle, se rapporte seulement aux publications dont nous avons sélectionné des extraits, ou aux textes de base que nous aurions choisi aussi s'il n'y avait pas de limite quant au nombre de pages de ce recueil.



Pictured is outlet of dual 1.8-m bolted Multi-Plate culverts—Brazil.

Selected Texts

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.

Page numbers of the original text appear inside the frames. Page numbers for the compendium are outside the frames and appear in the middle left or middle right outside margins of the pages. Page numbers that are given in the Table of Contents and in the Index refer to the compendium page numbers.

Each text begins with one or more pages of introductory material that was contained in the original publication. This material generally includes a title page, or a table of contents, or both. Asterisks that have been added to original tables of contents have the following meanings:

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

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Cette partie du recueil contient les pages sélectionnées de chaque texte qui est énuméré dans la Table des Matières. Les pages du texte original qui sont reproduites sont entourées d'un encadrement rectangulaire. Certaines pages ont dû être réduites pour pouvoir être placées dans l'encadrement. Le texte original n'a pas été changé sauf pour quelques explications qui ont été insérées. Donc, si le texte original contient des erreurs, elles sont reproduites dans le recueil.

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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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Por lo tanto, los textos seleccionados unicamente incluyen aquellas partes de los documentos originales que están precedidas por asteriscos en el índice de las publicaciones respectivas.

Líneas de guiones cruzando cualquier página del texto seleccionado significan que en ese lugar se ha omitido texto original. En varios lugares el texto seleccionado contiene notas aclaradoras que han sido introducidas por el personal del proyecto. Tales notas están insertadas en recuadros de guiones y comienzan con la palabra NOTE.

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dans cet extrait du document original sont 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 le personnel attaché à ce projet. Ces explications sont entourées d'un encadrement en pointillé et commencent toujours par le mot NOTE.

LOW COST ROADS

DESIGN, CONSTRUCTION AND MAINTENANCE

Drafted by a group of international experts
L. ODIER, R. S. MILLARD,
PIMENTEL dos SANTOS, S. R. MEHRA
under the responsibility of UNESCO

LONDON
BUTTERWORTHS

THE BUTTERWORTH GROUP

ENGLAND
Butterworth & Co (Publishers) Ltd
London: 88 Kingsway, WC2 B 6AB

AUSTRALIA
Butterworth & Co (Australia) Ltd
Sydney: 20 Loftus Street
Melbourne: 343 Little Collins Street
Brisbane: 240 Queen Street

CANADA
Butterworth & Co (Canada) Ltd
Toronto: 14 Curity Avenue, 374

NEW ZEALAND
Butterworth & Co (New Zealand) Ltd
Wellington: 49/51 Ballance Street
Auckland: 35 High Street

SOUTH AFRICA
Butterworth & Co (South Africa) (Pty) Ltd
Durban: 33/35 Beach Grove

ISBN 0 408 70079 3

Filmset by Filmtype Services Ltd, Scarborough

Printed in England by Camelot Press Ltd, Southampton

CONTENTS

5

ROAD DRAINAGE

5.1 SCOPE

Drainage is almost always the most important factor determining the performance of a road and when roads fail it is often because of inadequacies in drainage. Failure can happen either spectacularly, as, for example, when cuttings collapse and embankments and bridges are carried away in times of flood, or more insidiously when water penetrating into the road structure weakens it and the soil subgrade so that they are no longer strong enough to support traffic.

In the first part of this chapter, drainage of the road itself is considered. The control of erosion and the stability of embankments and cuttings are discussed, followed by recommendations on the drainage of the road structure. Wet weather during construction is always an impediment to sound and speedy work and notes are given on defensive measures.

6

In the construction of a road, consideration must be given to the natural drainage pattern of the area it traverses and the second part of this chapter discusses the location and waterway requirements for bridges and culverts. The design of bridge foundations and structures is a specialised subject and an indication is given of the principal factors which need to be considered.

5.2 DRAINAGE OF THE ROAD

If a road structure is to perform adequately, care must be taken to remove the surface run-off by a suitable crossfall and to ensure that any water which may gain access to the lower layers of the road structure is also removed. Precipitation is the chief source of water on the road surface. With permanent surfacings the run-off to the edge of the bitumen surface is very nearly complete, but water can infiltrate and/or scour the shoulders on the way to the drainage ditches. Because of the impossibility of preventing some infiltration at the edge of the bitumen surface, or through the shoulder, dense bases are desirable so that water cannot accumulate in the base.

Where open-textured or permeable material must be used, the open trench type of construction (shallow excavation between impermeable shoulders) should be avoided. Even with dense pavement materials it is good practice to construct the shoulders as an integral part of the sub-base and base, using the same material or some other impermeable material (Fig. 5.1).

5.2.1 EROSION CONTROL

Erosion is a work process, the erosive energy being supplied by nature in the form of wind and rain. The susceptibility of soils to erosion depends on the properties of the soil, the length of the slope, the gradient and the vegetative cover. Of these, the vegetative cover is by far the most important factor, since this dissipates the energy of the wind or the water. Silty, light sandy and uncompacted soils are more susceptible to erosion than the heavier clay soils, gravels and well-compacted materials. Local experience, especially agricultural practice, gives a useful guide to requirements in a particular environment. Recent research work indicates that it should soon be possible to define the susceptibility of soils to erosion in simple terms.[1]

In desert areas, wind erosion is instanced by the drifting of sands caused by saltation and can give rise to problems of accumulation of material on the road structure. In order to stop sand drifting over the road surface, it is good practice to lift the road profile above the level of the surrounding country thus increasing the wind velocity over the road surface and keeping the road surface clear. Establishment of cover, either in the form of vegetation or a mineral film of bitumen or similar material, over the loose sand in the area from which it is picked up by the wind, is the only real answer, but this is usually undertaken only as part of a much larger reclamation scheme.

The main erosive force of water comes from falling raindrops when the intensities of precipitation are greater than 25 mm (1 in) per hour. In large catchments, however, sustained rainfall of less than 25 mm (1 in) per hour can often cause severe erosion if the water becomes concentrated in channels or streams. In the tropics

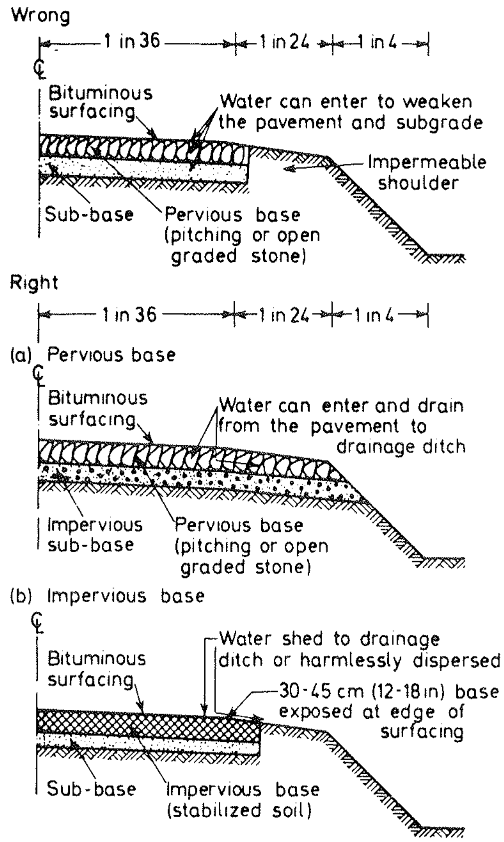

FIG. 5.1. *Drainage of pavement layers.*

some 40% of the rainfall is at intensities greater than 25 mm (1 in) per hour compared with only 5% in temperate climates, and it is this factor which makes tropical rainfall far more erosive than that in temperate climates.

The places most vulnerable to water erosion are the faces of cuttings and embankments, the road surface and shoulders, the sides, bottom and outfalls of drainage ditches. These will be dealt with in the following paragraphs.

5.2.1.1 *Cuttings*

The cutting slopes normally used on rural roads in developing areas are usually too steep to prevent all risk of slips. General practice is to cut initially to quite steep slopes and to clear slips as they occur. Steep slopes have the advantage that they reduce the area subjected to the impact of raindrops.

The particles of some clay soils in the tropics aggregate in clusters and produce a relatively free-draining structure[2, 3]; other soils that are rich in iron or aluminium harden on exposure. In shallow cuttings of up to 5 m (15 ft) in depth such soils may be cut with almost vertical faces.

With deeper cuttings more attention must be paid to the design of the slope, and benching and similar methods may be adopted to control the flow of water down the face of the cutting. Frequently guidance on safe slopes may be obtained from existing cuttings in similar soil nearby. Efforts should be made to encourage some form of vegetation to grow over the face of the cutting to protect it from the effects of splash erosion. Mulching of the freshly prepared slope surfaces with grass, branches, etc. provides immediate protection and encourages the establishment of vegetation.

It is common practice to construct a cut-off ditch along the top of the cutting to prevent flow of water over the face of the cutting. This practice is questionable, particularly where there is dense vegetation, for two reasons. Firstly, the drainage trench will cut through the root system of the vegetation which will considerably weaken the surface layer and may well itself initiate a slip. Secondly, such drainage ditches are difficult to inspect and usually become blocked. The water can then stand and infiltrate into the ground, often at the position of a critical slip plane, and this may give rise to a slip. Banks to direct the water laterally along a contour are preferred. If it is considered necessary to construct a cut-off ditch above a cutting, it should be behind a line at 45° through the toe of the cutting and at least 6 m (20 ft) back from the top of the cutting. In soils subject to erosion such ditches should be lined.

5.2.1.2 *Embankment slopes*

The majority of embankments are placed at a slope of 1½ horizontal to 1 vertical. With well-compacted soil, this slope is normally safe for embankments up to about 8 m (25 ft) in height provided vegetation is established on the slopes. For embankments more than 8 m (25 ft) high, the slope will depend on the materials being used, and means must be provided to break up the flow of water down the length of the embankment face.

It is usual practice on high embankments to carry the surface run-off from the road to selected points and discharge it down the embankment face by means of a drain lined with turf, concrete or metal from discarded bitumen drums. It is also good practice to construct benches to break the flow of water, or alternatively to place turf in horizontal lines along the face of the embankment at intervals of 1–2 m (4–7 ft). Similar results may be obtained by staking down green brushwood or similar materials.

On very lightly-trafficked roads, vegetation will grow on the roadway itself and indeed may be encouraged to provide a surface resistant to erosion. As traffic increases it becomes impossible to maintain vegetation on the running surface or shoulders. These exposed surfaces must be inclined to encourage surface water to flow off the road. The camber must be sufficiently steep to dispose of surface water efficiently yet not so steep as to encourage erosion or to interfere with the control of vehicles. On longitudinal grades steeper than 5%, in areas of high rainfall, because of difficulties in maintaining such surfaces against damage caused by erosion, it is desirable to provide a permanent surfacing.

10

5.2.1.3 *Drainage ditches*

In tropical areas drainage ditches have two main functions:

1. To provide a reasonable capacity to accommodate surface run-off.
2. To dispose of the collected water by infiltration into the soil, evaporation into the atmosphere and run-off to a natural drainage channel or into the surrounding ground.

Drainage ditches should be shaped to minimise the hazard to traffic, and care must be taken to ensure that the discharge from drainage ditches does not give rise to erosion.

Wide and shallow drainage ditches that keep any water as far away as practicable from the formation meet these requirements most closely, as they reduce water velocities in the invert and give large surface areas for infiltration and evaporation.

The slope of the sides of drainage ditches should generally not exceed 1 in 4 to minimise erosion. In addition to taking the surface run-off from the road and shoulders the side drains may also intercept sheet run-off from the surrounding country and they should discharge the waters collected where no damage to the road structure or to adjoining land will result. In the tropics, particularly where water-tables are deep, there is great potential for infiltration of water into the soil and for evaporation. Wide and shallow drainage ditches provide the maximum area for both phenomena to occur and such ditches are ideally suited to mechanical maintenance.

The minimum dimensions of side drains should be dictated by the dimensions of the motor graders which will maintain them. Wider ditches with gentle batters are less prone to blockage and, even with manual maintenance, require less maintenance effort. Where for economic reasons it may be necessary to have steep slopes on the sides of drainage ditches, as may be the case in cuttings, or where there is insufficient road reserve width, careful attention must be paid to maintenance. Where the volume of run-off is large, it will probably be necessary to pave the invert and line the walls. Typical drainage ditches are shown in Fig. 5.2.

With such ditches it is easy to lead water into contour drains at frequent intervals to discharge the water on to the surrounding ground. Where a minimum longitudinal fall of 1% for unpaved and 0·5% for paved drainage ditches cannot be obtained the water will not flow; percolation and evaporation must then be relied on for disposal of run-off, and the ditch designed accordingly. The maximum distance between contour drains must be limited, depending on the gradient and the cross-section of the ditch used, to prevent excessive water velocities which will cause erosion. Where the longitudinal gradient is small (up to 3%) contour drains will normally be between 500 m and 1000 m apart. On steeper gradients and on the insides of bends where superelevation is used they will be required at shorter intervals.[4]

Turnouts are required more frequently with deep narrow ditches than with wide shallow ditches, but with the deeper drain it is more difficult to make the turn-out. For this reason V-shaped drains cut with one pass of the grader are not particularly satisfactory. In cuttings, where space is limited, deep narrow drains are often used. Through soils that are subject to erosion it will often be necessary to line the walls and inverts of such drains.

5.2.2 CULVERTS

It is usually necessary to carry water under the road at intervals by means of culverts. Where this can be done at a natural drainage

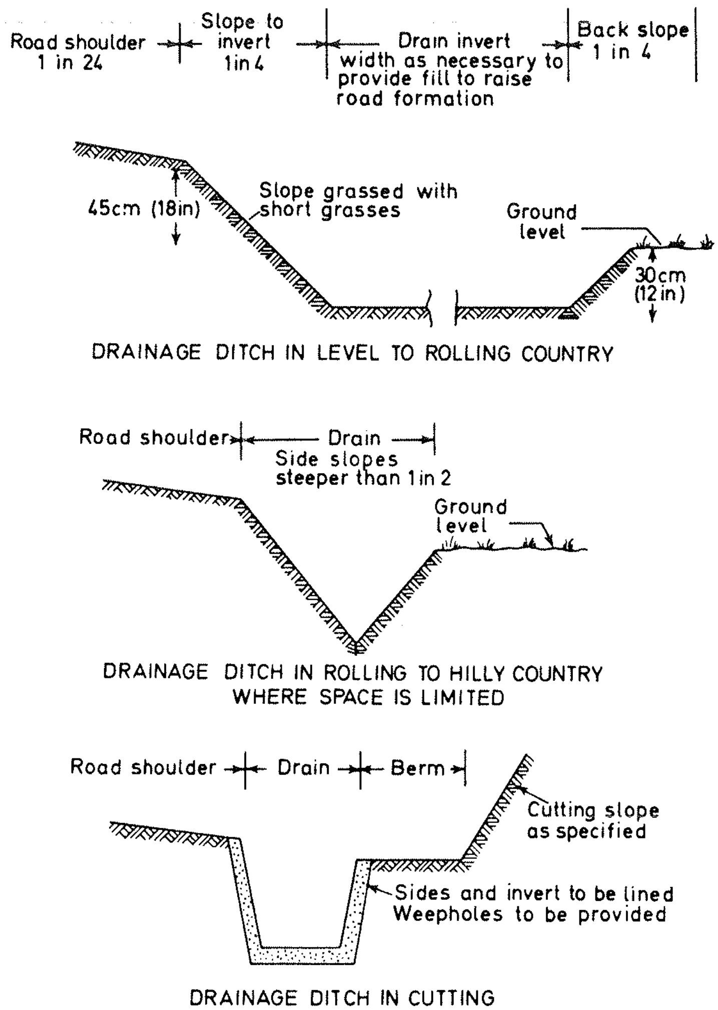

FIG. 5.2. *Typical drainage ditches.*

channel few problems arise, but where there is no natural drainage channel, and especially where the soil is easily erodable, particular attention will need to be paid to the discharge from the culvert in order to break up the concentrated flow.[5] One relatively simple measure is to spread out the discharge water over a wide area which may be lined with concrete or rock (Fig. 5.3).

In extreme cases, in hilly country for example, energy-dissipating spillways may be needed. These may be lined with concrete or masonry and should discharge into a stilling chamber.

5.2.3 SUBSURFACE WATER

The profile of the permanent water-table generally follows in a more subdued manner the general relief of the country. Additionally, temporary water-tables may occur, especially during the rainy seasons, at points in the profile where the occurrence of a more impermeable layer prevents downward percolation of rain water.

When a water-table, either permanent or temporary, is encountered one of two expedients may be adopted:

1. The road may be raised by means of an embankment to the desired elevation above the water-table, or
2. drainage arrangements may be made to lower and dispose of the water.

The first expedient is usually adopted in low-lying or poorly drained flat areas where it would be difficult if not impossible to lower the general water level. In these low-lying areas flooding from nearby watercourses is often a problem and it is desirable that the road be raised some 1 m (3 ft) above the highest recorded flood level. These embankments may themselves impede the disposal of flood water and on large schemes a hydrological survey may be desirable. Locally-available soils, even when they are heavy clays, may be used to build these embankments. However, when there is a permanent water-table close to the surface selected soils of low plasticity should be used for the upper 50 cm (2 ft) of fill.

The second expedient is applicable in localities where the road cuts through the seepage lines in hillsides. Spring lines are a common occurrence where there are rock outcrops or where impermeable clay layers are exposed. Water at these points must be led clear of the road structure by drainage. In sidelong slopes this will take the form of a ditch some 60 cm (2 ft) deep on the uphill side of the road with culverts at intervals to convey the water collected across the road. Where the road runs across the contours, the spring line may be continuous beneath the road and agricultural drains must be installed in the subgrade to lead the springs into the side drains. In low-lying country, open drains at the sides of the road can function to lower the water-table, provided outlets are available to conduct the water away.

Although the installation of drains is fairly simple when the water-table is permanent, difficulties arise when the water-table is a temporary phenomenon confined to the wet season. Fortunately the extent of these temporary conditions is usually limited by the

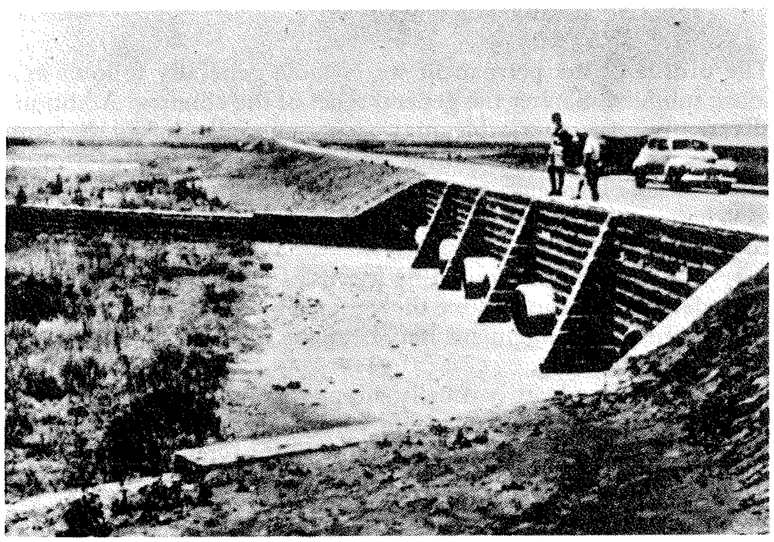

FIG. 5.3. *Culvert with fanned outlet to minimise erosion.*

construction of the other drainage arrangements to cater for surface run-off from the pavement. It is often not possible to locate these areas of temporary springs in advance of construction. After they have become evident during rainy weather, remedial measures must be taken to intercept the flow and convey it away from the pavement.

In designing retaining walls and bridge abutments, consideration must be given to the possibility of water collecting behind the walls. In situations where water could accumulate, a blanket of material, graded to prevent silting,[6] should be inserted against the wall and the water led away to the general drainage system by weepholes or preferably by a continuous back drain.[7] Similarly, springs located in the faces of cuttings must be tapped by drains or rock toes—even piping judiciously driven into the cutting face may suffice—as otherwise progressive slumping of the surface will occur. In all instances the aim must be to prevent the build-up of hydrostatic pressures which might cause failure.

When slips occur in cuttings and it is not possible to cut back to a more stable slope, counterfort drains filled with rubble and extending below the slip plane are the usual remedial measures adopted to stabilize the slope. To dispose of surface water from the face of the cutting and prevent further softening of the soil, a secondary system of drains laid in a herringbone pattern should be constructed in the surface of the cutting.[8]

5.2.4 DRAINAGE OF PAVEMENT LAYERS

Pavement bases may be designed either to exclude water altogether or alternatively to permit the entry and egress of water. When effectively impermeable bases with a low voids content are used, e.g. soil-cement or well-graded crushed stone, drainage of the base is not necessary. When permeable and porous base materials are used, e.g. stone pitching or poorly-graded crushed stone, particular attention must be given to the drainage of the base layer. Base and sub-base materials should extend across the shoulders to the edge of the drainage ditches and the surface of the sub-base layer should be given an adequate crossfall to assist this drainage (Fig. 5.1).

Recent investigations into moisture conditions under bituminous-surfaced roads in tropical areas[9] indicated that the occurrence of wet and weak subgrade conditions was rare and that, when it did occur, it could be attributed in the majority of instances to deficiencies in the drainage arrangements, which permitted the accumulation of surface water in the pavement layers. Had the layers been provided with adequate drainage outlets to the side drains then failure of the road would have been avoided. Fig. 5.4 shows typical edge failures which have occurred where trench type construction was adopted with a poorly-graded crushed stone base overlying a heavy clay subgrade.

5.2.5 SURFACE WATER DRAINAGE OF THE ROAD SURFACE AND SHOULDERS

Rain falling on road surfaces and shoulders must be conveyed efficiently and quickly to the side drains. Accumulations of water on earth and gravel roads will cause weakening and may ultimately lead to the road becoming impassable to traffic. Water lying on roads with permanent surfaces is an inconvenience and a hazard to road users and will usually lead to the formation of potholes.

To promote adequate drainage, surfaces are given a crossfall the value of which is determined by the nature of the surface. On soil

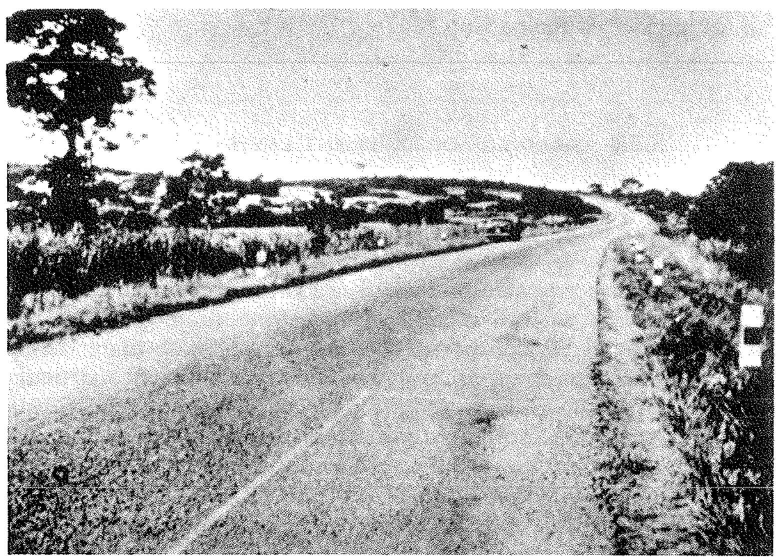

FIG. 5.4. *Typical edge failures due to using porous base materials in trench construction.*

surfaces, steep crossfalls are required depending on their permeability and the ease with which wheel-loads form indentations in the surface. On hard bituminous and concrete surfaces the crossfall is dictated by the tolerances to which these surfaces can be laid. While steep crossfalls help to dispose of water they are a hazard to traffic and care must be exercised to avoid steeply sloping shoulders which might cause vehicles leaving the roadway in an emergency to overturn. Table 5.1 summarises the range of values considered suitable for various surfaces.

Table 5.1 SUITABLE CROSSFALLS FOR ROAD SURFACES

Type of surface	*Crossfall* %
Earth and gravel road surfaces and shoulders	3 to 4
Bituminous and concrete road surfacings	2 to 3* (normally 2½)

* A crossfall of up to 1 in 10 (10%) may be used to provide superelevation on bends. (See Table 3.2 (Chap. 3).

Although a crowned road section is usual, lengths with continuous crossfall occur at bends and it may also be more practical to use a crossfall on some straight lengths of road. In hilly terrain the longitudinal gradients on the road may be steep and much in excess of the transverse crossfall. In such conditions water will flow predominantly along the road and arrangements must be made to collect this water at intervals, especially where the road changes direction and where concentrated flow leaving the road could cause erosion of embankments. A cattle grid type of arrangement (Fig. 5.5) is useful in such circumstances.

Regular maintenance of road shoulders is required if they are to operate effectively in the disposal of water from the pavement surface to the drains. Grit washed from the road surface tends to collect at the junction with the shoulder, particularly when grass is allowed to grow on the shoulder; also the grass will cause the shoulder material to bulk up above the pavement level. These accumulations must be regularly removed; otherwise they impede the drainage of the surface and water may find its way into the road base. An example of a well-laid-out carriageway with properly-maintained shoulders is shown in Fig. 5.6.

5.2.6 CARE OF EARTHWORKS AND PAVEMENT STRUCTURES DURING CONSTRUCTION

Generally in wet weather road building operations must stop, but much can be done both to protect the uncompleted road structure from damage by rain and to make it possible to resume work quickly as the weather improves.

5.2.6.1 *Cuttings and excavations*

When forming all cuttings and excavations, care should be taken to work from the lowest point and to maintain a slope on the floor of the cutting so that surface water can drain rapidly out of the

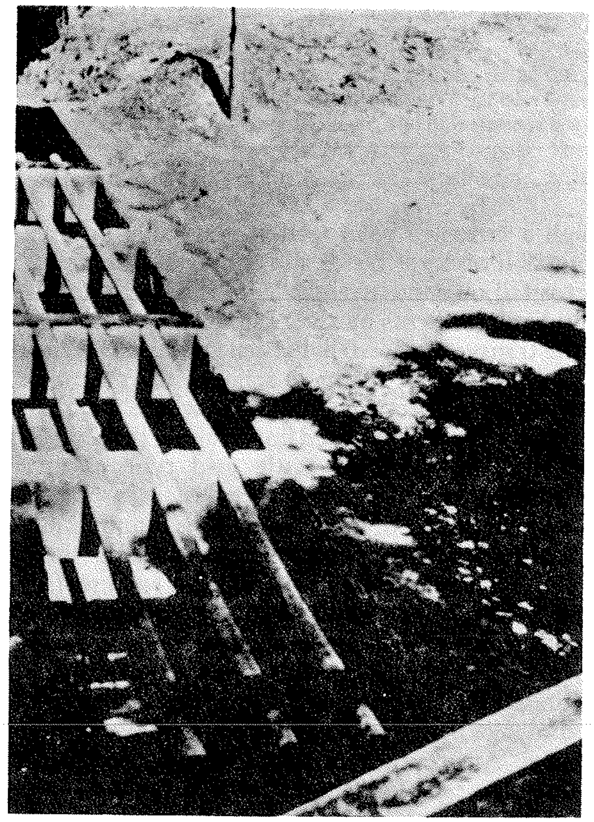

FIG. 5.5. *Cattle grid type drain to collect water flowing longitudinally down the road on steep slopes in Malaysia.*

cutting area (Fig. 5.7). Where surface flow can enter the cutting area from the surrounding ground an intercepting drain should be cut to lead the water around the cutting area. Care should also be taken to keep a smooth surface on the floor of the cutting and remove any ruts that form.

It is particularly important to leave a well-shaped surface at the end of the day's work. It is generally better to stop the operation of plant completely during periods of heavy rain until the rain has stopped and the surface water has run off. Trying to keep the plant operating during such periods usually leads to rutting and puddling of the surface; the plant cannot operate efficiently and the result is even longer delays waiting for the road to dry out when the rain has ceased.

5.2.6.2 *Fill*

The fill area is often the critical point in deciding when operations have to be stopped during wet weather. Material deposited loosely on the fill area has a high voids content and will allow rain to enter the soil and rapidly wet it up. If this happens the wet soil may have to be removed before work can continue. To prevent this, the soil

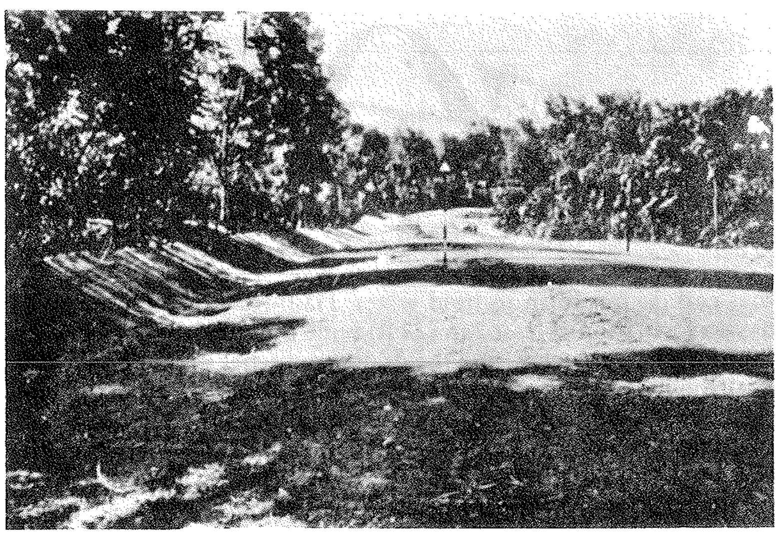

FIG. 5.6. *Properly shaped and well maintained road cross-section in Kenya.*

should be deposited in thin layers which should be rolled immediately to give the desired density with a smooth surface.

Embankments should be formed with a crown at the centre so as to shed the water over the sides of the embankment; care must be taken to avoid leaving any areas of uncompacted material at the edges. It is well worthwhile keeping a grader working continuously on the fill to maintain the surface in good shape. No uncompacted material should be left on the surface at the end of a day's work.

5.2.7 HAUL ROADS

Haul roads may be on the road alignment or they may be separate temporary roads made solely for the purpose of construction. Usually they will be temporary roads with earth or gravel surfaces. Provided they are kept in good shape, rain will rapidly run off the surface and earth-moving plant will be able to traverse them. The surface must be kept well-compacted and repeatedly graded so

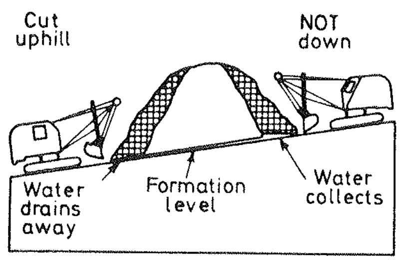

FIG. 5.7. *Correct method of cutting to get proper drainage.*

that ruts do not form to hold water. Haul roads should normally have a formation at least 7 m (24 ft) wide between the slopes down to the drainage ditches, and the drainage ditches should be shallow V-shaped drains, widened with a flat bottom if required, and capable of maintenance by a mechanical grader. Although haul roads may be only temporary works, money spent on constructing and maintaining them is usually more than repaid through increased efficiency of plant operation.

In regions with alternate wet and dry seasons, the engineer, while taking steps to prevent the accumulation of large quantities of water, may also use the rain falling in the wet season to minimise the effort needed in compacting embankments. Depending on the severity of the rains, construction may either be carried out during the wet season or be commenced soon after the cessation of the rains. In predominantly wet climates, it will normally be the aim to carry out earthworks in the seasons when rainfall is least and to provide a working platform by constructing at least the sub-base so that work can continue whenever possible during the wetter seasons.

5.3 BRIDGES AND CULVERTS

The construction of a road interferes with the drainage pattern of the area through which it runs. It is the responsibility of the road designer to make sure that the drainage structures are adequate to pass flood water without causing either harmful flooding on the upstream sides or scour and erosion where flow is concentrated.

5.3.1 WATERWAY REQUIREMENTS

For economic reasons it is not usual to design bridges and culverts to discharge the maximum floods which may occur and some flooding of the areas abutting on the watercourse may be acceptable, the amount and frequency depending on the cost and extent of the resulting damage in the lands inundated. Thus, while bridges on major rivers are designed to discharge floods with an expected frequency of 100 years, smaller drainage structures where only small areas of pasture land are involved and the risks to the road structure are small, may be designed on the basis of a one year storm. Where more extensive damage is possible, e.g. the breaching of the road embankment, the designer must balance the cost of repair and inconvenience to traffic against the additional cost of the drainage structure.

Flood flows to be expected depend on the size, gradient and other characteristics of the catchment area drained and on the precipitation within that area. Accurate predication of maximum flood flows is rarely possible since the hydrological data needed on rainfall and run-off is only available for catchments where long-term measurements have been made, usually for flood control, irrigation or power generation. The determination of adequate floodways is as yet more a matter of engineering judgement than of science. Local knowledge unsupported by written record may be quantitatively unreliable but will at least serve to indicate where flooding is likely to occur.

Data on the size of existing drainage structures, on the size, shape and nature of the catchment areas, on the velocity of flow in channels together with rainfall data (however rudimentary) and on the highest flood levels recorded or witnessed provide a basis, albeit often imprecise, for the estimation of waterway openings. In forming such estimates it should be noted that the development of the land, e.g. forest clearing, possibly stimulated by the construction of the road, can lead to a radical alteration in the run-off characteristics of the catchment area, which may produce significant increases in peak flood flows.

Many formulae have been used in various countries for estimating waterway sizes. A commonly employed method for run-off determination is the Rational Formula.[10]

$Q = C.I.A.$ (or $Q = \frac{1}{366} C.I.A.$ in metric units)

where Q = peak rate of run-off in cusecs (cumecs)

C = percentage of run-off depending on the characteristics of the catchment.

I = mean rate of rainfall during the time of concentration in inches/hr (mm/hr)

A = drainage area in acres (hectares)

Having determined the quantity of water to be passed, a velocity V must then be chosen which is a safe velocity from the point of view of scour, for the stream bed and banks and the structure through which it passes. As a general rule 2 m/s (6 ft/s) is a desirable maximum. Then the formula $A = Q/V$ will give the required waterway area.

It is important to note that the 'Rational Formula' assumes that the time of concentration is constant for a given catchment and that the peak rate of run-off is directly proportional to the mean rate of rainfall during the time of concentration. It is therefore a special case of the 'unit hydrograph' method[11] which is now generally accepted by hydrologists as being the most reliable and satisfactory method available at present for calculating rates of run-off from natural catchments. Probably the greatest source of error in in the Rational Formula is its inability to deal effectively with the characteristics of the catchment, since all variations in slope, shape, soil type and land use have to be taken into account by the appropriate selection of the value of the co-efficient 'C'. In the 'unit hydrograph' method the more realistic approach is adopted of not only varying 'C' but also varying the shape of the hydrograph.

It would, however, be unreasonable to suggest that engineers should abandon the Rational Formula in favour of hydrograph techniques until more accurate and reliable rainfall and run-off data are obtained. The accuracy of any methods depends to a large extent on the accuracy of the data employed in it and it is considered that the engineers concerned are already making reasonable use of the inadequate data available to them.

Alternatively a simple formula applicable to a limited region may be developed in which known flood flows are related to the area drained and the terrain and vegetation of the catchment,[12] e.g.

$$Q \text{ or } B = KA^n$$

where Q and A are respectively the peak run-off and drainage area as previously given in the Rational Formula.

$B =$ the area of waterway required

$K =$ a constant depending on the terrain and vegetation of the drainage area

and $n =$ a power less than unity.

The hydraulic calculations for bridges and culverts are straightforward.[13, 14] Where piers are used, some constriction of the river channel may be caused and the effect of backing up on upstream water levels needs to be considered.

In the early stages of the development of a road it may be reasonable to consider submerged structures and accept that passage across the river will be impossible at times. In these instances flow across the approach embankments may also occur and steps must be taken to pave or otherwise protect the downstream slope of the embankments to prevent them from being breached. Similar treatment would also be appropriate where information on maximum flood flows is scanty or unreliable but where the additional cost of providing a safe waterway would be large in relation to the risks and possible inconvenience involved.

Paved fords ('Irish Bridges') can usefully be employed on many roads in developing countries, particularly in arid regions where watercourses are normally dry with the exception of a few days per year, or even per decade. In all cases a balance must be struck between the importance of the route, the delays caused to road users and the additional cost of the alternative culvert or bridge. Ford pavings are usually concrete, bituminous material or masonry. Curtain walls are provided to resist scour; alternatively gabions or rip-rap may be used. Water-depth indicators to warn traffic should be provided at all fords where the water may rise to dangerous levels.

Another low-cost expedient is the submersible bridge, i.e. a bridge with openings adequate to take normal flow, but which may be covered and therefore unusable in times of flood. They range from long embankments of soil as used in low-lying areas, e.g. in deltas, to solid masonry structures sometimes used over shorter crossings. The surface must be kept devoid of obstructions such as parapets in order not to impede the passage of flood water. Even so, such bridges offer considerable resistance to flood flow and must be designed for flood and floating debris loads.

In areas subject to frequent flooding, the effort and cost of maintaining submersible bridges can be very great.

5.3.2 LOCATION OF STRUCTURES

Bridges are expensive structures and road location should attempt to minimise their number and size. When the road must cross a river or large stream, careful consideration must be given to the siting of the bridge. A small bridge some 10–20 m (10–20 yards) long is similar in cost to about 800 m ($\frac{1}{2}$ mile) of bituminised road while structures 100 m (100 yd) long may cost as much as 8–16 km (5–10 miles) of roadway. The shortest crossing is, however, not necessarily the cheapest, as foundation conditions and the stability of the river bed and banks must be considered.

Thorough site investigation including drilling and examination of cores is the prerequisite to the choosing of the most advantageous location. The overall construction cost must be considered with any increases in roadworks needed being offset by economies in the bridge structure due to its more favourable situation. It may often be advantageous to realign the water-course to improve the angle of crossing or to reduce the number of structures by obviating multiple crossings of the same water-course.

Any road traversing the countryside must cross the more minor and tributary drainage channels and the run-off from the pavement must ultimately be conveyed into natural water-courses. These flows are conveyed beneath the road by culverts or small bridges. The need for these at streams and other natural drainage channels will normally be self-evident. Extra culverts are required on side-long ground to conduct water from the side drains across the road. The frequency with which such culverts are needed depends on the terrain and on the intensity of rainfall. In extreme conditions up to 6 culverts per km (10 per mile) may be needed.

5.3.3 BRIDGE FOUNDATIONS

The type of foundation for bridges is determined from the site investigation. All soil layers that will be significantly stressed by the bridge abutments or piers must be examined, down to bedrock if necessary. At major crossings the advice of a geologist or of the geological survey department should be sought; river valleys are primary lines of erosion, their location often being dictated by geological lines of weakness, and the occurrence of buried valleys infilled with recent heterogeneous deposits of alluvium, which are most treacherous foundations, is an ever-present possibility.[6]

In minor structures, whether of precast concrete, reinforced concrete or corrugated metal, foundation pressures are usually low; in unstable ground conditions, the foundation should be over-excavated and back filled with a 60–120 cm (2–5 ft) layer of well-compacted granular fill material. It is in these conditions that flexible structures are seen to best advantage since structural distortion does not involve fracture and collapse.

The simplest type of foundation for bridges is that in which the abutments or piers rest directly on a suitable soil stratum. Safe bearing pressures should be based on the results of the site investigations[15] and allowance may need to be made for increased loading due to such hazards as the damming of the waterway with trees during floods.

The danger of scour must be considered where abutments and piers are not founded on bedrock. The base of foundations must be taken below the level to which the river bed is eroded by scouring action during floods. The depth of many rivers increases during times of flood at a rate greater than that at which the water level rises. Several records indicate that the ratio of the increase in the depth of the bed to the rise in the water level is as much as 4:1; in extreme instances the ratio has been as great as 7:1.[7] The erosive power of water varies as the square of the velocity and ultimately the channel reaches an equilibrium shape depending on the material of which the river bed is composed. Data on these equilibrium velocities for various bed materials are available and provide some indication of the likely amount of scour.[13] Again, experience of local conditions and existing structures is the best guide and this experience can often be related to characteristic features of the river, e.g. the meander length. In some instances the paving of the river bed or the protection of the river bed with rip-rap may be needed.

Abutments and piers founded directly on the bearing stratum are generally constructed with cofferdams; where dry weather flows are small and seasons well defined a simple bag cofferdam may suffice. In deeper water, or where the river bed above the bearing stratum consists of highly permeable or unstable deposits, a sheet pile cofferdam can be used and the piling may usefully be incorporated in the foundation as a protection against scour. Reinforced concrete caissons, constructed where necessary on a temporary dumpling, provide a useful alternative. For the heaviest foundations and largest structures, caissons sunk using compressed air may be necessary. In permanent deep water it is often economical to dispense with cofferdams by using piles as mentioned in the several methods given in the next paragraph. In these methods, no cofferdamming, underwater work or pumping is required, since all the construction is carried out above water.

In deep water and when the depth to a suitable bearing stratum is great, piles often provide the most economical foundation. They may support the imposed loads by end bearing or skin friction depending on the results of the site investigation. Piles, either driven or bored, may be made of steel, reinforced concrete or prestressed concrete and there are many, proprietary types and sinking methods available. Timber piles can be used with advantage provided they are located below permanent water at all times. In tidal water they would therefore not be suitable above low water level; below this level the possibility of attack by marine borers must be considered. Test loading of piles at an early stage during construction is essential to verify the adequacy of the design of the piled foundation.

5.3.4 BRIDGE SUPERSTRUCTURES

5.3.4.1 *Loading*

Bridges are subjected to dynamic loads from traffic, the wind and temperature variations. Traffic loads consist of vertical loads and braking and tractive forces; wind and floating debris loads tend to overturn the structure and temperature effects engender longitudinal stresses. In view of the complexity of these factors many road authorities have idealised the loadings for which bridges under their jurisdiction must be designed.[16] Both bridges and roads are generally designed for maximum permissible axle loads in the region of 10 tons. Where heavier axle loads are envisaged these must be specifically considered in the design.

Temperature variations tend to alter the length of bridges and, when these movements are restrained or prevented, the deck and supporting structure must be capable of resisting the stresses set up. Alternatively, joints may be used. The total amount of movement is directly proportional to the annual temperature range and where the movement does not exceed 5 mm (0·2 in) the simple breaking of the contact between the deck and piers is all that is needed. As movements increase, more sophisticated forms of joints and bearings will be required.[17]

Permanent transverse deck joints at piers and abutments should be designed to prevent entry of water through the deck to girder bearings. Scuppers should be provided at intervals along the edge of the carriageway and located so that their discharge is clear of girders and bearings. In this connection, where there is the likelihood of differential settlement between piers, it is well to make arrangements for jacking of the deck when inserting joints initially.

Bridge parapets should be crash-resistant and designed so that damaged sections can be readily removed for repair or replacement.

5.3.4.2 *Materials*

Steel, reinforced concrete and prestressed concrete are the materials most commonly used in bridge works at present. Cost and durability determine the choice in any given circumstances. Steelwork, prefabricated to a large degree in the factory, is easily transported over long distances; launching of long spans is easily accomplished (Fig. 5.8) and the use of friction grip bolts has greatly assisted assembly on site. The amount of temporary site works is a minimum but regular maintenance is needed, particularly in corrosive atmospheres. Reinforced concrete requires much formwork which must often be erected in difficult situations over water, but with diligent control over materials and workmanship the structure requires the minimum of maintenance subsequently.

Prestressed concrete, the latest candidate in the bridge-building field, possesses the advantages of both. The more efficient use of the component materials, high-tensile steel and high-quality concrete, result in light members which are amenable to precasting techniques in favourable conditions. These members are only a little more cumbersome than equivalent steel sections and possess the maintenance advantage of normal reinforced concrete. Timber, when readily available, provides a useful material for bridges in remote areas. It can be quicker and more economical to use local timber as there is no need to transport plant and materials over long distances.

Maintenance of the structure must be considered at the design stage, in relation to both materials and environment. For instance, steelwork will corrode, especially in coastal environments with on-shore winds. Careful attention should be paid in design to avoid situations which will cause water to be trapped where inspection

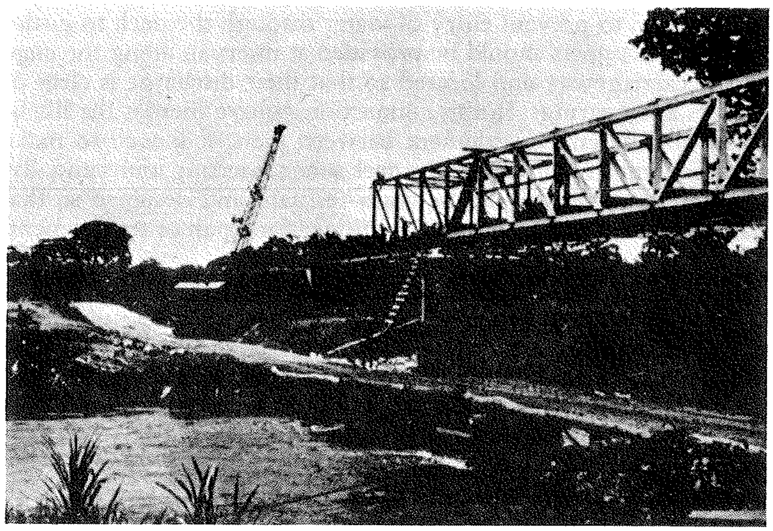

FIG. 5.8. *Dismantling launching nose from steel bridge in Sierra Leone.*

will be difficult and where cleaning and repainting cannot easily be carried out. Similarly in reinforced concrete an adequate cover of dense concrete is required to protect the reinforcing steel against corrosion, and the concrete mix must be sufficiently workable to ensure compaction around the reinforcement.[18, 19, 20, 21]

Bridges are still occasionally constructed in masonry or brickwork in areas where a local tradition of craftsmanship exists and such structures harmonise effectively with the surroundings (Fig. 5.9). Embellishment of modern bridges should emphasise their form and constructional lines. For example, parapets should be distinguishable from the load-carrying superstructure. Attempts to disguise new bridges in traditional garb usually results in a confused appearance.

Culverts are generally constructed of precast concrete pipes, reinforced in the larger diameters, or of corrugated metal pipes or sections, while box culverts are often used where short-span bridges would otherwise be required. Again the question of cost and subsequent maintenance is decisive. For small culverts, empty bitumen drums provide a useful source of shuttering when mass concrete is being employed. Precast reinforced concrete box culverts have the advantage of low head room for a given waterway area and, unlike a pipe, their tops can be incorporated in the road surface.

5.3.4.3 *Standardisation*

The adoption of standardised designs for the smaller drainage structures, including bridges up to about 15 m (18 yd) long, can lead to economies and improvement in quality. Many road divisions undertaking both maintenance and improvement can usually

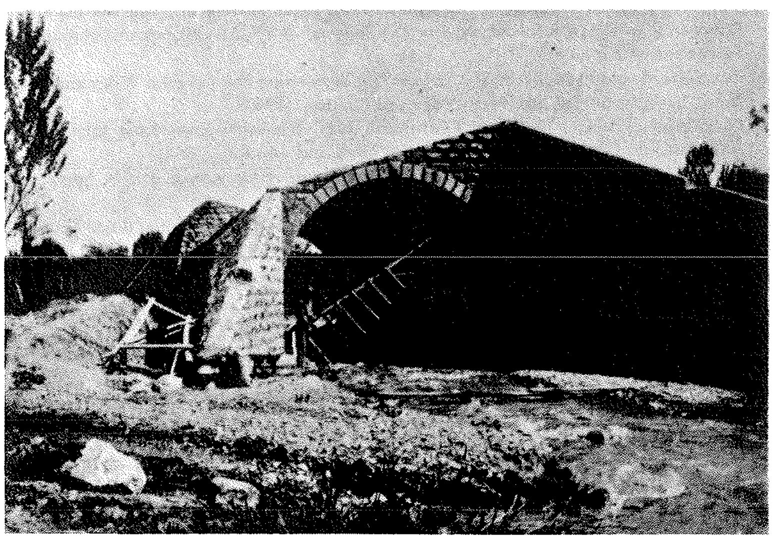

FIG. 5.9. *Masonry bridge under construction in Iran.*

support a small yard where precast concrete products can be made. Output might be continuous or seasonal and, in some countries, might provide a useful wet season occupation for key staff who would otherwise be lost to the road authority. Products might include concrete pipes, fencing posts, short decking slabs and kerbs, the items being stockpiled for future use.

When a major road construction project is being undertaken, precasting on a more extensive scale may be economical. Standard designs for prestressed beams are available[16]; the resulting speeding up of bridgeworks reduces congestion at these points during construction.

REFERENCES

1. Santos, M. P. P. Dos and Castro, E. De. 'Soil erosion in roads.' *Proc. 6th internat. Conf. Soil Mech.* **1**, 116–20. (1965).
2. Newill, D. 'A laboratory investigation of two red clays from Kenya.' *Geotechnique, Lond.* **11**(4), 302–308. (1961).
3. Strongman, F. S. 'The Materials Laboratory's evaluation of some Kenya materials.' *Conference on Civil Engineering Problems Overseas.* The Institution of Civil Engineers. London. (1964).
4. Bureau Central D'Etudes Pour Les Equipements D'Outre-mer. Recommendations du Comité Technique crée pour l'étude générale des Routes Economiques. Paris. (1953).
5. Turner, A. K. The control of roadside erosion. *Department of Scientific and Industrial Research, Road Research Laboratory, Overseas Bulletin* No. 17. Road Research Laboratory. Harmondsworth. (1963).
6. Terzaghi, K. and Peck, R. B. *Soil Mechanics in Engineering Practice.* John Wiley and Sons Inc; Chapman & Hall Ltd. New York and London. (1948).
7. Peck, R. B., Hanson, W. E. and Thornburn, T. H. *Foundation Engineering.* John Wiley and Sons Inc; Chapman & Hall Ltd. New York and London. (1954).
8. Capper, L. P. and Cassie, W. F. The mechanics of engineering soils. E. & F. N. Spon Ltd. London. (1954).
9. Williams, F. H. P., Russam, K. and O'Reilly, M. P. 'An Investigation of Road Foundations in East Africa.' *Proc. Aust. Road Res. Bd.* **1** (Pt 2). 841–50; Discussion, 850. (1962).
10. Watkins, L. H. Research on surface-water drainage. *Proc. Inst. C. E.,* 305–30 (March, 1963).
11. Linsley, R. K., Kohler, M. A. and Paulhus, J. L. H. *Hydrology for Engineers.* McGraw-Hill Book Company, Inc. New York. (1958).
12. Chow, Ven Te. 'Hydrologic determination of water areas for the design of drainage structures in small drainage basins.' *University of Illinois, Engineering Experiment Station Bulletin* No. 462. Urbana, Ill. (1962).
13. Gibson, A. H. *Hydraulics and its Application.* Constable and Co Ltd. London. (1948).
14. Department of Public Works, Division of Highways, State of California. *California Culvert Practice.* Department of Public Works. Sacramento. (1944).
15. British Standards Institution. *Site investigations.* BS. Code of Practice CP 2001. London. (1957).
16. Rowe, R. E. *Concrete Bridge Design.* C.R. Books Limited; John Wiley & Sons Inc. London and New York. (1962).

17. Black, W. and Adams, H. C. 'Survey of expansion joints and bearings in Britain. *International Association for Bridge and Structural Engineering. Seventh Congress.* Rio de Janeiro. (1964).
18. National Association of Australian State Road Authorities. Highway Bridge Design Specifications. Sydney. (1965).
19. Boudic, L. 'Manuel de construction des passerelles et ponts provisoires à l'usage des administrateurs de la France d'Outre-Mer.' Dunod. Paris. (1955).
20. Comrie, J., Editor. *Civil engineering Reference Book. Volume III*, 2nd ed. Butterworths & Co Ltd. London. (1961).
21. U.S. Bureau of Public Roads. Standard plans for highway bridges. *Vol. 1, Concrete Superstructures. Vol. 2, Structural Steel Superstructures. Vol. 3, Timber Bridges. Vol. 4, Typical Continuous Bridges.* Washington, D.C. (1962).
22. Rodier, J. and Auvray, C. Estimation des débits de crues décennales pour les bassins versants de superficie inférieure à 200 km^2 en Afrique occidentale. O.R.S.T.O.M. (July, 1965).
23. Delorme, A. Nouvelles considérations sur les débouches des petits ouvrages sous routes. *Annales des Ponts et Chaussées.* (November, 1959).

Masonry culvert is under construction in Oaxaco, Mexico.

VOLUME IV - HIGHWAY DRAINAGE GUIDELINES

Guidelines for the Hydraulic Design of Culverts

Prepared by
Task Force on Hydrology and Hydraulics
AASHTO Operating Subcommittee on Design

341 National Press Building
Washington, D.C. 20045

GUIDELINES FOR THE HYDRAULIC DESIGN OF CULVERTS

CONTENTS

HYDRAULIC DESIGN OF CULVERTS

1.0 Introduction

The function of a culvert is to convey surface water across or from the highway right-of-way. In addition to this hydraulic function, it must also carry construction and highway traffic and earth loads; therefore, culvert design involves both hydraulic and structural design. The hydraulic and structural designs must be such that risks to traffic, of property damage and of failure from floods are consistent with good engineering practice and economics. These guidelines are concerned with the hydraulic aspects of culvert design and make reference to structural aspects only as they are related to the hydraulic design.

Structures measuring more than 20 feet along the roadway centerline are conventionally classified as bridges. Many longer structures, however, are designed hydraulically and structurally as culverts. Culverts, as distinguished from bridges, are usually covered with embankment and are composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert. Bridges are not designed to take advantage of submergence to increase hydraulic capacity even though some are designed to be inundated under flood conditions. For economy and hydraulic efficiency, culverts should be designed to operate with the inlet submerged during flood flows, if conditions permit. At many locations, either a bridge or a culvert will fulfill both the structural and hydraulic requirements for the stream crossing. Structure choice at these locations should be based on construction and maintenance costs, risk of failure, risk of property damage, traffic safety, and environmental and aesthetic considerations. All considerations in structure selection will not be discussed here, but there are advantages favoring culverts because of traffic safety aspects of bridge railing and of problems with bridge deck icing and concrete deterioration.

Culverts are usually considered minor structures, but they are of great importance to adequate drainage and the integrity of the highway facility. Although the cost of individual culverts is usually relatively small, the total cost of culvert construction constitutes a substantial share of the total cost of highway construction. Similarly, the total cost of maintaining highway hydraulic features is substantial, and culvert maintenance accounts for a large share of these costs. Improved traffic service and a material reduction in the total cost of highway construction and maintenance can be achieved by judicious choice of design criteria and careful attention to the hydraulic design of each culvert.

irregularities existed during the flood, such as blockage of the channel from drift or ice, or backwater from stream confluences.

2.6 Existing Structures

Considerable importance should be placed on the hydraulic performance of existing structures and all information available should be gathered in the survey. The performance of structures some distance either upstream or downstream from the culvert site can be helpful in the design. Often, local residents, highway maintenance personnel, or others can furnish important highwater data and dates of flood occurrences at such structures.

Data at existing structures should include the following, if available:

1. Date of construction;
2. Major flood events since construction and dates of occurrence;
3. Performance during past floods;
4. Scour indicated near the structure;
5. Type of material in streambed and banks;
6. Alinement and general description of structure, including condition of structure, especially noting abrasion, corrosion or deterioration;
7. Alinement and general description of structure, including dimensions, shape and material and flowline invert elevations;
8. Highwater elevations with datum and dates of occurrence;
9. Location and description of overflow areas;
10. Photographs;
11. Silt and drift accumulation;
12. Evidence of headcutting in stream; and
13. Appurtenant structures such as energy dissipators, debris control structures, stream grade control devices.

2.7 Field Review

The engineer designing drainage structures should be thoroughly familiar with the site under consideration. Much can be learned from the survey notes, but the most complete survey cannot adequately depict all site considerations or substitute for a personal inspection by the designer. Often, a plans-in-hand inspection by the designer and the construction engineer will prove mutually beneficial by improving the drainage design and reducing construction problems.

3.0 Culvert Location

Culvert location deals with the horizontal and vertical alinement of the culvert with respect to both the stream and the highway. It is important to the hydraulic performance of the culvert, to stream stability, to construction and maintenance costs, and to the safety and integrity of the highway.

Culvert location in both plan and profile is of particular importance to the maintenance of sediment-free culvert barrels. Deposition occurs in culverts, obviously, because the sediment transport capacity of flow within the culvert is often less than in the stream. The following factors contribute to deposition in culverts:

1. At moderate flow rates, the culvert cross section is larger than that of the stream, thus the flow depth and sediment transport capacity is reduced.
2. Point bars form on the inside of stream bends and culvert inlets placed at bends in the stream will be subjected to deposition in the same manner. This effect is most pronounced in multiple-barrel culverts with the barrel on the inside of the curve often becoming almost totally plugged with sediment deposits.
3. Abrupt changes to a flatter grade in the culvert or in the channel adjacent to the culvert will induce deposition. Gravel and cobble deposits are common downstream from the break in grade because of the reduced transport capacity in the flatter section.

Deposition usually occurs at flow rates smaller than the design flow rate. The deposits may be removed during larger floods, dependent upon the relative transport capacity of flow in the stream and in the culvert, compaction and composition of the deposits, flow duration, ponding depth above the culvert and other factors.

3.1 Plan

Plan location deals basically with the route the flow will take in crossing the right-of-way. Regardless of the degree of sinuousity of the natural channel within the right-of-way, a crossing is generally accomplished by using a straight culvert either normal to or skewed with the roadway centerline.

Ideally, a culvert should be placed in the natural channel (Figure 1). This location usually provides good alinement of the natural flow with the culvert entrance and outlet and little structural excavation and channel work are required.

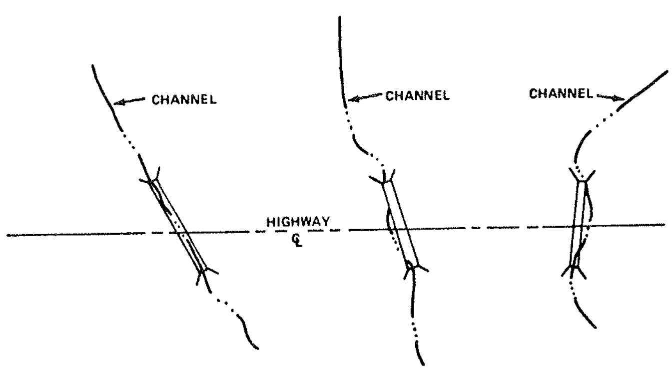

Fig. 1—Culvert located in natural channel.

2.0 Surveys

For purposes of this section, site information from whatever source is broadly classified as survey data. Sources of data include aerial or field survey; interviews; water resource, fish and wildlife, and planning agencies; newspapers; and flood plain zoning studies. Complete and accurate survey information is necessary to design a culvert to best serve the requirements of a site. The individual in charge of the drainage survey should have a general knowledge of drainage design and coordinate the data collection with the hydraulic engineer. The amount of survey data gathered should be commensurate with the importance and cost of the proposed structure.

2.1 Topographic Features

The survey should provide the designer with sufficient data for locating the culvert and determining the hydraulic design controls. All significant physical features and culture in the vicinity of the culvert site should be located by the survey, and especially those features which could be affected by the installation or operation of the culvert. Such features as residences, commercial buildings, croplands, roadways and utilities can influence a culvert design; therefore, their elevation and location should be obtained.

The extent of survey coverage required for culvert design is related to topography and stream slope. In streams with relatively flat slopes, the effects of structures may be reflected a considerable distance upstream and require extensive surveys to locate features which may be affected by the culvert installation.

2.2 Drainage Area

Drainage area is an important factor in estimating the flood potential; therefore, the area of the watershed should be carefully defined by means of a transit-stadia survey, photogrammetric maps, Geological Survey topographic maps[1] or a combination of these.

In locations where accurate definition of drainage areas from maps is difficult, the map information should be supplemented by survey. Noncontributing areas, such as areas contributing to sinkholes and playa lakes may need to be defined. The survey should note land usage, type and density of vegetation, and any manmade changes or developments, such as dams, which could significantly alter runoff characteristics.

[1] Purchase orders for maps should be addressed to Distribution Section, U.S. Geological Survey, 1200 South Eads Street, Arlington, Virginia 22202, for areas east of the Mississippi River, including Puerto Rico and the Virgin Islands, and to Distribution Section, U.S. Geological Survey, Federal Center, Denver, Colorado 80225, for areas west of the Mississippi River, including Alaska, Hawaii, Louisiana, Guam, and American Samoa. Alaskan maps may be ordered from Distribution Section, U.S. Geological Survey, 310 First Avenue, Fairbanks, Alaska 99701.

2.3 Channel Characteristics

The physical characteristics of the existing stream channel should be described by the survey. For purposes of documentation and design analysis, sufficient channel cross sections, a streambed profile and the horizontal alinement should be obtained to provide an accurate representation of the channel, including the flood plain area. The channel profile should extend beyond the proposed culvert location far enough to define the slope and locate any large streambed irregularities, such as headcutting.

General characteristics helpful in making design decisions should be noted. These include the type of soil or rock in the streambed, the bank conditions, type and extent of vegetal cover, amount of drift and debris, ice conditions, and any other factors which could affect the sizing of the culvert and the durability of culvert materials. Photographs of the channel and the adjoining area can be a valuable aid to the designer and serve as excellent documentation of existing conditions.

2.4 Fish Life

Survey data should include information regarding the value of the stream to fish life and the type of fish found in the stream. The necessity to protect fish life and to provide for fish passage can affect many decisions regarding culvert, channel change, and riprap designs and construction requirements for protection of the stream environment. Data required, as well as criteria for design and construction, are generally available from State and Federal fish and wildlife agencies.

2.5 Highwater Information

Reliable, documented highwater data, when available, can be a valuable design aid. Often, the designer must rely upon highwater marks as the only basis on which to document past floods. Highwater marks can also be used to check results of flood estimating procedures, establish highway grade lines and locate hydraulic controls, but considerable experience is necessary to properly evaluate highwater information.

Data related to highwater should be taken in the vicinity of the proposed structure, but it is sometimes necessary to use highwater marks from upstream or downstream points. The location of the highwater mark with respect to the proposed structure should be recorded. Highwater elevations should be referenced to the project datum.

If highwater information is obtained from residents, the individuals should be identified and the length of residency indicated. Other sources for such data might include commercial and school bus drivers, mail carriers, law enforcement officers, highway and railroad maintenance personnel or other persons who have frequently traveled through the area over a long period of time.

Unusual highwater elevations should be examined to ascertain whether

Where location in the natural channel would require an inordinately long culvert, some stream modification may be in order (Figure 2). Such modifications to reduce skew and shorten culverts should be carefully designed to avoid erosion and siltation problems.

Culvert locations normal to the roadway centerline are not recommended where severe or abrupt changes in channel alinement are required upstream or downstream of the culvert. Short radius bends are subject to erosion on the concave bank and deposition on the inside of the bend. Such changes upstream of the culvert result in poor alinement of the approach flow to the culvert, subject the highway fill to erosion and increase the probability of deposition in the culvert barrel. Abrupt changes in channel alinement downstream of culverts may cause erosion on adjacent properties.

In flat terrain, drainage is often provided by excavated channels. Highway planning should be coordinated with the drainage authority where drainage improvements are planned. Where planned channels are not at the location of natural drainage swales, concurrent channel and highway construction is desirable. If concurrent construction is not possible, it will be necessary to provide highway culverts for the existing drainage pattern. The drainage authority may contribute toward modifications to accommodate future channel construction, revise drainage plans to conform with highway culvert locations, or make the necessary changes in highway drainage at the time of channel construction.

3.2 Profile

Most culvert locations approximate the natural streambed though other locations may be chosen for economy in the total cost to construct and

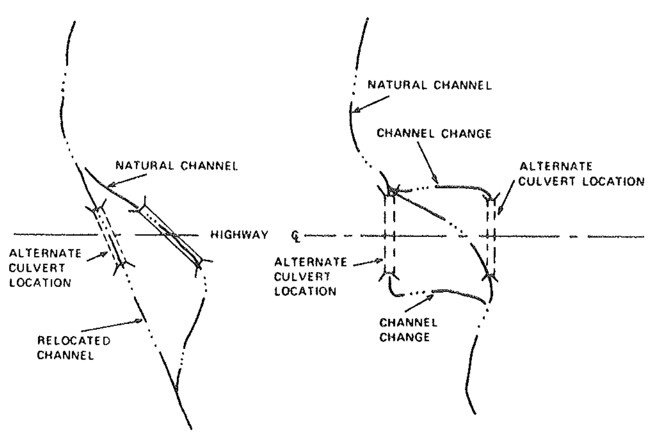

Fig. 2—Methods of culvert location where location in the natural channel would involve an inordinately long culvert.

maintain. Modified culvert slopes, or slopes other than that of the natural stream, can be used to arrest stream degradation, induce sedimentation, improve the hydraulic performance of the culvert (Section 5.6.4), shorten the culvert, or reduce structural requirements. Modified slopes can also cause stream erosion and deposition; therefore, slope alterations should be given special attention to ensure that detrimental effects do not result from the change.

Channel changes often are shorter and steeper than the natural channel. A modified culvert slope can be used to achieve a flatter gradient in the channel so that degradation will not occur.

Figure 3 illustrates possible culvert profiles.

Where channel excavation is planned, culvert invert elevations can be established to accommodate drainage requirements if concurrent channel and highway construction is possible. If concurrent construction is not feasible, a joint or cooperative project should be investigated so that highway culverts can be designed and constructed to serve current highway drainage requirements as well as future needs for land drainage.

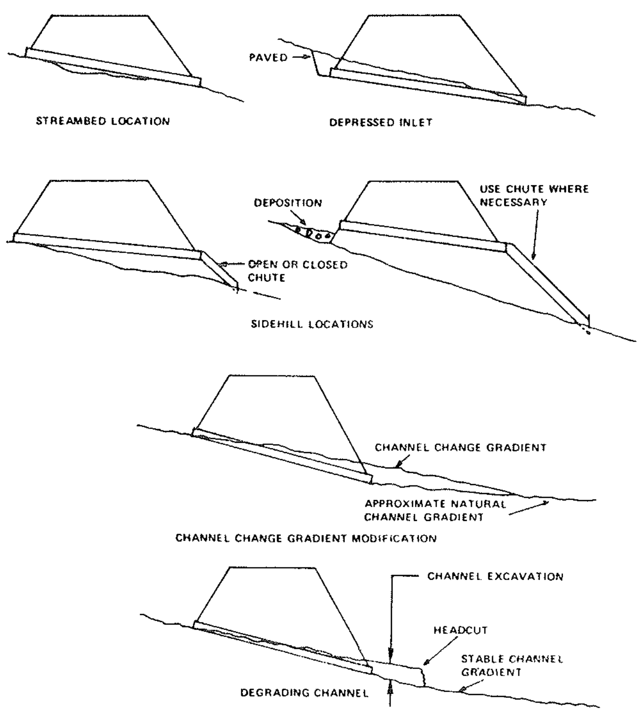

Fig. 3—Possible culvert profiles.

4.0 Culvert Type

Culvert type selection includes the choice of material, shape and cross section and the number of culvert barrels. Total culvert cost can vary considerably depending upon the culvert type selection. Fill height, terrain, shape of the existing channel, roadway profile, allowable headwater, stream stage-discharge and frequency-discharge relationships, cost and service life are some of the factors which influence culvert type selection.

4.1 Shape and Cross Section

The shape of a culvert is not the most important consideration at most sites, so far as hydraulic performance is concerned. Rectangular, arch or circular shapes of equal hydraulic capacity are generally satisfactory. It is often necessary, however, for the culvert to have a low profile because of the terrain or because of limited fill height. Construction cost, the potential for clogging by debris, limitations on headwater elevation, fill height, and the hydraulic performance of the design alternatives enter into the selection of the culvert shape. Several commonly used culvert shapes are discussed in the following paragraphs.

4.1.1 Circular

The most commonly used culvert shape is circular. This shape is structurally efficient under most loading conditions. Various standard lengths of circular pipe in standard strength classes are usually available from local suppliers at reasonable cost. The need for cast-in-place construction is generally limited to culvert end treatments and appurtenances. Design and construction specifications and methods of determining maximum cover for circular concrete and metal pipes are included in publications of the American Association of State Highway and Transportation Officials, Federal Highway Administration, the American Society of Testing Materials, various State highway agencies, and others.

4.1.2 Pipe Arch and Elliptical

Pipe arch and elliptical shapes are generally used in lieu of circular pipe where there is limited cover or overfill. Structural strength characteristics usually limit the height of fill over these shapes except when the major axis of the elliptical shape is laid in the vertical plane. When compared to circular sections, these shapes are more expensive for equal hydraulic capacity because of the additional structural material required.

4.1.3 Box or Rectangular

A culvert of rectangular cross-section can be designed to pass large floods and to fit nearly any site condition. A rectangular culvert lends itself more readily than other shapes to low allowable headwater situations, since the height may be decreased and the total span increased to satisfy the location requirement. The required total span can consist of one or multiple cells. Modified box shapes in the form of hexagons or octagons have been used and proved economical under certain construction situations. The longer construction time required for cast-in-place boxes can be an important consideration in the selection of this type of culvert. Precast box sections have been used to overcome this disadvantage.

4.1.4 Arches

Arch culverts have application in locations where less obstruction to a waterway is a desirable feature, and where foundations are adequate for structural support. Such structures can be installed to maintain the natural stream bottom for fish passage, but the potential for failure from scour must be carefully evaluated. Structural plate metal arches are limited to use in low cover situations but have the advantage of rapid construction and low transportation and handling costs. This is especially advantageous in remote areas and in rugged terrain.

4.1.5 Multiple Barrels

Culverts consisting of more than one barrel are useful in wide channels where the constriction or concentration of flow is to be kept to a minimum. Low roadway embankment offering limited cover may require the use of a series of small openings. The barrels may be separated by a considerable distance in order to maintain flood flow distribution. The practice of altering channel geometry to accommodate a wide culvert will generally result in deposition in the widened channel and in the culvert. Where overbank flood flow occurs, relief culverts with inverts at the flood plain evelation should be used to avoid the need for channel alteration.

In the case of box culverts, it is usually more economical to use a multiple structure than a wide single span. In some locations, multiple barrels have a tendency to catch debris which clogs the waterway. They are also susceptible to ice jams and the deposition of silt in one or more barrels. Alinement of the culvert face normal to the approach flow and installation of debris control structures can help to alleviate these problems.

4.2 Materials

The selection of the material for a culvert is dependent upon several variables such as durability, structural strength, roughness, bedding conditions, abrasion and corrosion resistance, and watertightness.

The more common culvert materials used are:

Concrete (reinforced and non-reinforced)
Steel (smooth and corrugated)
Corrugated aluminum

Other materials which are used in special situations are:

Vitrified clay
Asbestos cement
Plastic
Bituminous fiber
Cast iron
Wood
Stainless Steel

Water and soil environment, construction practices, availability of materials and costs vary considerably depending on location; therefore, listing criteria for selecting culvert material appears to be impracticable as a general guideline. Discussions on the use of certain materials from the durability and hydraulic standpoint are given in Sections 5, 6, and 10.

The most economical culvert is one which has the lowest total annual cost over the design life of the structure. The initial cost should not be the only basis for culvert material selection. Replacement costs and traffic delay are usually the primary factors in selecting a material that has a long service life. If two or more culvert materials are equally acceptable for use at a site, including hydraulic performance and annual costs for a given life expectancy, consideration should be given to material selection by the contractor.

4.3 End Treatments

Culvert end structures, prebuilt or constructed-in-place, are attached to the ends of a culvert barrel to reduce erosion, inhibit seepage, retain the fill, improve the aesthetics and hydraulic characteristics and make the ends structurally stable. Several common types of culvert ends are listed in the following paragraphs.

4.3.1 Projecting

A culvert is considered to have a projecting inlet or outlet when the culvert barrel extends beyond the face of the roadway embankment. This common type of culvert end has no end treatment and is vulnerable to various types of failures. It is the least desirable from the hydraulic standpoint when used as an inlet to corrugated metal, thin-edged barrels. Rigid sectional pipe is vulnerable to displacement at culvert outlets, if not adequately supported. The projecting end is economical but its appearance is not pleasing and use should be limited to smaller culverts placed at minor locations, such as at driveways and in ditches where there would be no safety hazard to traffic.

4.3.2 Mitered

A mitered culvert end is formed when the culvert barrel is cut to conform with the plane of the embankment slope. This type of treatment is used primarily with large metal culverts to improve the aesthetics of the culvert ends. It is structurally inadequate to withstand hydraulic, earth and impact loads unless it is well anchored and protected. The hydraulic performance of this type of inlet is approximately the same as a thin-edged projecting inlet.

4.3.3 Pipe End Sections

Pipe end sections, sometimes called flared or terminal end sections, are prefabricated metal or precast concrete sections placed onto the ends of small culverts (Figure 4). These sections are used to retain the embankment and improve the aesthetics, but usually do not improve the structural stability of the culvert end. Commonly used pipe end sections do not improve the hydraulic performance of culverts appreciably over the performance of a headwall (For inlet improvements, see Section 5.6).

4.3.4 Headwalls and Wingwalls

Headwalls and wingwalls are generally cast-in-place concrete structures commonly constructed on the ends of culvert barrels for the following reasons:

1. To retain the fill material and reduce erosion of embankment slopes;
2. To improve hydraulic efficiency;
3. To provide structural stability to the culvert ends and serve as a counter weight to offset buoyant or uplift forces; and
4. To inhibit piping (Section 6.2).

Although headwalls are sometimes skewed to the culvert barrel to fit the embankment slope, an alinement normal to the direction of flow provides a

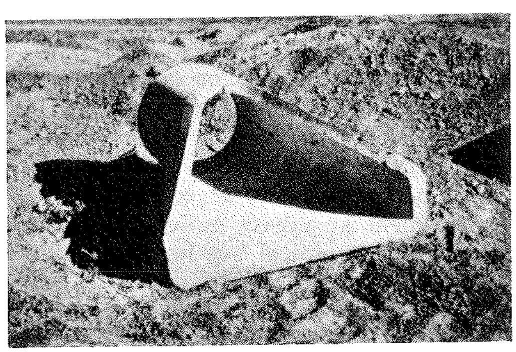

Fig. 4—Flared-end section.

more hydraulically efficient opening. Minor warping of the fill can accommodate this more favorable orientation at most locations (Figure 5).

Wingwalls aid in maintaining the approach velocity, aline and guide drift and funnel the flow into the culvert entrance. Wingwalls should be flush with box culvert barrels to avoid snagging drift.

5.0 Hydraulic Design

The hydraulic design of a culvert consists of an analysis of the performance of the culvert in conveying flow from one side of the roadway to the other. To meet this conveyance function adequately, the design must include consideration of the variables discussed in the following paragraphs.

5.1 Design Flood Discharge

The flood discharge used in culvert design is usually estimated on the basis of a preselected recurrence interval, and the culvert is designed to operate in a manner that is within acceptable limits of risk at that flow rate. Refer to Volume II, Highway Drainage Guidelines, "Guidelines on Hydrology," Section 5, for a discussion of the selection of the design flood frequency and the estimation of flood magnitudes. Recognizing that floods cannot be predicted precisely and that it is seldom economically feasible to design for the very rare flood, all designs should be reviewed for the extent of probable damage should the design flood be exceeded.

Fig. 5—Fill warped to fit culvert headwall normal to culvert.

5.2 Headwater Elevation

Any culvert which constricts the natural stream flow will cause a rise in the upstream water surface to some extent. The total flow depth in the stream measured from the culvert inlet invert is termed headwater. Design headwater elevations and selection of design floods should be based on these risk conditions:

1. Damage to adjacent property;
2. Damage to the culvert and the roadway;
3. Traffic interruption;
4. Hazard to human life; and
5. Damage to stream and floodplain environment.

Potential damage to adjacent property or inconvenience to owners should be of primary concern in the design of all culverts. In urban areas, the potential for damage to adjacent property is greater because of the number and value of properties that can be affected. If roadway embankments are low, flooding of the roadway and delay to traffic are usually of primary concern, especially on highly traveled routes.

Culvert installations under high fills may present the designer an opportunity for use of a high headwater or ponding to attenuate flood peaks. If deep ponding is considered, the possibility of catastrophic failure should be investigated because a breach in the highway fill could be quite similar to a dam failure. When headwater depths will exceed, say 20 to 25 feet for the estimated 100-year flood, the roadway embankment will function as a dam and an appropriate investigation should be made to evaluate the risk in case of the occurrence of a larger flood or blockage of the culvert by debris. In some instances, design of the highway fill as a dam and use of emergency facilities such as spillways and relief culverts should be considered as alternative designs to the construction of larger structures or changes in the roadway profile.

The study of culvert headwater should include verification that watershed divides are higher than design headwater elevations. If the divides are not sufficiently high to contain the headwater, culverts of lesser depths or earthen training dikes may be used, in some instances, to avoid diversion across drainage divides. In flat terrain, drainage divides are often undefined or nonexistent and culverts should be located and designed for least disruption of the existing flow distribution. In these locations culverts can be considered to have a common headwater elevation, though this will not be precisely so. Figure 6 illustrates a design technique that can be used to select culvert sizes in this type of terrain.

5.3 Tailwater

Tailwater is the flow depth in the downstream channel measured from the invert at the culvert outlet. It can be an important factor in culvert hydraulic design because a submerged outlet may cause the culvert to flow full rather than partially full.

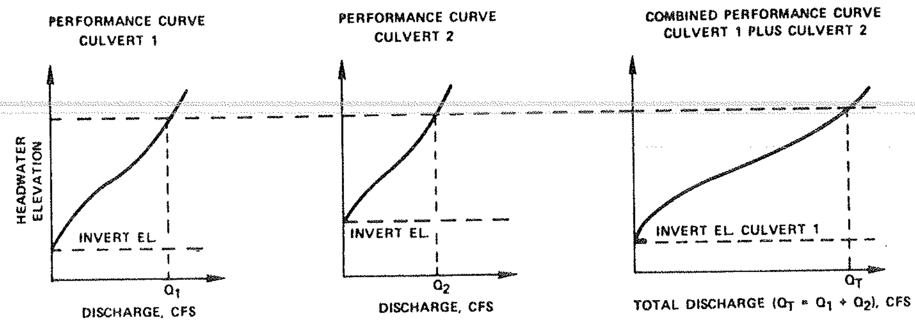

Fig. 6—A design technique for selecting culvert sizes in flat terrain.

A field inspection of the downstream channel should be made to determine whether there are obstructions which will influence the flow depth. Tailwater depth may be controlled by the stage in another stream, headwater from structures downstream of the culvert, reservoir water surface elevations, tide stages or other downstream features.

5.4 Outlet Velocity

The outlet velocity of highway culverts is the velocity measured at the downstream end of the culvert and it is usually higher than the maximum natural stream velocity. This higher velocity can cause streambed scour and bank erosion for a limited distance downstream from the culvert outlet. Local scour at or near the culvert outlet should not be confused with degradation and headcutting in the stream.

Variation in shape and size of a culvert seldom has a significant effect on the outlet velocity except at full flow. The slope and roughness of the culvert barrel are the principle factors affecting outlet velocity. If the outlet velocity of a culvert is believed to be detrimental and it cannot be reduced satisfactorily by changing the barrel roughness or adjusting the barrel slope, it may be necessary to use some type of outlet protection or energy dissipation device. Inspection of existing culverts in the area will be helpful in making this judgment. Various types of outlet treatment are included in Section 5.8 of these guidelines.

5.5 Culvert Hydraulics

The culvert size and type can be selected after the determination of the design discharge, culvert location, tailwater and controlling design headwater. The hydraulic performance of culverts is complex and the flow characteristics for each site should be analyzed carefully to select an economical installation which will perform satisfactorily over a range of flow rates.

Headwater and capacity computations can be made by using mathematical equations, electronic computer programs or nomographs. References 1, 2, and 3 are widely used for the hydraulic design of culverts.

5.5.1 Conditions of Flow

There are two major conditions of culvert flow: (1) flow with inlet control and, (2) flow with outlet control. For each type of control, a different combination of factors is used to determine the hydraulic capacity of a culvert. Prediction of the condition of culvert flow is difficult; therefore, most designers assume that the culvert will flow with the most adverse condition. This assumption is both conservative and expeditious.

5.5.1.1 Inlet Control

A culvert operates with inlet control when the flow capacity is controlled at the entrance by the depth of headwater and the entrance geometry, including the barrel shape, cross-sectional area and the inlet edge. Sketches to illustrate inlet control flow for unsubmerged and submerged projecting entrances are shown in Figure 7.

For a culvert operating with inlet control, the roughness and length of the culvert barrel and outlet conditions (including tailwater) are not factors in determining culvert hydraulic performance. The entrance edge and the

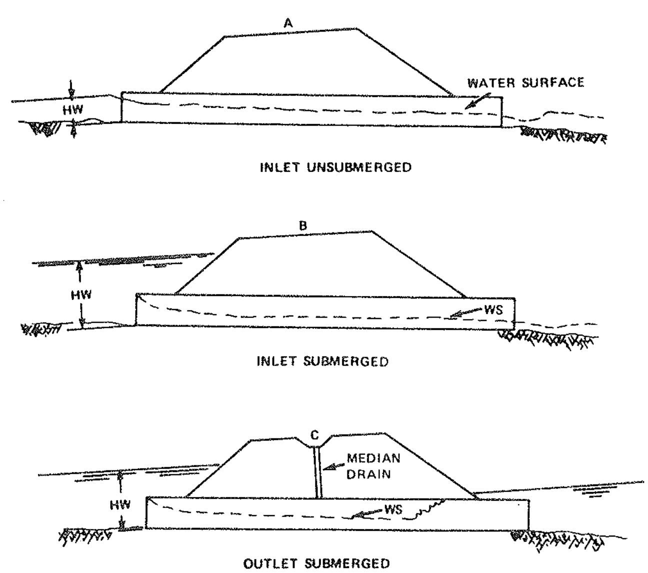

Fig. 7—Inlet control.

overall entrance geometry have much to do with culvert performance in this type of flow; therefore, special entrance designs can improve hydraulic performance and result in a more efficient and economical culvert. Types of entrances are discussed in Section 5.6.

5.5.1.2 Outlet Control

In outlet control, the culvert hydraulic performance is determined by the factors governing inlet control plus the controlling water surface elevation at the outlet and the slope, length, and roughness of the culvert barrel. Culverts operating in outlet control may flow full or partly full, depending on various combinations of the above factors. In outlet control, factors that may affect performance appreciably for a given culvert size and headwater are barrel length and roughness and tailwater depth. Although entrance geometry is a factor, only minor improvement in performance can be achieved by modifications to the culvert inlet.

Typical types of outlet control flow are shown in Figure 8.

5.5.2 Performance Curves

Performance curves are plots of discharge versus culvert headwater depth or elevation. A culvert may operate with outlet or inlet control over the

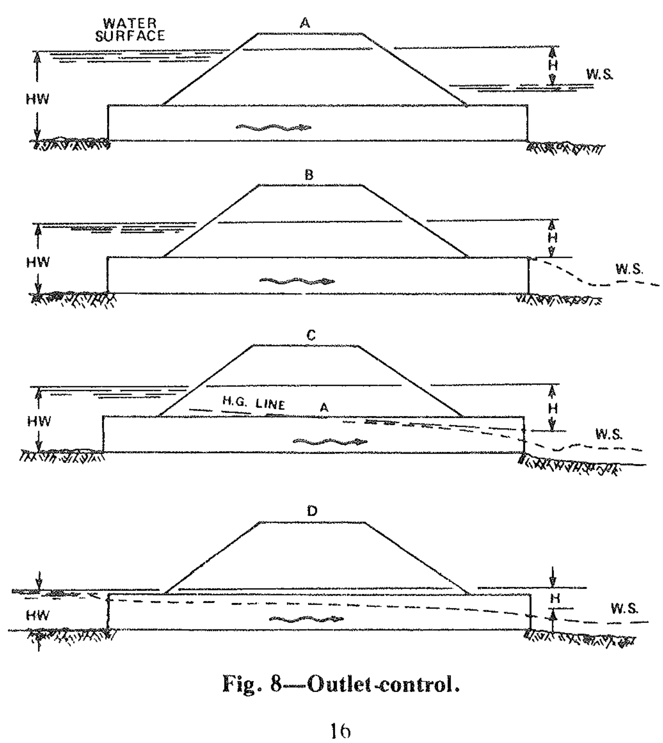

Fig. 8—Outlet-control.

entire range of flow rates or control may shift from the inlet to the outlet. For this reason, it is necessary to plot both inlet and outlet control curves to develop the culvert performance curve.

In culvert design, the designer usually selects a design flood frequency, estimates the design discharge for that frequency and sets an allowable headwater elevation based on the selected design flood and considerations cited in Section 5.2. There are, however, uncertainties in estimating flood peaks for any desired recurrence interval and a probability or chance that the design frequency flood will be exceeded during the life of the project. (See Volume II, Highway Drainage Guidelines, Guidelines for Hydrology). Because of these uncertainties, it is necessary for the designer to develop information from which he can evaluate the culvert performance, or headwater—capacity relationship, over a range of flow rates. With this information on culvert performance, the risks involved in the event of large floods can be evaluated. This evaulation should include the probability of occurrence, the possibility of traffic interruption by flow over the highway, and damages that would occur to the highway and other property.

Performance curves aid in the selection of the culvert type, including size, shape, material, and inlet geometry, which fulfills site requirements at the least annual cost. The curves also may reveal opportunities for increasing the factor of safety and improving the hydraulic capacity at little or no increase in cost. A typical culvert performance curve is shown in Figure 9. Flood frequency has been added to the abscissa to aid in evaluating the risk of exceeding the design headwater with the selected culvert design.

5.6 Entrance Configurations

Entrance configuration is defined as the cross sectional area and shape of the culvert face and the type of inlet edge. When a culvert operates in inlet control, headwater depth and the entrance configuration determine the culvert capacity and the culvert barrel usually flows only partially full. Entrance geometry refinements can be used to reduce the flow contraction at the inlet and increase the capacity of the culvert without increasing the headwater depth. The amount or degree of refinement warranted is dependent upon the slope and roughness of the culvert barrel, headwater elevation controls, tailwater, design flood discharge and the probability of exceedance, risk of damage, construction costs, the safety factor incorporated into the design, and other factors. Performance curves are an indispensible aid in evaluating the degree of inlet refinement that is warranted (3).[2]

In connection with inlet improvements, two points should be emphasized. First, culverts operating in outlet control usually flow full at the design flow rate. Therefore, inlet improvements on these culverts only reduce the entrance loss coefficient, k_e, which results in only a small decrease in the required headwater elevation. Second, inlet improvements are made for the purpose of causing a culvert flowing with inlet control to flow full or nearly full at the design discharge. It should be recognized that outlet control may

[2] Underlined numbers in parenthesis refer to publications listed in Section 15.0, References.

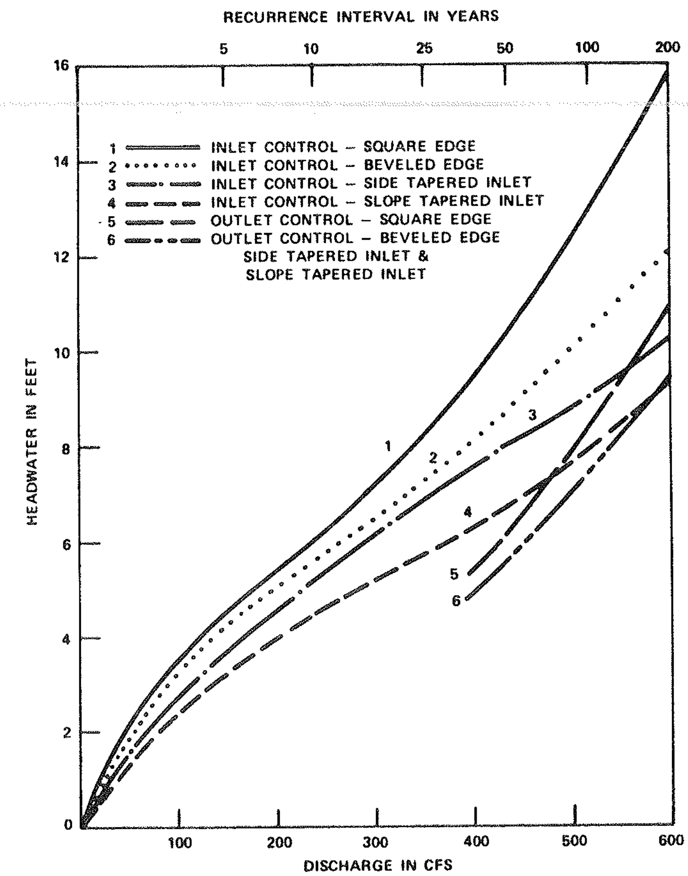

Fig. 9—Performance curves for single box culvert 90 degree wingwall.

govern for discharges larger than the design flood peak and outlet control has a more rapidly increasing headwater elevation requirement for increasing discharges than inlet control. Because of uncertainties in estimating flood peaks and the chance that the design frequency flood will be exceeded, the risk of damage from larger floods may warrant incorporating an increased factor of safety in culvert capacity at some sites.

Table 1 gives entrance loss coefficients, k_e, for computing entrance losses for outlet control flow. In inlet control, the effect of the entrance configuration is inherent in empirical charts and nomographs for the headwater—discharge relationships developed from research (1, 2, 3).

Various types of culvert entrances are shown in Figures 10 through 18 and discussed in the following paragraphs. Reference 3 contains a full discussion of inlet improvements, design charts and procedures.

TABLE 1—ENTRANCE LOSS COEFFICIENTS

Outlet Control, Full or Partly Full

Entrance head loss $H_e = k_e\,(V^2/2g)$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Miltered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Miltered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both *inlet* and *outlet* control. Some end sections, incorporating a *closed* taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

54

5.6.1 Conventional

Commonly used inlets consist of projecting culvert barrels or projecting inlets, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope, or step mitered to approximate the fill slope. For a given headwater elevation, the conventional

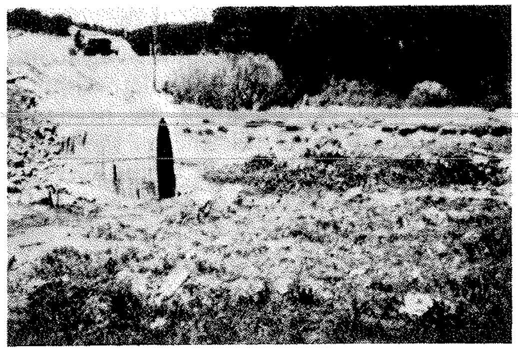

Fig. 10—Thin-edge projecting inlet.

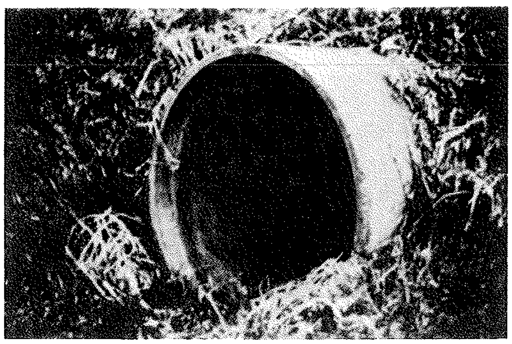

Fig. 11—Groove end projecting inlet.

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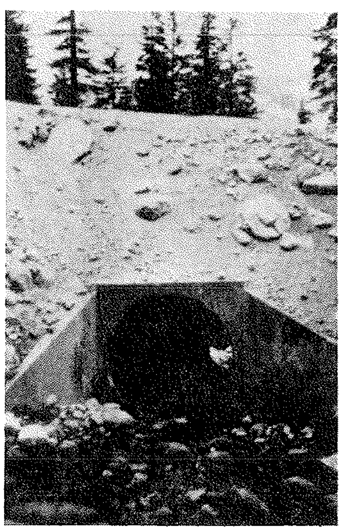


Fig. 12—Square edge inlet in headwall with wingwalls.

bell or groove end of a concrete pipe has a greater capacity than a square-edged inlet, whether projecting or in a headwall, and a square-edged inlet has greater capacity than a thin edged, mitered or projecting inlet. Although the entrance loss coefficient cannot be used in computing the headwater elevation for culverts operating with inlet control, the efficiency of the various inlets for both inlet and outlet control is in general indicated by the k_e values shown in Table 1. Conventional inlets are shown in Figures 10 through 14.

Fig. 13—Mitered inlet with slope paving.

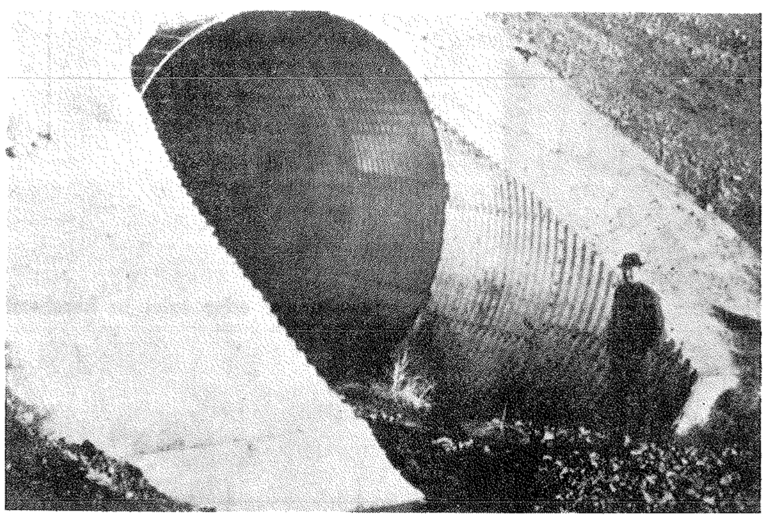


Fig. 14—Step-mitered inlet.

5.6.2 Beveled

Bevels similar to but larger than chamfers on the inlet edges of a culvert are the simplest type of inlet improvement. The bevels may be plane surfaces or rounded and are proportioned according to culvert barrel or face dimensions. The top and sides of box culverts and the perimeter of other shapes should be beveled, except that bevels may be omitted from that portion of the perimeter of round and arch shapes which is tangent to an inlet apron. The bell or groove end of a concrete pipe is equal in performance to a beveled entrance and is superior to the performance of a square-edged inlet in a headwall, as when the groove end is cut off. The entrance of a thin-walled culvert can be improved by incorporating the thin edge in a headwall or in a headwall with bevels.

Bevels also improve the performance of culverts operating with outlet control, but not as much as with inlet control. The entrance loss coefficient, k_e, is reduced by the use of beveled edges and they should be considered since little additional cost is involved.

A beveled inlet is shown in Figure 15.

5.6.3 Side-Tapered Inlets

Further increase in culvert capacity by reducing the flow contraction at the entrance is possible by use of an enlarged face area and a transition from the enlarged face to the culvert barrel. On a box culvert, this is called a side-tapered inlet because the inlet face is the same height as the culvert barrel and the transition from face size to barrel size is accomplished by tapering the sidewalls. Side-tapered or flared inlets for pipe culverts may have a face in the shape of an oval, a circle, or a pipe-arch. Flared or warped wingwalls or a simple headwall may be used with this type of inlet.

The intersection of the transition section and the barrel is termed the throat section. For side-tapered inlets, the hydraulic control may be at the face or at the throat. Since flow contraction at the throat is less than at the face and the throat is at a lower elevation, it is advantageous to design side-tapered inlets so that control will be at the throat. This is accomplished

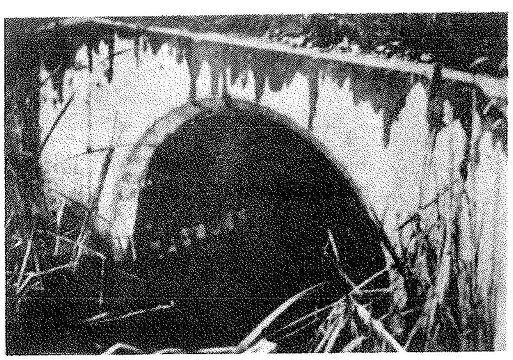


Fig. 15—Beveled inlet with headwall.

by making the face sufficiently large that control will be at the throat at most flow rates.

The advantages of a side-tapered inlet for culverts flowing in inlet control are increased flow capacity or a lower required headwater elevation for a given flow rate and a possible reduction in the size of culvert barrel required. Some increase in forming costs may be experienced for the transition or inlet section, but any such increased cost has been difficult to detect in those built to date.

Side-tapered inlets are shown in Figures 16 and 17.

5.6.4 Slope-Tapered Inlets

Slope-tapered inlets are similar to side-tapered inlets except that the slope in the transition section is steeper than the slope of the culvert barrel. With control at the throat, more head is available at the control section and at given headwater elevations, culvert capacity is greater than with other inlet configurations. The total annual cost of various alternate designs should be considered in culvert selection. If a slope-tapered inlet is hydraulically feasible, the increased costs for structural excavation should be offset by advantages of increased culvert flow capacity and/or reduced culvert barrel size and cost.

Slope-tapered inlets can be used on either rectangular or circular culverts, but circular culverts require a special transition to the barrel section.

Figure 18 shows a slope-tapered inlet under construction.

A full discussion of inlet improvements and design aids are contained in Reference 3.

5.7 Barrel Characteristics

In inlet control flow, culvert barrel characteristics of roughness, length and slope do not affect culvert capacity. It should be understood, however, that these characteristics often determine whether or not the culvert will flow with inlet or outlet control. With a given culvert slope, a rough pipe will flow

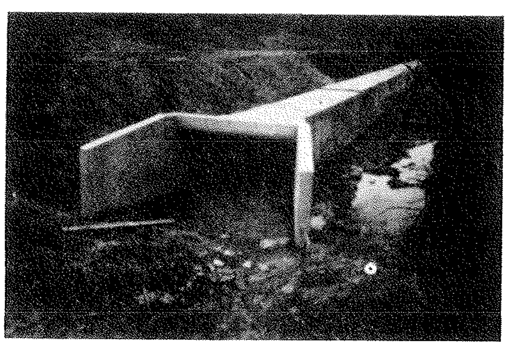


Fig. 16—Side-tapered inlet on box culvert.

TRANSPORTATION TECHNOLOGY SUPPORT
FOR DEVELOPING COUNTRIES

COMPENDIUM 3

Small Drainage Structures

Pequeñas estructuras de drenaje

Petits ouvrages de drainage

prepared under contract AID/OTR-C-1591, project 931-1116,
U.S. Agency for International Development

Transportation Research Board
Commission on Sociotechnical Systems
National Research Council

NATIONAL ACADEMY OF SCIENCES WASHINGTON, D.C. 1978

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Small drainage structures-Pequeñas estructuras de drenaje-Petits ouvrages de drainage.

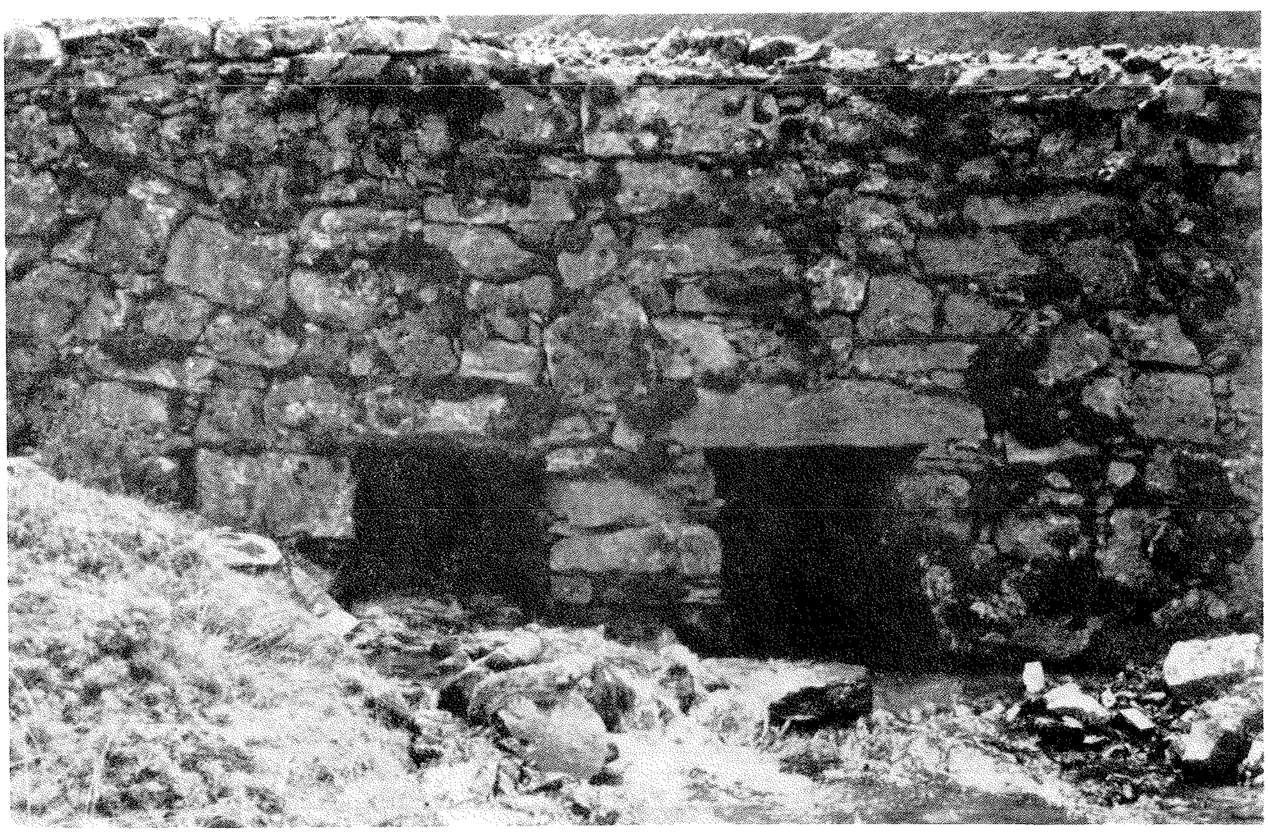
(Transportation technology support for developing countries; compendium 3)
"Prepared under contract AID/OTR-C-1591, project 931-1116, U.S. Agency for International Development."
English, French, and Spanish.
Bibliography: p.
Includes index.
1. Road drainage—Addresses, essays, lectures. 2. Culverts—Addresses, essays, lectures. I. Title. II. Title: Pequeñas estructuras de drenaje. III. Title: Petits ouvrages de drainage. IV. Series.
TE215.N3 1978a 625.7'34 78-32156
ISBN 0-309-02810-8

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: Dual stone box culvert in Bolivia.



In general, culvert outlet end treatment does not affect culvert capacity. The exception to this would be an energy dissipation device which raised the pressure line or effective tailwater at the outlet and caused the culvert to flow with outlet control rather than inlet control. Outlet structures are used for three purposes:

1. to retain the embankment,
2. to provide structural support for the end of the culvert, (Section 6.1) and
3. to inhibit scour damage to the roadway embankment, downstream channel and adjacent property.

Scour at culvert outlets is caused by high velocity flow, flow confined to a lesser width and greater depth than in the natural channel and eddies resulting from flow expansion. Scour prediction is somewhat subjective since the velocity at which erosion will occur is dependent upon the characteristics of the channel bed and bank material, velocity and depth of flow in the channel and at the culvert outlet, velocity distribution, and the amount of sediment and other debris in the flow. Scour developed at the outlet of similar existing culverts in the vicinity is always a good guide in estimating potential scour at the outlet of proposed culverts.

Scour does not develop at all suspected locations because the susceptibility of the stream to scour is difficult to assess and the flow conditions which will cause scour do not occur at all flow rates. At locations where scour is expected to develop only during relatively rare flood events, the most economical solution may be to repair damage after it occurs.

At many locations use of simple outlet treatment such as headwalls, cutoff walls and aprons of concrete or riprap will provide adequate protection against scour. At other locations, use of a flatter slope or a rougher culvert material may be sufficient to prevent damage from scour.

When the outlet velocity will greatly exceed the maximum velocity in the downstream channel, consideration should be given to energy dissipation devices such as stilling basins and riprap basins. It should be recognized, however, that such structures are costly, many do not provide protection over a wide range of flow rates, some require a high tailwater to perform their intended function, and the outlet velocity of most culverts is not high enough to form a hydraulic jump which is efficient in dissipating energy. Therefore, selection and design of an energy dissipation device to meet needs at a site requires a thorough study of expected outlet flow conditions and the performance of various devices. The cost of formal dissipation devices for the design flow rate may be such that outlet protection for a lower discharge is indicated and some damage from larger floods accepted.

Design information for some of the more commonly used energy dissipators is contained in References 5 through 11.

6.0 Special Hydraulic Considerations

In addition to the hydraulic considerations discussed in the preceding sections, other factors must be considered in order to assure the integrity of culvert installations and the highway.

6.1 Anchorage

The forces acting on a culvert inlet during high flows are variable and highly indeterminate. Vortexes and eddy currents cause scour which can undermine the culvert inlet, erode the embankment slope and make the inlet vulnerable to failure. Flow is usually constricted at the inlet and inlet damage (Figure 19) or lodged drift can accentuate this constriction. The large unequal pressures resulting from this constriction are, in effect, buoyant forces which can cause entrance failures, particularly on corrugated metal pipe with mitered, skewed, or projecting ends (12).

Anchorage at the culvert entrance helps to protect against these failures by increasing the dead load on the end of the culvert, protecting against bending damage and by protecting the fill slope from the scouring action of the flow. End anchorage can be in the form of slope paving, concrete headwalls, or grouted stone, but the culvert end must be anchored to the end treatment to be effective. In some locations, prefabricated metal end sections should also be anchored to increase their resistance to failure.

Outlet ends of culverts need anchorage at many locations. Sectional rigid pipe is susceptible to separation at the joints when scour undermines the outlet end. Tiebars are commercially available to prevent separation of concrete pipe joints. Metal culvert ends projected into ponds or tidal waters or through levees are susceptible to failure from buoyant forces if tide gates are used or if the ends are damaged by debris.

Figures 20 and 21 show culverts which failed from buoyant forces at the inlet end.

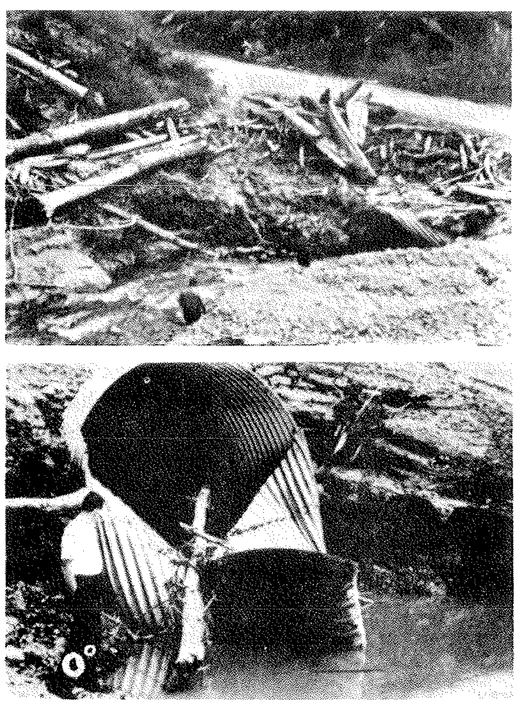


Fig. 19—Damage to culvert inlets from hydraulic forces and drift.

Fig. 20—Culvert and roadway fill failure from buoyant forces. Culvert carried downstream.

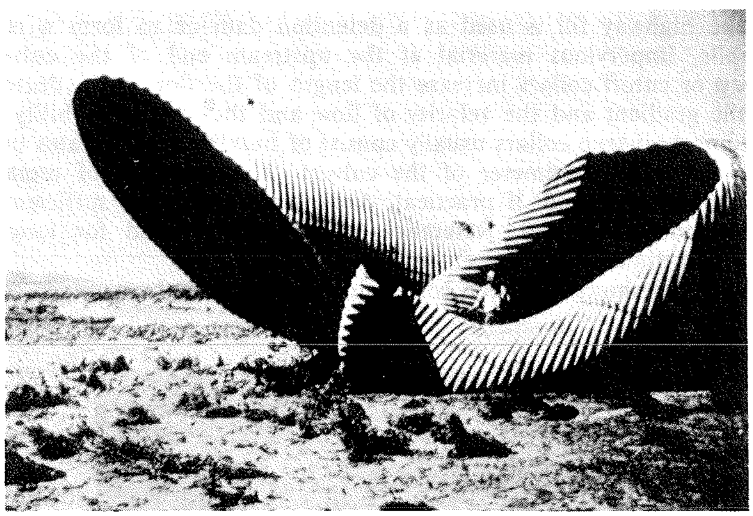


Fig. 21—Bending at culvert inlet from buoyant forces. Both ends of culvert are seen in this view.

6.2 Piping

Piping is a phenomenon caused by seepage along a culvert barrel which removes fill material, forming a hollow similar to a pipe, hence the term piping (Figure 22). Fine soil particles are washed out freely along the hollow and the erosion inside the fill may ultimately cause failure of the culvert or the embankment. Piping may also occur through open joints into the culvert barrel.

The possibility of piping can be reduced by decreasing the velocity of the seepage or by decreasing the size of the moving stream. Methods of achieving these objectives are discussed in the following sections.

6.2.1 Joints

In order to decrease the velocity of the seepage flow, it is necessary to increase the length of the flow path and thus decrease the hydraulic gradient. The most direct flow path for seepage and thus the highest hydraulic gradient is through open pipe joints. Therefore, it is important that culvert joints be as water tight as practical. If piping through joints could become a problem, flexible, long-lasting joints should be specified as opposed to mortar joints.

6.2.2 Anti-Seep Collars

Piping should be anticipated along the entire length of the culvert when ponding above the culvert is expected for an extended length of time, such as when the highway fill is used as a detention dam or to form a reservoir. Headwalls, impervious material at the upstream end of the culvert and anti-seep or cutoff collars increase the length of the flow path, decrease the hydraulic gradient and the velocity of flow and thus the probability of pipe formation. Anti-seep collars usually consist of bulkhead type plates or blocks around the entire perimeter of the culvert. They may be of metal or of reinforced concrete and, if practical, dimensions should be sufficient to key into impervious material. Reference 13 is recommended for longitudinal spacing and dimension requirements.

Figure 23 shows anti-seep collars installed on a culvert under construction.

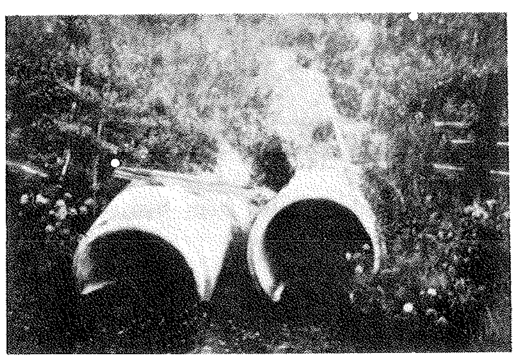


Fig. 22—Void from piping along culvert barrel. Inadequate space between pipes for good compaction.

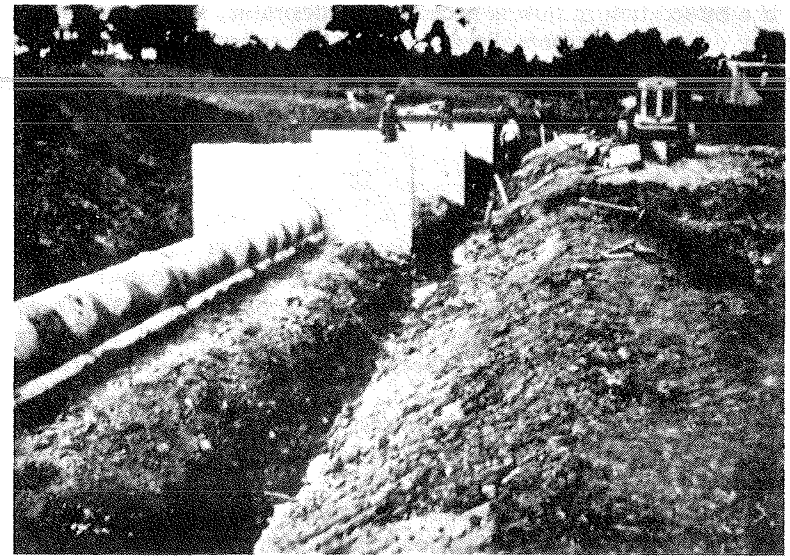


Fig. 23—Anti-seep collars.

6.2.3 Weep Holes

Weep holes are sometimes used to relieve uplift pressure. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and to prevent the formation of piping channels. The filter materials should be designed as underdrain filter so that it will not become clogged and so that piping cannot occur through the pervious material and the weep hole. Plastic woven filter cloth (6) should be placed over the weep hole in order to keep the pervious material from being carried into the culvert.

Weep holes are not generally required in culverts and their use is becoming less prevalent. If drainage of the fill behind the culvert wall is believed necessary, a separate underdrain system should be installed.

6.3 Junctions and Bifurcations

It is sometimes necessary to combine the flow of two culverts into a single barrel. The junction should be designed so that a minimum amount of turbulence and adverse effect on each branch will result. This is accomplished by considering the flow momentum in each branch and numerous other variables such as the timing of peak flows, low flow in one branch and high flow in the other. Supercritical flow velocities add to the complexity of the problem. References 14 and 15 and other technical publications treat the subject of junctions for supercritical flow. In critical locations, laboratory verification of junction design is advisable.

If a bifurcation in flow is necessary or desirable, it is recommended that the flow division be accomplished outside the culvert barrel. Problems with clogging by debris and the desired proportioning of flow between branches can be handled much more easily outside of the culvert.

6.4 Training Walls

Where supercritical flow conditions prevail in a curved approach to a culvert, training walls are needed to aline flow with the culvert inlet and to equalize flow rates in the barrels of multiple barrel culverts. In locations where overtopping of the channel or culvert or inefficient operation could result in catastrophic failure, laboratory verification of the training wall design is advisable.

Training walls may also be required at culvert outlets to aline flow with the downstream channel if this alinement cannot be accomplished in the culvert barrel.

Design of the training walls at the culvert inlet shown in Figure 24 was verified by laboratory testing and the walls have been proven by operation during floods.

6.5 Sag Culverts

A sag culvert, often called an inverted siphon, is not a siphon because the pressure in the barrel is not below atmospheric. Sag culverts of pipe or box section are used extensively to carry irrigation water under highways. They are used infrequently for highway drainage and should be avoided on intermittent or alluvial streams because of problems with siltation and stagnation.

Hydraulically, a sag culvert operates with outlet control and losses through the culvert can be computed by the procedures used for conventional culverts. Bend losses can be added to the usual losses, but these losses are usually negligible because of low velocities. Bend loss coefficients can be found in Reference 13.

Fig. 24—Training walls at culvert entrance.

6.6 Irregular Alinement

At some locations, it may be desirable to incorporate bends, either in plan or profile, in culvert alinement. When irregular alinement is advisable or desirable, bends should be as gradual and as uniform as is practical to fit site conditions. Changes in alinement may be accomplished either by curves or angular bends. When large changes are necessary, mild bends, e.g., 15° at intervals of 50 feet, should be used. Passage of debris should be considered in selecting the angle, interval and number of bends used to accomplish the change in alinement.

If the culvert will operate with inlet control, bend losses do not enter into the headwater computation. If it will operate with outlet control, bend losses will be small. In critical locations, they should be calculated and added to the usual losses. Bend loss coefficients can be found in Reference 12.

6.7 Cavitation

The phenomenon known as cavitation occurs as a result of local velocity changes at surface irregularities which reduce the pressure to the vapor limit of the liquid. Tiny vapor bubbles form at the point of lowest pressure and are carried downstream into a zone of higher pressure where they collapse. As the countless bubbles collapse, extremely great local pressure is transmitted radially outward at the speed of sound, followed by a negative pressure wave which may lead to a repetition of the cycle. Boundary materials in the vicinity are subjected to rapidly repeated stress reversals and may fail through fatigue. (16) Surface pitting is the first sign of such a failure.

Cavitation is seldom a problem in highway culverts because of relatively low velocities and because flow rates are not sustained for a long period. Abrasion damage is sometimes mistaken for cavitation damage.

6.8 Tidal Effects and Flood Protection

Where areas draining through culverts are adversely affected by tide or flood stages, flap gates may be desirable to prevent backflow (Section 6.1). Sand, silt, debris or ice will cause these gates to require considerable maintenance to keep them operative. Head losses due to the operation of flap gates may be computed using loss coefficients furnished by the manufacturer.

7.0 Multiple Use Culverts

Culverts often serve purposes in addition to drainage. There are cost advantages of multiple use but one purpose or the other is often inadequately served. The cost advantages of multiple use should be weighed against the possible advantages of separate facilities for each use.

7.1 Utilities

It is sometimes convenient to locate utilities in culverts, particularly if jacking, boring or an open cut through an existing highway can be avoided by such a location. The space occupied in the culvert is usually relatively small and the obvious effects on culvert hydraulic performance insignificant. Consideration of this multiple use, however, should include recognition of the flood flow and debris hazard to the utility and the probability of reduced culvert capacity from debris caught on the utility line. Also, increased stream scour often occurs at pipelines at the upstream and downstream ends of culverts. This multiple use is not generally recommended if separate facilities are practicable.

7.2 Stock and Wildlife Passage

Culverts can serve both for drainage and for stock and wildlife passes. Culvert size may be determined either by hydraulic requirements or by criteria established for the accommodation of the stock or game which will use the structure. Criteria for the accommodation of stock and wildlife is not included in these Guidelines. Scour protection at the outlet may be necessary to insure acceptable access conditions for livestock. As with other multiple-use culverts, satisfactory performance for both intended uses should be assured or separate facilities provided.

7.3 Land Access

Culverts often serve both as a means of land access and drainage, particularly on highways with controlled access. This use is common in areas where land use on both sides of the highway is under common control. The culvert size will generally be determined by the physical dimensions of the equipment or vehicles which will make use of the facility. Scour protection not considered necessary for hydraulic reasons may be required at the outlet to facilitate access to the culvert. A smaller culvert at an offset location or at a lower elevation than the multiple-use culvert may be required to accommodate low flows. Where a low-flow culvert is placed at a lower elevation than the multiple-use culvert, precautions against headcutting from the stream to the outlet of the multiple-use culvert may be necessary. Good drainage at the culvert ends is necessary to the successful use of culverts for land access.

7.4 Fish Passage

In some locations, the need to accommodate migrating fish is an important consideration in the design of a stream crossing. New roadway locations should be coordinated with State fish and wildlife agencies at an early date so stream crossings which require fish passage can be identified. These agencies normally request provision for fish passage for all streams with fish migrations and streams that have suitable habitat to support fish runs. Questions regarding fish passage criteria should be reviewed in the field during project development and discussed with the agency making the request. At some locations, the agency may request that the culvert design include a fish barrier to prevent migration of rough fish into an upstream lake.

When fish passage is requested, the priority order of alternatives is: (a) highway relocation to avoid the crossing, (b) construction of a bridge, and (c) construction of a suitable culvert.

Many fish and wildlife agencies have established design criteria for fish passage through culverts. These include maximum allowable velocity, minimum water depth, maximum culvert length and gradient, type of structure, and construction scheduling.

Several types of culvert installations have been used satisfactorily for fish passage (17, 18). These include:

1. *Open Bottom Culverts:*

Culverts supported on spread footings to permit retention of the natural streambed. The culvert size must be adequate to maintain natural stream velocities at moderate flows and the foundation must be in rock or scour resistant material (Figure 25).

2. *Oversized or Depressed Culverts:*

Oversized culverts with the bottom of the culvert placed below the streambed so that gravel will deposit and develop a nearly natural streambed within the culvert (Figure 26). Sometimes, baffles are necessary to hold gravel and rock in place.

3. *Culverts with Baffles:*

Many baffle configurations have proved to be satisfactory. The baffle geometry shown in Figure 27 is used by several States in the Pacific Northwest. These baffles are not satisfactory for steep gradients or large flows. With relatively steep gradients (2 to 5 percent), they aid passage only to a flow depth of about one foot over the baffle crest.

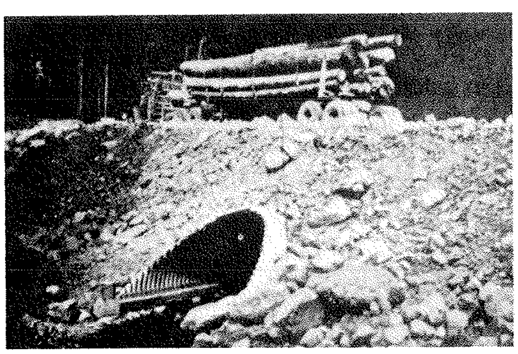


Fig. 25—Culvert on footings to retain natural streambed for fish passage.

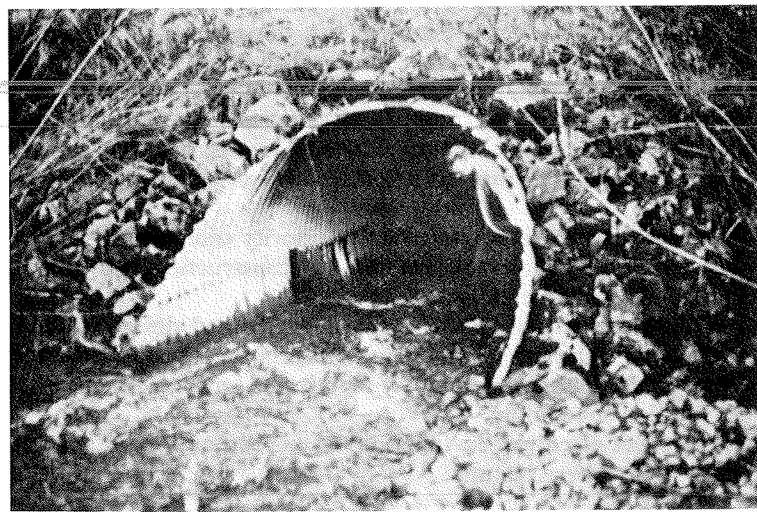


Fig. 26—Culvert invert placed below streambed. Baffles used to hold gravel in place and provide natural streambed for fish passage.

4. *Weirs:*

70

Weirs in the channel downstream of the culvert, constructed so as to maintain the desired depth through the culvert, is probably the most practical way to meet a minimum water depth requirement for a given species of fish. (Figure 28). The weir must be of substantial design to withstand flood flows, and provisions must be made for fish to bypass the weir. The by-pass provided is dependent on the species of fish. References 19 and 20 will aid in the design of weirs and bypasses for fish passage.

5. *Special Treatment:*

In wide, shallow streams, one barrel of a multiple barrel culvert can be depressed to carry low flow or weirs can be installed at the upstream end of some barrels to provide for fish passage through other barrels at low flow.

The addition of baffles in culverts to aid fish passage may cause the culvert to flow with outlet control at relatively low flow rates. Neglecting the culvert area occupied by the baffles does not adequately account for energy losses from turbulence generated by the baffles. Reference 20 is recommended for the determination of hydraulic performance of culverts with baffles.

8.0 Irrigation

Conventional culverts and sag culverts are often used to convey irrigation water under a highway. Freeboard in irrigation canals is usually small and

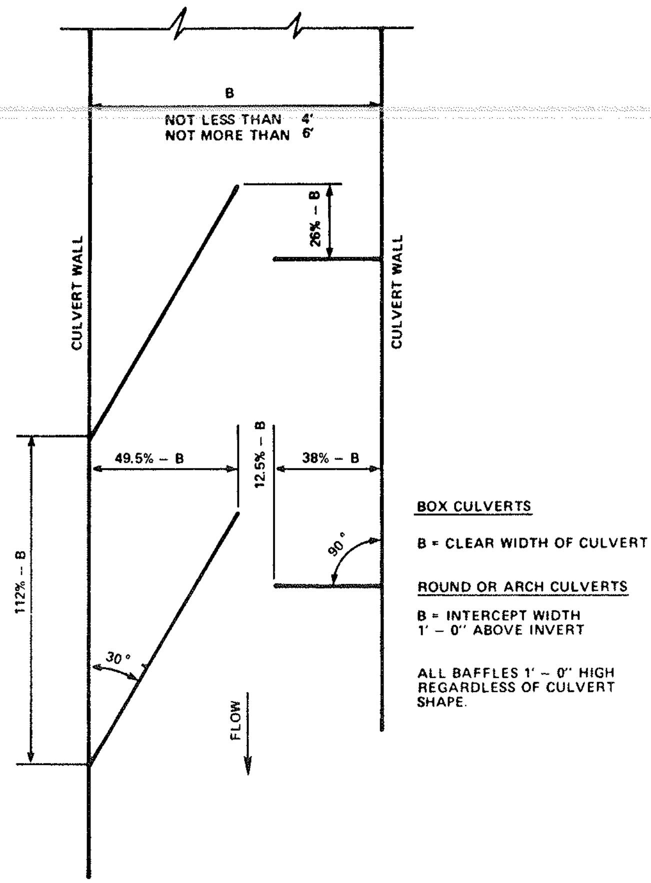


Fig. 27—Baffle geometry used in culverts designed for fish passage.

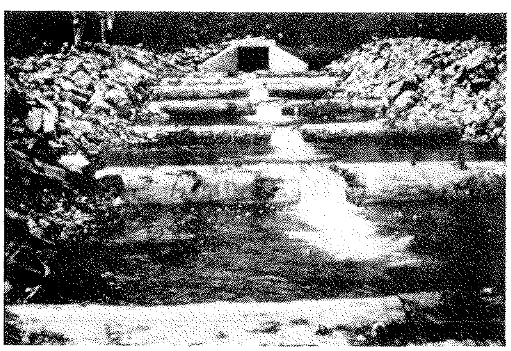


Fig. 28—Weirs downstream of culvert to facilitate fish passage.

the hydraulic design of the culvert should be such that service to irrigable lands will not be impaired by loss of head in the culvert.

Culvert construction in irrigation canals should be scheduled to avoid conflict with the irrigation season and supervised carefully to minimize the possibility of sediment disrupting the water supply.

9.0 Debris Control

Accumulation of debris at a culvert inlet can result in the culvert not performing as designed. The consequences may be damages from inundation of the road and upstream property.

The designer has three options for coping with the debris problem: retain the debris upstream of the culvert, attempt to pass debris through the culvert, or use a bridge (22, 23).

If the debris is to be retained by an upstream structure or at the culvert inlet, frequent maintenance may be required. If debris is to be passed through the structure or retained at the inlet, a relief opening should be considered, either in the form of a vertical riser or a relief culvert placed higher in the embankment (Figure 29).

It is often more economical to construct debris control structures after problems develop since debris problems do not occur at all suspected locations.

9.1 Debris Control Structure Design

The design of a debris control structure must be preceded by a thorough study of the debris problem. Among the factors to be considered are:

(a) Type of debris;
(b) Quantity of debris;
(c) Expected changes in type and quantity of debris due to future land use;

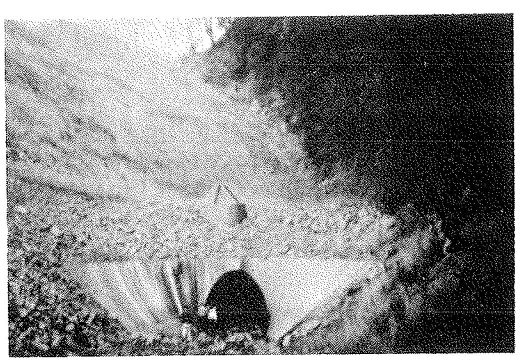


Fig. 29—Vertical riser for relief.

(d) Streamflow velocity in vicinity of culvert entrance;
(e) Maintenance access requirements;
(f) Availability of storage area;
(g) Standard of planned maintenance for debris removal; and
(h) Assessment of damage due to debris clogging, if protection is not provided.

9.2 Maintenance

Provisions for maintenance access are necessary for debris control structures. For high embankments, this may be difficult. If access to the debris control structure is not practical, a parking area for mechanical equipment such as a crane may be necessary in order to remove debris without disrupting traffic.

Many debris barriers require cleaning after every storm. The standard or frequency of maintenance should be considered in selecting the debris control structure. If a low standard of maintenance is anticipated, the designer should choose to pass the debris through the structure.

10.0 Service Life

Commonly used culvert materials are durable at most locations but some soil and water environments are hostile and service life must be a consideration in material selection and culvert design. Conditions which affect the service life of culvert materials are corrosion, abrasion, and freezing and thawing action. Measures to increase service life are sometimes costly and the total annual cost should be considered when designs are prepared. Periodic culvert replacement may be the most feasible alternative. Driveway culverts, for instance, are generally easy to replace and traffic service would not be a problem when replacement becomes necessary. Culverts under high traffic volume highways or high fills, on the other hand, are more difficult and costly to repair or replace and more precaution against failure from a hostile environment is warranted.

Many of the conditions which affect service life can be evaluated and service life estimated prior to the selection of culvert material. The type and degree of protection needed can then be determined (24 through 30). One of the most reliable methods available to the designer is to examine existing culverts in the same stream channel or in similar streams in the same area.

10.1 Abrasion

Abrasion loss is the erosion of culvert materials by the bedload carried by streams (Figure 30). The principal factors to be considered are the frequency and duration of runoff events which transport significant amounts of abrasive materials, the character and volume of the bedload, and the resistance of the culvert material to abrasion.

Fig. 30—Loss of culvert material from abrasion.

In some locations culverts can be protected from abrasion by use of debris control structures to remove the abrasive sediment load from the flow (Section 9.2).

Provision for abrasive wear can be made by the use of sacrificial thickness of structural material in the invert. In metal culverts, the sacrificial material may be either additional metal thickness or portland cement concrete invert paving. Provision for abrasion in concrete culverts generally consists of requiring additional cover over reinforcing steel and more durable concrete mixes.

Invert treatment of planking with metal plate or railroad rails, channels, or other steel shapes placed longitudinally in the bottom of the culvert can be used where severe abrasion is anticipated or experienced (Figure 31).

10.2 Corrosion

Environmental conditions that are generally considered to contribute to the corrosion of metal culvert pipe are acidic and alkaline conditions in the soil and water and the electrical conductivity of the soil. Another contributing factor in corrosion is the frequency and duration of flows transporting bedloads which abrade or otherwise damage protective coatings.

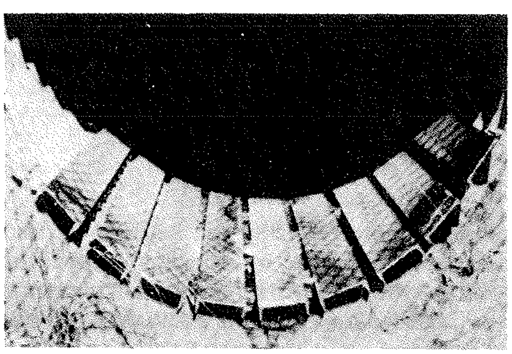


Fig. 31—Downstream end of culvert invert treatment for protection against abrasion.

Salt water causes corrosion of steel, and depending on the salt concentration, will corrode aluminum but experience with aluminum in salt water environments to date indicates that aluminum culverts are fairly resistant to corrosion at such locations. Aluminum should not be used in alkaline environments or where other metals such as iron, copper or their salts are present. Experience has not been good with metals in organic muck in estuarine environments. Concrete deteriorates slowly in contact with chlorides, sulphates and certain magnesium salts. Alternate wetting and drying with seawater is also detrimental to concrete. In general, most culvert materials exposed to seawater require some type of protection to assure adequate service life.

Coal mines and certain other mining operations can produce free acid or acid forming elements which are corrosive to nearly all culvert materials. Vitrified clay, stainless steel and bituminized fiber have been successfully used in severe acid environments as culvert and as lining materials. Ends of bituminized fiber should be protected since limited observations indicate that delamination occurs in direct sunlight.

Alkaline water and soils containing sulphates and carbonates cause rapid deterioration of concrete culverts. This deterioration can be retarded by the use of Type V and other limited calcium aluminate cement or higher cement content concretes.

Protection of metal culverts from corrosion usually consists of bituminous or asbestos bonded bituminous coating or coating and paving. Conclusions regarding the use of bituminous protective coatings are not consistent. Some States have found significant increases in service life while others have concluded that such coatings are not cost effective. Asbestos bonded metal appears to give better resistance to deterioration. Bituminous coatings are not successful in highly hostile environments because of insufficient bond to the metal, holidays, and damage to the coatings in handling and placing. All bituminous coatings are vulnerable to petroleum wastes and spills and to destruction by fire.

Mill-applied thermoplastic coatings on corrugated metal culverts are of more uniform thickness, less subject to damage in handling and installation and have fewer manufacturing flaws than bituminous coats. They are superior to bituminous coatings in abrasion resistance and, although experience is relatively short, it appears that culverts with these coatings will survive for a reasonable period in corrosive environments.

A National Cooperative Highway Research Program (NCHRP) Synthesis Report (1975) will provide guidelines for the selection of durable materials and protective measures for various corrosive environments.

11.0 Safety

The primary responsibility for traffic safety in the hydraulic design of culverts is met by providing structures adequate to avoid hazardous flooding and failure of highways. It is also important that culverts be located so that the structure will present a minimum hazard to traffic.

Culvert ends should be located outside the safe recovery area, where possible, and continued across medians except where safe recovery areas can be provided otherwise. Some culvert ends can be made traffic safe by the use of traversable grates, but only if the grates will not become a hazard by causing the highway to flood. Grate hydraulic capacity and the potential for clogging by debris must be considered before selecting this method for making culvert ends traffic safe (31, 32, 33).

At locations where culvert ends cannot be located outside the safe recovery area and where grates would be impractical or unsafe, guardrail protection should be provided.

Culverts can also be an attractive nuisance and a hazard to children. At locations where long culverts could be a hazard, fencing or grates should be provided to prevent entry.

12.0 Design Documentation

Design data should be assembled in an orderly fashion and retained for future reference. The amount and detail of documentation for each culvert site should be commensurate with the risk and the importance of the structure. Post-construction review of data and documentation may be necessary for the following reasons:

1. The performance of structures over a period of time is very helpful in evaluating design policies and procedures and the validity of design assumptions;
2. In the event of failure, contributing factors can be identified and considered in the design of replacement structures;
3. Source of information when structure is replaced, extended or improved;
4. Source of information for the design of other structures in the vicinity;
5. Source of information in the event of litigation.

12.1 Compilation of Data

Data can be compiled in a variety of ways and should include these items:

1. Copies of all pertinent correspondence;
2. Topography of site;
3. Drainage area map;
4. Stream profile and cross sections;
5. Historical highwater documentation;
6. Information on existing structures in the vicinity;
7. Hydrologic design computations;
8. Hydraulic design calculations and culvert performance curves;
9. Foundation investigation;
10. Structure plans; and
11. Economic analysis of structure selection.

12.2 Retention of Records

Provisions should be made to retain records of culvert designs until the highway is reconstructed or the culverts replaced. Records may be retained in design files or on microfilm and should be readily available when needed for reference or review.

13.0 Hydraulic Related Construction Considerations

Assembly or construction, bedding and backfill are as important to satisfactory culvert service as the hydraulic and structural design. In addition, there are hydraulic related factors that should be considered by construction engineers.

13.1 Verification of Plans

Plans should be checked to verify that site conditions have not changed from the time of location surveys to construction. Changes in culvert design required because of differences between location and construction surveys should be made in consultation with the design engineers. Some changes could significantly affect either the hydrology at the site or the hydraulic performance of the culvert designed for the site.

Changes in land use in the watershed such as clear cutting of forests or urbanization, can change the hydrology at the site and debris considerations used in the design. Development near the site could change damage risk considerations for the design.

Changes in stream alignment and profile can result in different flow conditions than those for which the design was prepared. Changes in headwater elevation-capacity relations and outlet velocity may require consideration of changes in culvert type, size, or shape and of the need for protection against scour at the outlet.

13.2 Temporary Erosion Control

During construction, care should be taken to minimize the erosion at culvert inlets and outlets, and siltation within the culvert. Temporary siltation pools and check dams should be considered at the culvert inlets or outlets. Temporary erosion control methods are discussed in Volume III of these guidelines entitled, "Guidelines for Erosion and Sediment Control in Highway Construction" and Reference 34.

13.3 Construction and Documentation

Records should be kept of the construction of each culvert installation. The final location and slope of the culvert should be recorded on the "as-built" plans. This information is useful for evaluating overall performance of the installation.

Construction personnel are encouraged to inform the designer of any difficulties which are encountered and to make suggestions to improve future designs.

14.0 Hydraulic Related Maintenance Considerations

Culvert designs should be prepared recognizing that all structures require periodic maintenance inspection and repair. Where possible, some means should be providef for personnel and equipment access to the structures to facilitate this activity. Culverts must be kept in good repair and reasonably clean at all times if they are to function as intended (35).

Maintenance personnel should advise design engineers of culvert locations which require considerable annual maintenance. It may be that the maintenance is not necessary to the integrity of the structure or a problem may exist which should be corrected by a design modification.

14.1 Maintenance Inspections

Culvert failures can be both disastrous and expensive. A comprehensive program for maintaining culverts in good repair and operating condition will reduce the probability of failures and prove to be cost effective. The program should include periodic inspections with supplemental inspections following flood events. Conditions which appear to require remedial construction should be referred to the hydraulic engineer for the design of corrective measures.

14.2 Flood Records

An inspection of culverts should be made during and after major floods to observe the culvert operation and record high water marks. Conditions which require corrective maintenance should be noted including debris accumulations, silting, erosion, piping, scour, and structural damage. Performance information that reflects a need for design or construction changes or unusally large flood peaks should be submitted to the hydraulic design section for review.

14.3 Reconstruction and Repair

Maintenance inspections will often reveal the need for major repairs, culvert appurtenant structures such as energy dissipators, extensive scour protection, and sometimes reconstruction. The repair of various types of culvert distress and failures is discussed in Reference 35.

Extensive and costly repair, construction and reconstruction should be coordinated with the hydraulic design section. This is advisable particularly when conditions have changed from those which prevailed at the time the existing culvert was designed. Urbanization or other changes in the watershed, channelization of the stream, flood control storage, or any of numerous other changes which affect hydrology may require reconsideration of the culvert type and size, allowable headwater elevations and acceptable risk at the culvert site. Physical changes at the site and in the stream, such as aggradation or degradation, may make it advisable to reconstruct rather than undertake major repairs or modifications.

Most culvert replacements by maintenance forces should be coordinated with design for possible revisions in structure geometry and size. Culvert failures may occur because of unusual floods, inadequate size or for reasons not related to hydraulic adequacy such as piping, scour, corrosion, abrasion, inadequate foundation and buoyance. For this reason, overflow over the roadway or culvert failure may require replacement with a larger culvert, a change in inlet geometry of the existing culvert, replacement with an equivalent culvert and precautions against failure from other causes, or an identical replacement culvert may be indicated.

15.0 References

(1) Herr, L. A., *Hydraulic Charts for the Selection of Highway Culverts*, Hydraulic Engineering Circular No. 5, Federal Highway Administration, U.S. Government Printing Office, Washington, D.C., 1965, 54 p.

(2) Herr, L. A., Bossy, H. G., *Capacity Charts for the Hydraulic Design of Highway Culverts*, Hydraulic Engineering Circular No. 10, Federal Highway Administration, Office of Engineering, Washington, D.C., 1965 90 p.

(3) Harrison, L. J., Morris, J. L., Normann, J. M., and Johnson, F. L., *Hydraulic Design of Improved Inlets for Culverts*, Hydraulic Engineering Circular No. 13, Federal Highway Administration, U.S. Government Printing Office, Washington, D.C., 1972, 150 p.

(4) Normann, J. M., Bossy, H. G., *Hydraulic Flow Resistance Factors for Corrugated Metal Conduits*, Federal Highway Administration, U.S. Government Printing Office, 1970, 48 p.

(5) Simons, D. B., Stevens, M. A., Watts, F. J., *Flood Protection at Culvert Outlets*, Report No. CER-69-70DBS-MAS-FJW4, Colorado State University, Ft. Collins, Colorado and Wyoming State Highway Department, Cheyenne, Wyoming, 1970, 218 p.

(6) Highway Research Board, *Design of Culverts, Energy Dissipators and Filter Systems*, Highway Research Record Number 373, 8 Reports, Highway Research Board, Washington, D.C., 1971, pp. 1-73.

(7) Chang, F. M., and Karim, M., *Erosion Protection for the Outlet of Small and Medium Culverts*, South Dakota State University, Brookings, South Dakota, and South Dakota Department of Highways, Pierre, South Dakota, 1970, 52 p.

(8) American Society of Civil Engineers, *Symposium of Stilling Basins and Energy Dissipators*, Journal of the Hydraulics Division, Proceedings Symposium Series No. 5, 1961, 8 Papers, with Discussions.

(9) Fletcher, B. P., Grace, Jr., J. L., *Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets*, Miscellaneous Paper H-72-5, U.S. Army Waterways Experiment Station, Vicksburg, Mississippi, 1972, 42 p.

(10) Bohan, J. P., *Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets*, Miscellaneous Paper H-70-2, U.S. Army Waterways Experiment Station, Vicksburg, Mississippi, 1970, 54 p.

(11) Schilling, M. C., *Culvert Outlet Protection Design: Computer Program Documentation.* Wyoming State Department of Highways for the Federal Highway Administration. Available from the National Technical Information Service, Springfield, Virginia 22151, 1974, 243 p.

(12) Edgerton, Roy C., *Culvert Inlet Failures—A Case History,* Highway Research Board Bulletin 286, Drainage Structures, Design and Performance, 1960, Publication 856, Washington, D.C., 1961, 31 p.

(13) Bureau of Reclamation, U.S. Department of Interior, *Design of Small Dams,* U.S. Government Printing Office, Washington, D.C., 1973, 816 p.

(14) Chow, V. T., *Open Channel Hydraulics,* McGraw-Hill, 1970, pp. 512-516.

(15) Behlke, C. E., Pritchett, H. D., *The Design of Supercritical Flow Channel Junctions,* Highway Research Record Number 123, Highway Research Board, Washington, D.C., 1966, pp. 17-35.

(16) Rouse, H., *Engineering Hydraulics,* John Wiley & Sons, Inc., 1949, pp. 29.

(17) Kay, A. R., Lewis, R. B., *Passage of Anadromous Fish Thru Highway Drainage Structures,* Research Report 629110, State of California, Department of Public Works, Division of Highways, 1970, 15 p.

(18) McClellan, T. J., *Fish Passage Through Highway Culverts,* U.S. Department of Transportation, Federal Highway Administration, Region 10, Portland, Oregon, 1970.

(19) Clay, C. H., *Design of Fishways and Other Fish Facilities,* Queen's Printer, Ottawa, Canada, 1961.

(20) Watts, F. J., *Design of Culvert Fishways,* Water Resources Research Institute, University of Idaho, Moscow, Idaho, 1974, 62 p.

(21) Normann, J. M., *Hydraulic Aspects of Fish-Ladder Baffles in Box Culverts,* U.S. Department of Transportation, Federal Highway Administration, Office of Engineering, Washington, D.C., 1974, 29 p.

(22) State of California, *California Culvert Practice,* 2nd Edition, State of California, Department of Public Works, Division of Highways, 119 p.

(23) Reihsen, G., Harrison, L. J., *Debris Control Structures,* Hydraulic Engineering Circular No. 9, U.S. Department of Transportation Federal Highway Administration, U.S. Government Printing Office, Washington, D.C., 1971, 38 p.

80 (24) Haviland, J. E., Bellair, P. J., and Morrell, V. D., *Durability of Corrugated Metal Culverts,* Highway Research Report Number 242, Highway Research Board, Washington, D.C., 1968, pp. 41-66.

(25) Beaton, J. L., Stratfull, R. F., *Field Test for Estimating Service Life of Corrugated Metal Pipe,* Highway Research Board Proceedings, Volume 41, 1962, pp. 255-272.

(26) Berg, V. E., *Culvert Performance Evaluation,* Washington State Highway Commission, Department of Highways, 1965.

(27) Nordlin, E. F., Stratfull, R. F., *A Preliminary Study of Aluminum as a Culvert Material,* Highway Research Record No. 95, Highway Research Board, Washington, D.C., 1965, pp. 1-70.

(28) Lowe, T. A., Koeph, A. H., *Corrosion Performance of Aluminum Culvert,* Highway Research Record No. 56, 1964, pp. 98-115.

(29) Peterson, D. E., *Evaluation of Aluminum Alloy for Use in Utah's Highways,* Utah State Department of Highways, 1973, 43 p.

(30) Braley, S. A., *Acid Drainage from Coal Mines,* Trans. AIME, (Mining Branch), Volume 190, 1951, pp. 703-707.

(31) Highway Research Board, *Traffic-Safe and Hydraulically Efficient Drainage Structures,* NCHRP Synthesis Report No. 3, Highway Research Board, Washington, D.C., 1969, 38 p.

(32) Federal Highway Administration, *Handbook of Highway Safety Design and Operating Practices,* U.S. Department of Transportation Federal Highway Administration, 1973, 112 p.

(33) AASHTO, *Highway Design and Operational Practices Related to Highway Safety,* 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 1974.

(34) Dunkley, C. L., *Suggestions for Temporary Erosion and Siltation Control Measures,* U.S. Department of Transportation, Federal Highway Administration, Office of Engineering, Washington, D.C., 1973, 40 p.

(35) AASHTO, *An Informational Guide for Physical Maintenance,* American Association of State Highway and Transportation Officials, Washington, D.C., 1971, 52 p.

Photogrammetric Engineering

Volume XXVII SEPTEMBER, 1961 *Number 4*

Contents

(Continued on page 501)

Second-class postage paid at Menasha, Wisconsin and at additional mailing offices.

Published five times a year, during the months of March, April, June, September and December, by the American Society of Photogrammetry, Publication Office, Curtis Reed Plaza, Menasha, Wis., and at Society's Address—1515 Massachusetts Ave., N.W., Washington 5, D. C. The Editor's address is 3523 Raymond St., Chevy Chase 15, Md.

Subscription Rates—No charge to Society members in good standing; $6 annually for subscribers in the United States and its possessions; and $6.50 annually, elsewhere. Society pays the postage. Price of individual back numbers—in U. S. and its possessions, $1.75 per copy for single purchase of 3 or less and $1.65 per copy for 4 or more; elsewhere $1.85 per copy for single purchase of 3 or less and $1.75 per copy for 4 or more. Orders for subscriptions or for individual copies should be sent to the Society's address.

Printed in the United States of America

(Continued from page 499)

82

Announcement

PHOTOGRAMMETRIC ENGINEERING

Important Changes Coming

Those who have been confused by the 5 issues-per-year basis will be highly pleased to learn that the Board of Direction has approved changing to 6 issues per year, that is semi-monthly.

The change will not be fully effective until 1963 with the January issue being numbered 1. In 1962 there will be 5 issues as now. The months of issue will be March, May (formerly April), July (formerly June), September and November (formerly December). For advertising, there will be no change in base rates, frequency reduction, agency commission and also the 2% if earned. Beginning with the second issue in 1962, the closing dates will be changed.

Drainage Studies from Aerial Surveys

IRWIN STERNBERG,
District Location Engineer,
Arizona Highway Dept., Tucson, Ariz.

ABSTRACT: *Vertical aerial photographs examined stereoscopically provide a useful three-dimensional medium whereby drainage areas can be successfully determined with sufficient accuracy for the design of culverts for highway drainage.*

Discussed in the paper is the use of large-scale photographs for determining the placement of these culverts and other items concerned with the collection and dispersal of surface water during run-off periods.

Methods, corrections to be applied, and techniques which have been successfully employed, all of which are within the capabilities of the average field engineer with limited photogrammetric training and equipment, are described. Examples are given to show the degree of accuracy which can be expected.

INTRODUCTION

THE purpose of this paper is two-fold. The first is to prescribe a method whereby stereoscopic pairs of aerial photographs can be used by field engineers with limited photogrammetric training, for determining drainage areas with sufficient accuracy and detail for use in estimating culvert sizes. The second is to consider the use of such photographs for the actual positioning of culverts, bridges, dikes, channel changes and similar features of design with an accuracy sufficient for use as a guide in construction plans and estimates.

While the investigation was confined to the southern part of the State of Arizona, the same methods with minor differences should be applicable to other areas. No claims are made for originality in developing the following techniques. No doubt they have been used previously, but a presentation of the developed procedures will no doubt be interesting to many who are involved in locating modern highways and in other undertakings involving the location and design of drainage structures.

Drainage areas are usually determined by utilizing existing maps or by traversing each watershed. Both methods have their shortcomings. Maps can be unreliable or so lacking in detail as to preclude accurate determination of larger areas or any determination at all of the smaller areas. Traversing, either by stadia, plane-table, or transit and tape, is both time and labor consuming, especially in regions of rough terrain or where ground cover interferes.

In the arid Southwest, the extent and character of drainage areas are very important in determining culvert sizes. During the rainy seasons storms are frequent and although of short duration they can be violent in character. This characteristic plus the impervious nature of the soil and the sparseness of vegetation contribute to the rapid run-offs encountered in this section of the country. Washes which are normally dry suddenly become raging torrents, while in the flat desert regions, washes frequently overflow the surrounding areas. It is necessary therefore to provide adequate drainage across highways both to safeguard the highways against excessive erosion and to protect the surrounding land.

In the past much of this run-off was taken care of by constructing dips across the highways. When properly constructed and protected, these dips were both economical and satisfactory. However, with the rapid increase in amount and speed of traffic, and the increased importance of the highway in the economy of the country, the practicality of dips was decreased, and enforced delays to the motorist during flash floods were not only irksome but expensive. And of course on divided highways of the Interstate System and other heavily travelled limited-access facilities, such a treatment would be not only archaic but unworkable.

In Arizona it has been found that the Talbot equation for determining culvert sizes is quite satisfactory. This equation is an empirical formula based on a large number of observations. It works well with a flow velocity of ten feet per second or less and a

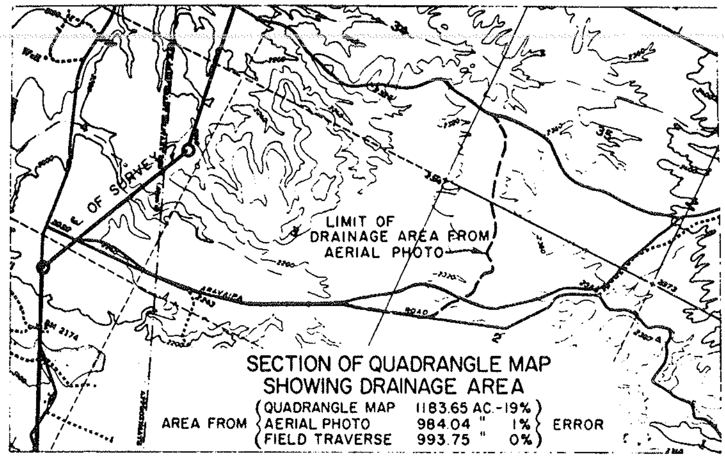


FIG. 1. Sketch illustrating the differences in drainage area extent which may occur between actual photographs and small-scale contour maps, due to excessive contour interval.

maximum rainfall of 4 inches per hour. The general formula is $A = C_l\sqrt[4]{M^3}$ where A is the area of the required waterway in square feet, M is the area drained in acres and C is a coefficient depending on the contour and character of the land drained. This coefficient varies from $C=1$ for steep rocky ground to $C=1/5$ or less for comparatively flat areas. Other formulas for determining waterway areas can of course be used.

The character and slope of the terrain is therefore important. Its extent and whether the run-off is confined, or covers an extensive area, must also be considered. Experience counts much in selection of the run-off coefficient for use in the equation and in selecting the size of the culvert and determining its placement.

Drainage areas to be used in the application of this equation were, and still are in many cases, determined by measurement in the field or from existing maps. In the comparatively unsettled West suitable maps are scarce, are not too accurate, and many are lacking in sufficient detail to be reliable or useful. The recent $7\frac{1}{2}$-minute quadrangle topographic maps published by the United States Geological Survey are very helpful and are quite accurate. Unfortunately they are not now plentiful and in many cases the contour interval is too large to ensure a true determination of the drainage area boundaries.

Figure 1 illustrates how errors could inadvertently occur in outlining an area where the contour interval is too great. Shown is a section taken from a recent $7\frac{1}{2}$-minute quadrangle map with a contour interval of 40 feet. The solid black line shows the boundary of a drainage area as it would be determined from the contour information given. The dashed black line shows where the boundary differs from the solid black line as the result of a stadia traverse run in the field by regular field methods. A low ridge cuts transversely across the area with its crest along the dashed black line. It so happens that the elevation along the ridge is about 2,340 feet, while the elevation of the trough behind it to the east is around 2,330 feet. The contours on the map therefore give no indication that such a ridge exists. The effect of this ridge on the drainage area is clearly evident in the figure.

Determining Drainage Areas

Aerial photography naturally is suggested as a possible solution to the problem which then is resolved into the various possibilities open to the use of this medium. It is desirable to limit its use to such forms as are usable by field survey personnel who have a minimum of photogrammetric training and to such equipment as can be made available to those men.

Where an area has been photographed for reconnaissance survey purposes the aerial photographs thus secured can be used, particularly where the drainage area is of considerable extent. Photographic mosaics can also be used but the three-dimensional effect attained from stereoscopic examination of pairs of aerial photographs is very desirable in tracing the boundaries of watershed areas. The contours of topographic maps compiled photogrammetrically are extremely useful but the maps are generally of such limited extent that they are of little value except for very small areas. For larger areas and when available, as in Arizona, manuscript maps compiled from existing photography at the scale of one-mile-to-one-inch for use in preparing the general county highway maps, are very useful. But here again drainage detail is not sufficient for determining the boundaries of the smaller areas. In many cases therefore it becomes desirable to photograph the region under consideration, especially for drainage area studies. Such photography is inexpensive and its cost can be saved many times over in time and labor. A scale of 2000 feet-per-inch for normal size areas is generally satisfactory although a smaller scale can be used for areas of greater size. Large scale photography might be considered in some instances, and for small areas the large scale in many cases would be desirable. But as the scale is increased the coverage is decreased and the larger areas become unwieldly in size.

The following discussion is based primarily on the use of aerial photographs, as the use of other media, such as maps, requires no special comment. The discussion is also limited to small and medium-size areas which can be plotted on one or two strips of photographs. Larger areas can perhaps best be determined by other methods—such as on existing quadrangle maps or on drainage maps especially prepared in the photogrammetric laboratory.

First the scale of the photography is determined and the centerline of the highway is plotted on the photographs. On photography made prior to the location of the survey center line, this plotting will have to be done by photographic identification. On photography taken without control being premarked on the ground by photographic targets, the scale can be determined with sufficient accuracy from existing features—such as section lines, property lines, existing roads or other configurations where the length is known or can be determined. On photography where photographic targets appear as placed on control points, the scale of course can be easily determined; this will frequently be the case as placement of photographic targets prior to photography is becoming more and more a prevalent practice.

Drainage area boundaries should be plotted stereoscopically using a pocket or mirror stereoscope and a red pencil. It is necessary to trace the boundary on only one overlapping area and preferably the one on which the entire area occurs. On larger areas which extend over adjacent flight strips, care should be taken in transferring the watershed boundaries from the edge of one photograph to another. This should be done preferably by radial plot to minimize errors. Where the elevations within an area vary by less than 200 feet, or where the drainage areas are not of great extent, these areas can be planimetered directly on the photographs and converted to acres or square miles according to the average scale as determined. For larger areas and where differences of elevation are still within the approximate 200 foot range, a scale based on an average elevation of the drainage area to be considered can be used, and the area converted on this basis.

Where there is a difference in elevation of 200 feet or more within the limits of the drainage area, adjustments should be made for image displacement due to relief. If this is not done errors may occur which could materially affect the size of a culvert or waterway. In such cases it is more feasible and convenient to transfer the drainage areas and other pertinent features from the photographs onto tracing paper before the adjustments are made.

In order to determine these adjustments the displacement formula $d_e = rh/H$ is used. In this d_e is the displacement due to relief, r is the distance on the photograph from the principal point to the image of the top of the object, h is the ground elevation of the object and H is the flight height of the photograph relative to the same elevation datum as h. Elevations around the perimeter of each area, as required for making the adjustments, can be determined from the photographs with sufficient accuracy by means of parallax measurements using either a parallax bar or an engineer's scale. Necessary measurements can then be made and the adjusted areas drawn and planimetered on the tracing. If topographic maps of sufficient detail and accuracy of the area are available, the elevations can be taken directly from the contours on these maps with a considerable saving in time. Other means of obtaining the elevations would also be acceptable as elevations to the nearest 50 feet will be accurate enough.

Relief displacement can then be easily determined to a selected datum applying the above formula and the corrections made as in Figure 2. The drainage area thus adjusted can be planimetered as usual, computing its extent to a scale as determined by elevation of the datum plane selected. It appears to be more convenient, although not necessary, to select this datum plane so that it will pass through one of the lower elevations along the center-line of the survey as plotted on each photographic print.

Corrections for tilt are not necessary where

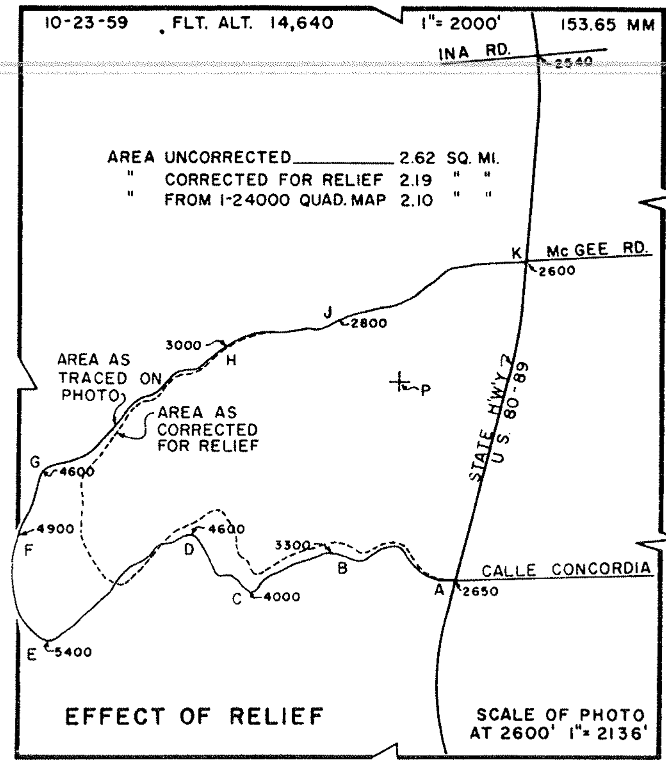


FIG. 2. Sketch illustrating effect of excessive relief on drainage area extent with the result of corrections made by application of the displacement formula.

the vertical photographs do not contain more tilt than is allowed by the usual specifications. Moreover, corrections for tilt require some ground survey data, are quite complicated to make, and are generally beyond the understanding of field engineers to compute. Any errors in drainage area due to tilt in vertical photographs should not exceed the limits of accuracy required. Investigations have shown that error in area in excess of 3 per cent because of tilt is unlikely.

Drainage areas in flat desert or similar regions are more difficult to determine than where topography is rolling or rugged, due partly to the lack of relief and partly to the fact that shifting channels during storm periods will sometimes alter the smaller areas to a considerable extent. This is particularly true where the area under consideration is only a part of a larger major area. Also the enormous amount of sediment carried down these shallow channels during a cloudburst will frequently fill a shallow wash and will cause the stream to cut out another channel with perhaps a cross-over into an adjacent area. Close examination of aerial photographs will make these occurrences evident or their possibilities known much better than can be determined in the field. Abandoned channels will be evident and in many cases future behavior of the stream flow can be predicted.

Other Drainage Data Obtainable from Aerial Photographs

In addition to quantitative data pertaining to drainage area size which is determinable from aerial photographs, other vital quantitative data and qualitative information may be obtained. Not only are all of these data not obtainable from topographic maps, but they are difficult and expensive to ascertain by investigative methods on the ground.

Vertical aerial photography viewed stereoscopically is particularly adaptable to the determination of type and extent of ground cover, and the extent of ponding and water retarding features of each drainage area. These features are vital factors regarding surficial drainage and are essential components of a judicial analysis of a specific drainage problem. They cannot be obtained from topographic maps or easily obtained in the field.

Ground cover, the extent being dependent on the type and intensity, reduces the run-off volume and retards the run-off velocity. When veiwed stereoscopically, aerial photographs make possible the engineer determining the amount and extent of ground cover, the exposure and the ground slope. He can also to some extent determine the character and type of the cover. This knowledge is important in identifying underlying types of soil, and judging internal drainage characteristics of soils. All of these are essential for accurate determination of sizes and shapes of structure openings.

Ponding or retarding of water above drainage structure openings is another significant factor to consider in the structure design. Some drainage channels, whether wide or narrow, are deep with steep and fast drainage characteristics. These require structures of considerable head room. Other areas are extensively wide and flat in character and may be broken by a large lake. Such areas tend to collect large amounts of surficial run-off and to act as dispersal areas up above the site in the drainage channel where the drainage structure must be placed. Where this happens the structure opening may be reduced somewhat from the larger size indicated by the factor obtained from the drainage area only, i.e. without reduction. When aerial photographs are used these drainage features are easily and accurately determined and result in a better and more economically drained highway area.

Examples of Drainage Area Determination

Figure 2 illustrates the amount of drainage boundary displacement that can occur due to extreme relief. At this point the existing highway is adjacent to the Santa Catalina Mountains; the elevation difference between the low and the high points of the area to be measured amounts to 2,750 feet in a distance of approximately 0.8 mile. Slopes are very rugged and the determination of the area by ordinary field methods would be extremely difficult. The photographs were taken from a flight height of approximately 12,000 feet with a six inch focal-length lens. Scale was estimated from known distances along the highway.

This photograph was chosen because of the extreme conditions encountered and because there was available a recent 7½-minute quadrangle map which could be used for comparison. At the time the photography was obtained there was no intention of use for other than pictorial purposes. It should be noted that the drainage area as sketched was not completely covered by the photograph, and it was necessary to estimate the upper limits of the boundary. The boundary was determined stereoscopically from this and an adjacent photograph.

In spite of the described limitations, there was only 0.09 of a square mile difference between the area computed from the quadrangle map and from the photography after correction for relief displacement as shown. This is an error of only 4.3 per cent, assuming the area as computed from the quadrangle map to be correct. Without relief displacement correction, the error would have been in the neighborhood of 25 per cent. This is of course an extreme condition and much better accuracy can be expected in the majority of cases.

On one recent project—6.4 miles long—50 separate drainage areas were considered; these had an aggregate area of 11,567 acres. Although the relief was moderate the country was difficult to traverse; to measure the areas by transit and stadia would have required the time of three men for at least two weeks. The areas were plotted on existing photographs of approximately 2,000 feet per inch scale on which targets marking control point positions appeared; these were plotted and computed by one man in about two days with greater detail than would have resulted from regular field methods.

Relief differential was moderate and it was not necessary to correct for displacement of watershed boundaries due to this factor. In three cases the larger drainage areas encountered extended beyond the limits of the photographs and it was necessary to determine these on existing topographic maps. Comparisons with existing 7½-minute quadrangle maps in this area showed a difference of under 0.5 per cent. A stadia survey over one area of 984 acres (that shown in Figure 1) took two men one full day to complete and showed a difference of under 1 per cent between the field survey and the area as taken from the photograph. The savings here in both time and labor are obvious.

Positioning of Culverts and Related Drainage Features

After the drainage areas have been determined, it becomes necessary to select the proper size of structure and to locate it on the ground in the most desirable and economical position. Generally this is done by field examination of each drainage channel along the center line of the survey, measuring or

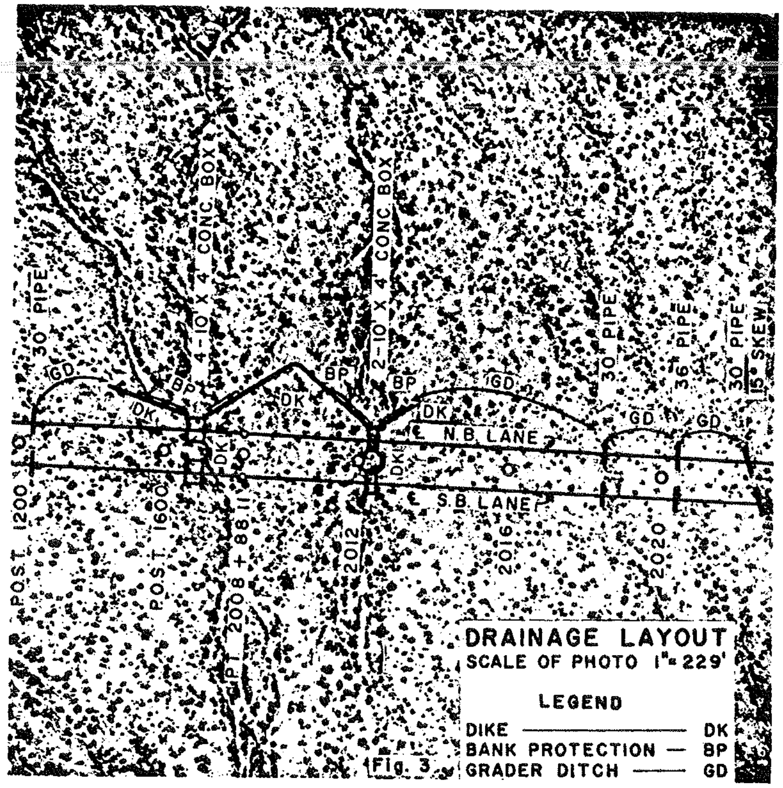


FIG. 3. Drainage layout detailed on an aerial photograph showing method of placement of structures, dikes and other drainage features.

estimating the stationing, estimating the angle of skew and making sufficient notes so the structure can be designed and incorporated in the plans.

There is nothing wrong with this method. It has worked for years, but it is time-consuming and frequently it is difficult to determine exactly the most desirable place in which to construct the culvert, especially where vegetation along the stream is thick or where the channel is complexly divided or braided.

Through use of aerial photographs of suitable scale, culverts can be positioned accurately and in many cases better than by examination on the ground. Photographs of a large scale are desirable. Those at 250 feet per inch have proved quite satisfactory. This scale is large enough to supply needed detail while coverage is sufficient to follow the course of a stream far enough to place the culvert in its most efficient position, and to determine the need for dikes, channel changes or other items to assure the control of the stream flow.

If cross-sectioning is to be done photogrammetrically, vertical photographs secured for this purpose will be ideal. These can also be used for the preparation of large-scale topographic maps with contours at a sufficiently small interval for bridge sites and interchanges. If such photography is not to be secured, it will probably be necessary to fly the area specifically for the drainage study. In any event the photographs should be taken after the center line has been run and after the area has been targeted so that the center-line can be accurately positioned and can be stationed on the photographs, and also so that the exact scale of the photography can be determined. Narrow Chart-Pak stripping is ideal for delineating the center-line so it will be seen on the photographs. Besides being clean to handle it can be applied more rapidly than a wax, china marking-type pencil.

Culverts can now be spotted with considerable accuracy, skew determined and necessary dikes, channel changes and other detail spotted. Bridge sites can be studied and tentative bridge positions determined. Later using the same photography and in preparation for the bridge design, large-scale topographic maps can be made of the bridge sites.

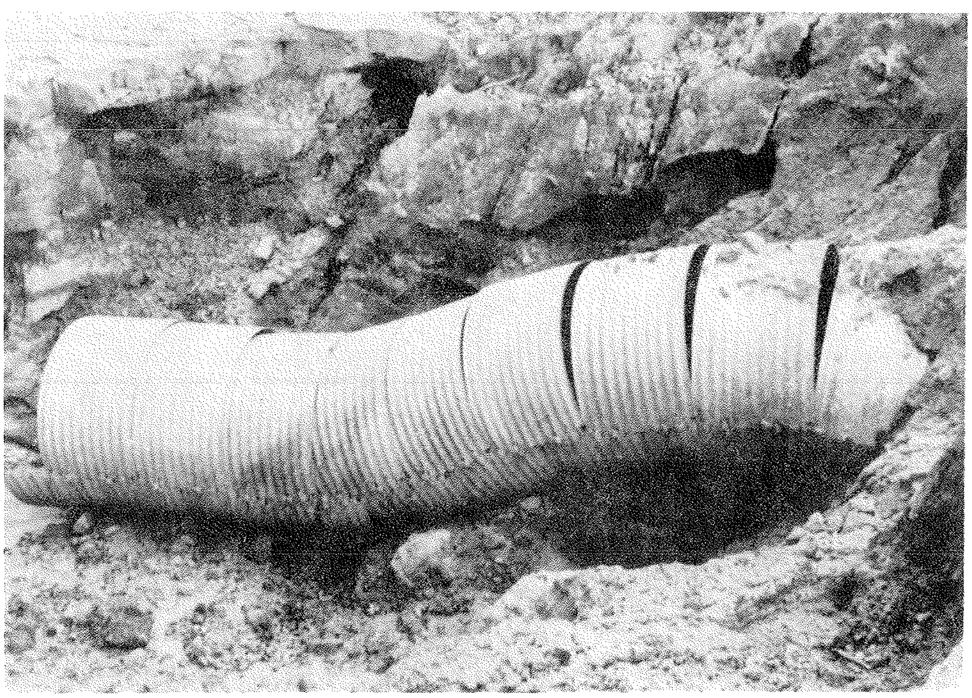
In many cases areas subject to flooding can be noted on the photographs, and an engineer practiced in photographic interpretation can spot other potential areas where serious trouble might be encountered during peak run-off periods.

It should not be inferred that all field work can be eliminated by use of the described methods. While it will be desirable to examine many culvert sites on the ground before construction plans are completed, the field trips can be planned at more convenient times and they will progress much more rapidly than would ordinarily be the case. In some cases where ground cover is extremely thick the method may not be practical or might require more detailed field checking.

After all drainage studies have been made it would be desirable to ink the placements permanently on a set of the photographic prints. The photographs will not only be valuable to design engineers while preparing the construction plans but also of value to construction engineers and engineering crews when the highway is being built. Figure 3 shows one print prepared in this manner. Tabs were used on the print for stationing and other information in the interest of clarity on the halftone reproduction. Red ink or tempera normally used would make this refinement unnecessary.

In this instance cost and time are not as critical as in drainage area determinations. Drainage studies are generally made by the location engineer during survey progress and while the crew is otherwise engaged. There is a saving in cost and time, however, and better, more efficient, and more workable drainage systems will result from this method.

90



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Hydraulic Charts for the Selection of Highway Culverts

Hydraulic Engineering Circular No. 5 December 1965*

Prepared by the Hydraulics Branch, Bridge Division, Office of Engineering, Federal Highway Administration, Washington, D.C. 20590

CONTENTS

*Reprinted April 1977

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HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS

Prepared by Lester A. Herr
Chief, Hydraulics Branch, Bridge Division

In Collaboration with Herbert G. Bossy
Highway Research Engineer, Hydraulic Research Division

Introduction

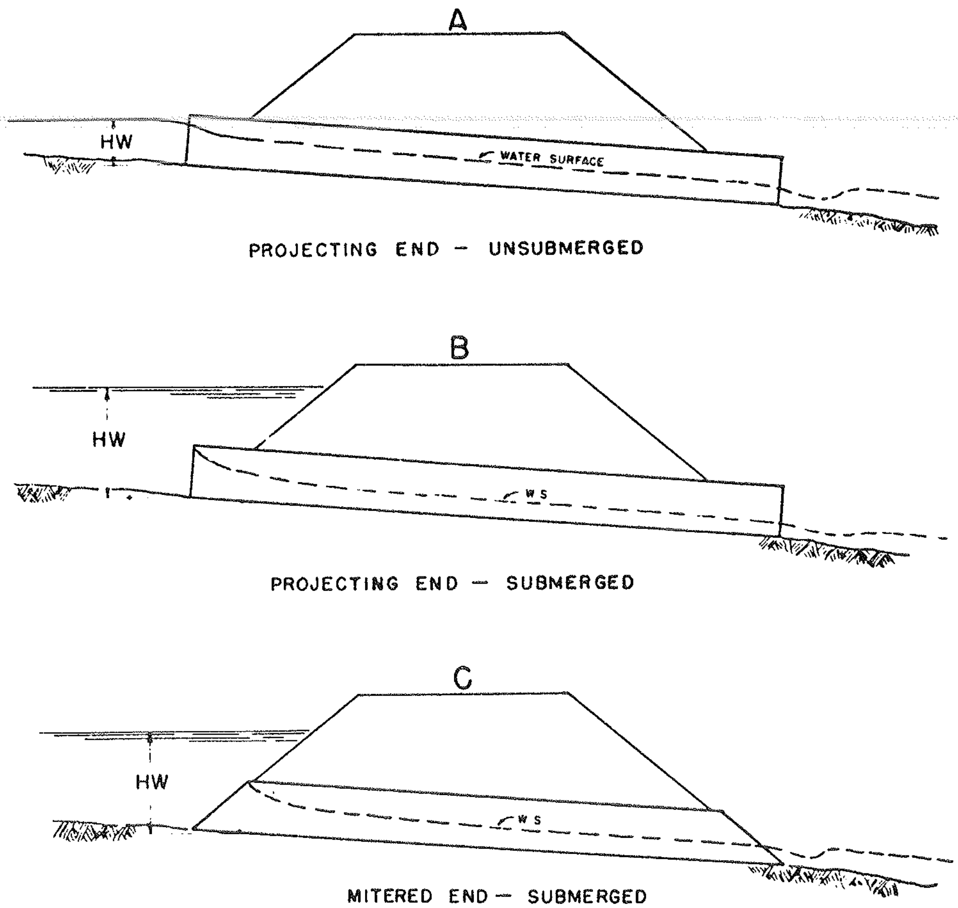
Designing highway culverts involves many factors including estimating flood peaks, hydraulic performance, structural adequacy, and overall construction and maintenance costs. This circular contains a brief discussion of the hydraulics of conventional culverts and charts for selecting a culvert size for a given set of conditions. Instructions for using the charts are provided. No attempt is made to cover all phases of culvert design. Subsequent circulars will cover culverts with modified inlets and outlets designed to increase performance or to apply to a particular location. Some approximations are made in the hydraulic design procedure for simplicity. These approximations are discussed at appropriate points throughout the circular.

For this discussion, conventional culverts include those commonly installed, such as circular, arch and oval pipes, both metal and concrete, and concrete box culverts. All such conventional culverts have a uniform barrel cross section throughout. The culvert inlet may consist of the culvert barrel projected from the roadway fill or mitered to the embankment slope. Sometimes inlets have headwalls, wingwalls and apron slabs, or standard end sections of concrete or metal. The more common types of conventional culverts are considered in this circular.

Culvert Hydraulics

Laboratory tests and field observations show two major types of culvert flow: (1) flow with inlet control and (2) flow with outlet control. For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area of the culvert barrel, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the elevation of the tailwater in the outlet channel and the slope, roughness and length of the culvert barrel.

It is possible by involved hydraulic computations to determine the probable type of flow under which a culvert will operate for a



INLET CONTROL

Figure I

given set of conditions. The need for making these computations may be avoided, however, by computing headwater depths from the charts in this circular for both inlet control and outlet control and then using the higher value to indicate the type of control and to determine the headwater depth. This method of determining the type of control is accurate except for a few cases where the headwater is approximately the same for both types of control.

Both inlet control and outlet control types of flow are discussed briefly in the following paragraphs and procedures for the use of the charts are given.

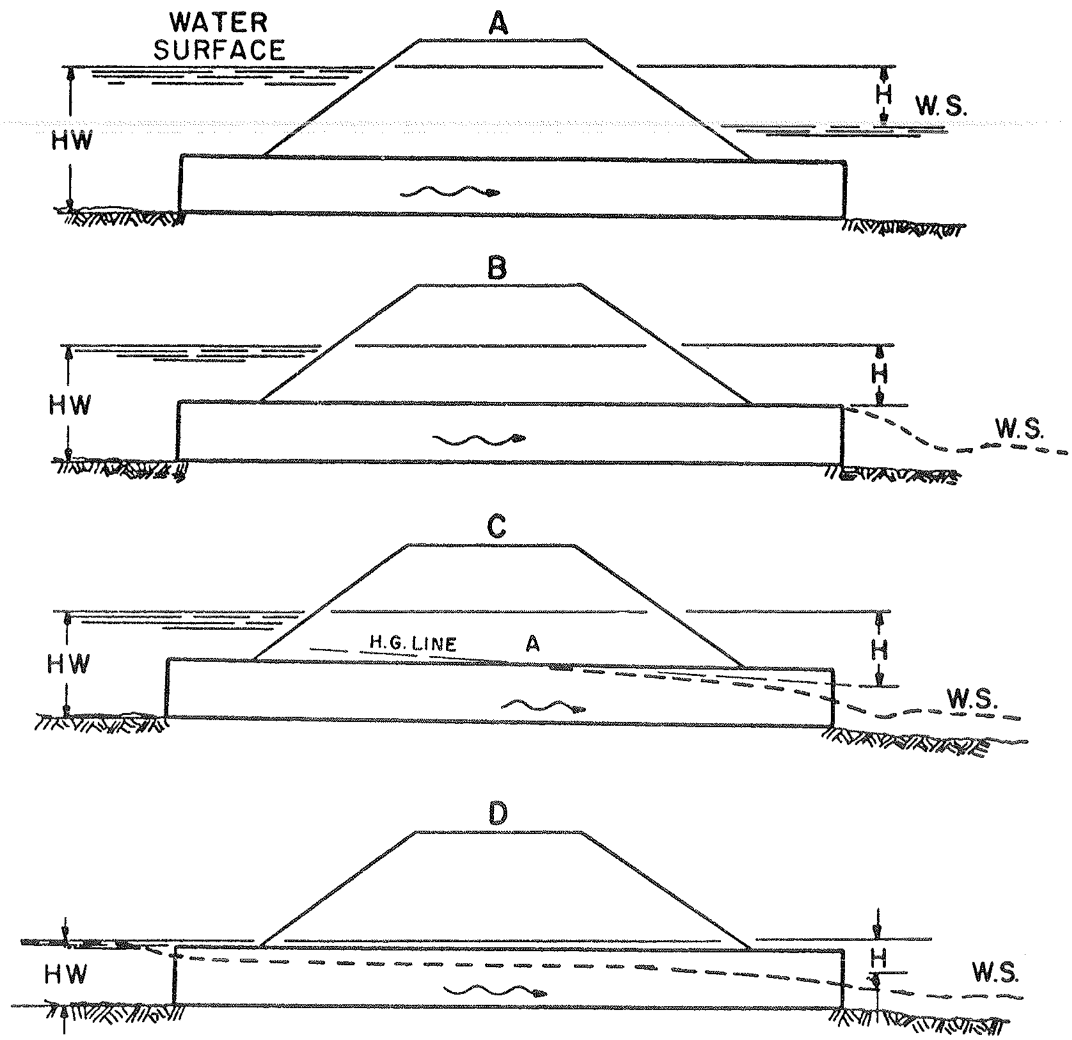
Culverts Flowing With Inlet Control

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater (HW) and the entrance geometry, including the barrel shape and cross-sectional area, and the type of inlet edge. Sketches of inlet-control flow for both unsubmerged and submerged projecting entrances are shown in figures 1A and 1B. Figure 1C shows a mitered entrance flowing under a submerged condition with inlet control.

In inlet control the roughness and length of the culvert barrel and outlet conditions (including depth of tailwater) are not factors in determining culvert capacity. An increase in barrel slope reduces headwater to a small degree and any correction for slope can be neglected for conventional or commonly used culverts flowing with inlet control.

In all culvert design, headwater or depth of ponding at the entrance to a culvert is an important factor in culvert capacity. The headwater depth (or headwater HW) is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth + velocity head). Because of the low velocities in most entrance pools and the difficulty in determining the velocity head for all flows, the water surface and the energy line at the entrance are assumed to be coincident, thus the headwater depths given by the inlet control charts in this circular can be higher than will occur in some installations. For the purposes of measuring headwater, the culvert invert at the entrance is the low point in the culvert opening at the beginning of the full cross-section of the culvert barrel.

Headwater-discharge relationships for the various types of circular and pipe-arch culverts flowing with inlet control are based on laboratory research with models and verified in some instances by prototype tests. This research is reported in National Bureau of Standards Report No. 4444 entitled "Hydraulic Characteristics of Commonly Used Pipe Entrances", by John L. French and "Hydraulics of Conventional



OUTLET CONTROL

Figure 2

95

Highway Culverts", by H. G. Bossy[1/]. Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the U. S. Geological Survey.

These research data were analyzed and nomographs for determining culvert capacity for inlet control were developed by the Division of Hydraulic Research, Bureau of Public Roads. These nomographs, Charts 1 through 6, give headwater-discharge relationships for most conventional culverts flowing with inlet control through a range of headwater depths and discharges. Chart No. 7, discussed on p. 5-13, is included in this revised edition to stress the importance of improving the inlets of culverts flowing with inlet control.

Culverts Flowing With Outlet Control

Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it, (see fig. 2). If the entire cross section of the barrel is filled with water for the total length of the barrel, the culvert is said to be in full flow or flowing full, figures 2A and 2B. Two other common types of outlet-control flow are shown in figures 2C and 2D. The procedures given in this circular provide methods for the accurate determination of headwater depth for the flow conditions shown in figures 2A, 2B and 2C. The method given for the part full flow condition, fig. 2D, gives a solution for headwater depth that decreases in accuracy as the headwater decreases.

The head H (fig. 2A) or energy required to pass a given quantity of water through a culvert flowing in outlet control with the barrel flowing full throughout its length is made up of three major parts. These three parts are usually expressed in feet of water and include a velocity head H_v, an entrance loss H_e, and a friction loss H_f. This energy is obtained from ponding of water at the entrance and expressed in equation form

$$H = H_v + H_e + H_f \qquad (1)$$

The velocity head H_v equals $\frac{V^2}{2g}$, where V is the mean or average velocity in the culvert barrel. (The mean velocity is the discharge Q, in cfs, divided by the cross-sectional area A, in sq. ft., of the barrel.)

The entrance loss H_e depends upon the geometry of the inlet edge. This loss is expressed as a coefficient k_e times the barrel velocity head or $H_e = k_e \frac{V^2}{2g}$. The entrance loss coefficients k_e for various types of entrances when the flow is in outlet control are given in Appendix B, Table 1, (p. 5-49).

1/ Presented at the Tenth National Conference, Hydraulics Division, A.S.C.E., August 1961.

The friction loss H_f is the energy required to overcome the roughness of the culvert barrel. H_f can be expressed in several ways. Since most highway engineers are familiar with Manning's n the following expression is used:

$$H_f = \left[\frac{29n^2 L}{R^{1.33}}\right] \frac{V^2}{2g}$$

where

n = Manning's friction factor (see nomographs and page 5-30 for values)
L = length of culvert barrel (ft.)
V = mean velocity of flow in culvert barrel (ft./sec.)
g = acceleration of gravity, 32.2 (ft./sec.^{2})
R = hydraulic radius or $\frac{A}{WP}$ (ft.)

where

A = area of flow for full cross-section (sq. ft.)
WP = wetted perimeter (ft.)

Substituting in equation 1 and simplifying, we get for <u>full flow</u>

$$H = \left[1 + k_e + \frac{29n^2 L}{R^{1.33}}\right] \frac{V^2}{2g} \qquad (2)$$

97

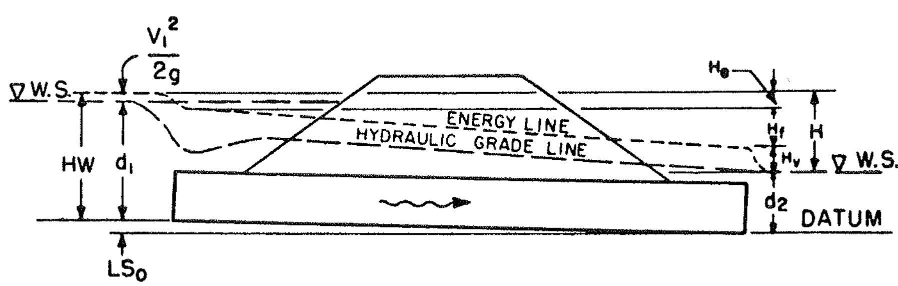


Figure 3

Figure 3 shows the terms of equation 2, the energy line, the hydraulic grade line and the headwater depth, HW. The energy line represents the total energy at any point along the culvert barrel. The hydraulic grade line, sometimes called the pressure line, is defined by the elevations to which water would rise in small vertical pipes attached to the culvert wall along its length. The energy line and the pressure line are parallel over the length of the barrel except in the immediate vicinity of the inlet where the flow contracts and re-expands. The difference in elevation between these two lines is the velocity head, $\frac{V^2}{2g}$.

The expression for H is derived by equating the total energy upstream from the culvert entrance to the energy just inside the culvert outlet with consideration of all the major losses in energy. By referring to figure 3 and using the culvert invert at the outlet as a datum, we get:

$$d_1 + \frac{V_1^{\,2}}{2g} + LS_o = d_2 + H_v + H_e + H_f$$

where

d_1 and d_2 = depths of flow as shown in fig. 3

$\frac{V_1^{\,2}}{2g}$ = velocity head in entrance pool

LS_o = length of culvert times barrel slope

then

$$d_1 + \frac{V_1^{\,2}}{2g} + LS_o - d_2 = H_v + H_e + H_f$$

and

$$H = d_1 + \frac{V_1^{\,2}}{2g} + LS_o - d_2 = H_v + H_e + H_f$$

From the development of this energy equation and figure 3, head H is the difference between the elevations of the hydraulic grade line at the outlet and the energy line at the inlet. Since the velocity head in the entrance pool is usually small under ponded conditions, the water surface or headwater pool elevation can be assumed to equal the elevation of the energy line. Thus headwater elevations and headwater depths, as computed by the methods given in this circular, for outlet control, can be higher than might occur in some installations. Headwater depth is the vertical distance from the culvert invert at the entrance to the water surface, assuming the water surface (hydraulic grade line) and the energy line to be coincident, $d_1 + \frac{V_1^{\,2}}{2g}$ in figure 3.

Equation 2 can be solved for H readily by the use of the full-flow nomographs, Charts 8 through 14. Each nomograph is drawn for a particular barrel shape and material and a single value of n as noted on the respective charts. These nomographs can be used for other values of n by modifying the culvert length as directed in the instructions (p. 5-29) for the use of the full-flow nomographs.

In culvert design the depth of headwater HW or the elevation of the ponded water surface is usually desired. Finding the value of H from the nomographs or by equation 2 is only part of the solution for this headwater depth or elevation. In the case of figure 2A or figure 3, where the outlet is totally submerged, the headwater pool elevation (assumed to be the same elevation as the energy line) is found by adding H to the elevation of the tailwater. The headwater depth is the difference in elevations of the pool surface and the culvert invert at the entrance.

When the tailwater is below the crown of the culvert, the submerged condition discussed above no longer exists and the determination of headwater is somewhat more difficult. In discussing outlet-control flow for this condition, tailwater will be assumed to be so low that it has no effect on the culvert flow. (The effect of tailwater will be discussed later.) The common types of flow for the low tailwater condition are shown in figures 2B, 2C and 2D. Each of these flow conditions are dependent on the amount of discharge and the shape of the culvert cross section. Each condition will be discussed separately.

Full flow at the outlet, figure 2B, will occur only with the higher rates of discharge. Charts 15 through 20 are provided to aid in determining this full flow condition. The curves shown on these charts give the depth of flow at the outlet for a given discharge when a culvert is flowing with outlet control. This depth is called critical depth d_c. When the discharge is sufficient to give a critical depth equal to the crown of the culvert barrel, full flow exists at the outlet as in figure 2B. The hydraulic grade line will pass through the crown of the culvert at the outlet for all discharges greater than the discharge causing critical depth to reach the crown of the culvert. Head H can be measured from the crown of the culvert in computing the water surface elevation of the headwater pool.

When critical depth falls below the crown of the culvert at the outlet, the water surface drops as shown in either figures 2C or 2D, depending again on the discharge. To accurately determine headwater for these conditions, computations for locating a backwater curve are usually required. These backwater computations are tedious and time consuming and they should be avoided if possible. Fortunately, headwater for the flow condition shown in figure 2C can be solved by using the nomographs and the instructions given in this circular.

For the condition shown in figure 2C, the culvert must flow full for part of its length. The hydraulic grade line for the portion of the length in full flow will pass through a point where the water breaks with the top of the culvert as represented by point A in figure 2C. Backwater computations show that the hydraulic grade line if extended as a straight line will cut the plane of the outlet cross section at a point above critical depth (water surface). This point is at a height approximately equal to one half the distance between critical depth and the crown of the culvert. The elevation of this point can be used as an equivalent hydraulic grade line and H, as determined by equation 2 or the nomographs, can be added to this elevation to find the water surface elevation of the headwater pool.

The full flow condition for part of the barrel length, figure 2C, will exist when the headwater depth HW, as computed from the above headwater pool elevation, is equal to or greater than the quantity

$$D + (1 + k_e) \frac{V^2}{2g}$$

where V is the mean velocity for the full cross section of the barrel; k_e, the entrance loss coefficient; and D, the inside height of the culvert. If the headwater is less than the above value, a free water surface, figure 2D, will extend through the culvert barrel.

The part full flow condition of figure 2D must be solved by a backwater computation if accurate headwater depths are desired. Details for making this computation are not given in this circular. Instead the solution used is the same as that given for the flow condition of figure 2C, with the reservation that headwater depths become less accurate as the discharge for a particular culvert decreases. Generally, for design purposes, this method is satisfactory for headwater depths above 0.75D, where D is the height of the culvert barrel. Culvert capacity charts found in Hydraulic Engineering Circular No. 10 give a more accurate and easy solution for this free surface flow condition.

Headwater depth HW can be expressed by a common equation for all outlet-control conditions, including all depths of tailwater. This is accomplished by designating the vertical dimension from the culvert invert at the outlet to the elevation from which H is measured as h_o. The headwater depth HW equation is

$$HW = H + h_o - LS_o \qquad (3)$$

All the terms in this equation are in feet. H is computed by equation 2 or found from the full-flow nomographs. L is the length of culvert in feet and S_o the barrel slope in ft. per ft. The distance h_o is discussed in the following paragraphs for the various conditions of outlet-control flow. Headwater HW is the distance in feet from the invert of the culvert at the inlet to the water surface of the headwater pool.

When the elevation of the water surface in the outlet channel is equal to or above the elevation of the top of the culvert opening at the outlet, figure 2A, h_o is equal to the tailwater depth. Tailwater depth TW is the distance in feet from the culvert invert at the outlet to the water surface in the outlet channel. The relationship of HW to the other terms in equation 3 is illustrated in figure 4.

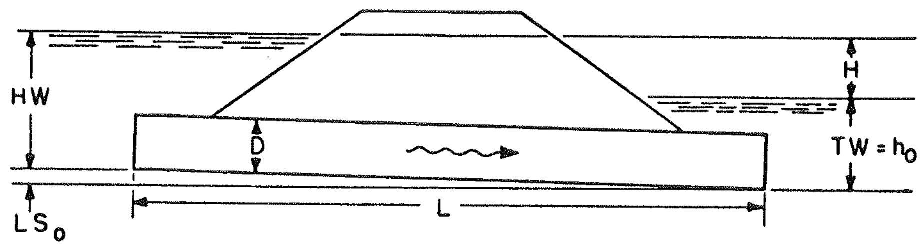


Figure 4

If the tailwater elevation is below the top of the culvert opening at the outlet, figure 2B, 2C and 2D, h_o is more difficult to determine. The discharge, size and shape of culvert, and the TW must be considered. In these cases, h_o is the greater of two values (1) TW depth as defined above or (2) $\frac{d_c + D}{2}$. The latter dimension is the distance to the equivalent hydraulic grade line discussed previously. In this fraction d_c is the critical depth, as read from Charts 15 through 20 and D is the culvert height. The value of d_c can never exceed D, making the upper limit of this fraction equal to D. Where TW is the greater of these two values, critical depth is submerged sufficiently to make TW effective in increasing the headwater. The sketch in figure 5 shows the terms of equation 3 for this low tailwater condition. Figure 5 is drawn similar to figure 2C, but a change in discharge can change the water surface profile to that of figure 2B or 2D.

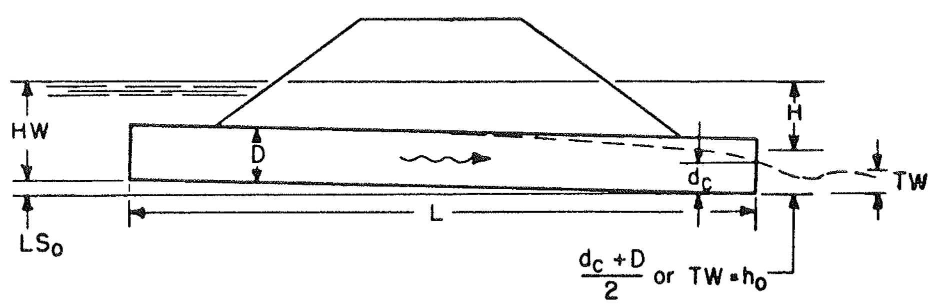


Figure 5

Computing Depth of Tailwater

In culverts flowing with outlet control, tailwater can be an important factor in computing both the headwater depth and the hydraulic capacity of a culvert. Thus, in many culvert designs, it becomes necessary to determine tailwater depth in the outlet channel.

Much engineering judgment and experience is needed to evaluate possible tailwater conditions during floods. A field inspection should be made to check on downstream controls and to determine water stages. Oftentimes tailwater is controlled by a downstream obstruction or by water stages in another stream. Fortunately, most natural channels are wide compared to the culvert and the depth of water in the natural channel is considerably less than critical depth, thus the tailwater is ineffective and channel depth computations are not always warranted.

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation (see page 5-12) if the channel is reasonably uniform in cross section, slope and roughness. Values of n for natural streams for use in Manning's equation may be found in Table 2, appendix B, p. 5-50. If the water surface in the outlet channel is established by downstream controls, other means must be found to determine the tailwater elevation. Sometimes this necessitates a study of the stage-discharge relationship of another stream into which the stream in question flows or the securing of data on reservoir elevations if a storage dam is involved.

Velocity of Culvert Flow

A culvert, because of its hydraulic characteristics, increases the velocity of flow over that in the natural channel. High velocities are most damaging just downstream from the culvert outlet and the erosion potential at this point is a feature to be considered in culvert design.

Energy dissipators for channel flow have been investigated in the laboratory and many have been constructed, especially in irrigation channels. Designs for highway use have been developed and constructed at culvert outlets. All energy dissipators add to the cost of a culvert, therefore, they should be used only to prevent or to correct a serious erosion problem. (See references 4 and 5.)

The judgment of engineers working in a particular area is required to determine the need for energy dissipators at culvert outlets. As an aid in evaluating this need, culvert outlet velocities should be computed. These computed velocities can be compared with outlet velocities of alternate culvert designs, existing culverts in the area, or the natural stream velocities. In many streams the maximum velocity in the main channel is considerably higher than the mean velocity for the whole channel cross-section. Culvert outlet velocities should be compared with maximum stream velocities in determining the need for channel protection. A change in size of culvert does not change outlet velocities appreciably in most cases.

Outlet velocities for culverts flowing with inlet control may be approximated by computing the mean velocity for the culvert cross section using Manning's equation

$$V = \frac{1.49}{n} R^{2/3} \quad S_o^{1/2}$$

Since the depth of flow is not known the use of tables or charts is recommended in solving this equation[2/]. The outlet velocity as computed by this method will usually be high because the normal depth, assumed in using Manning's equation, is seldom reached in the relatively short length of the average culvert. Also, the shape of the outlet channel, including aprons and wingwalls, have much to do with changing the velocity occurring at the end of the culvert barrel. Tailwater is not considered effective in reducing outlet velocities for most inlet control conditions.

In outlet control, the average outlet velocity will be the discharge divided by the cross-sectional area of flow at the outlet. This flow area can be either that corresponding to critical depth, tailwater depth (if below the crown of the culvert) or the full cross section of the culvert barrel.

Performance Curves

Although the procedure given in this circular is primarly for use in selecting a size of culvert to pass a given discharge at a given headwater, a better understanding of culvert operation can be gained by plotting performance curves through some range of discharges and barrel slopes. Such curves can also be used to compare the performance of different sizes and types of culverts. The construction of such curves is described in Appendix A, page 5-45.

Inlets and Culvert Capacity

Inlet shape, edge geometry and skew of the entrance affects culvert capacity. Both the shape and edge geometry have been investigated by recent research but the effect of skew for various flow conditions has not been examined. Results show that the inlet edge geometry is particularly important to culvert performance in inlet-control flow. A comparison of several types of commonly used inlets can be made by referring to charts 2 and 5. The type of inlet has some effect on capacity in outlet control but generally the edge geometry is less important than in inlet control. (See reference 6.)

2/ See references page 5-14.

As shown by the inlet control nomograph on Chart 5, the capacity of a thin edge projecting metal pipe can be increased by incorporating the thin edge in a headwall. The capacity of the same thin edged pipe can be further increased if the entrance is rounded, bevelled or tapered by the addition of an attachment or the building of these shapes into a headwall. Although research on improving culvert entrances is not complete, sufficient data are available to permit the construction of Chart 7, an inlet control nomograph for the performance of a bevelled inlet on a circular culvert. A sketch on the nomograph shows the dimensions of two possible bevels. Although nomographs have not been prepared for other barrel shapes, the capacity of box culverts can be increased at little cost by incorporating a bevel into the headwall. In computing headwater depths for outlet control, when the above bevel is used, k_e equals 0.25 for corrugated metal barrels and 0.2 for concrete barrels.

Figure 6 shows a photograph of a bevel constructed in the headwall of a corrugated metal pipe.

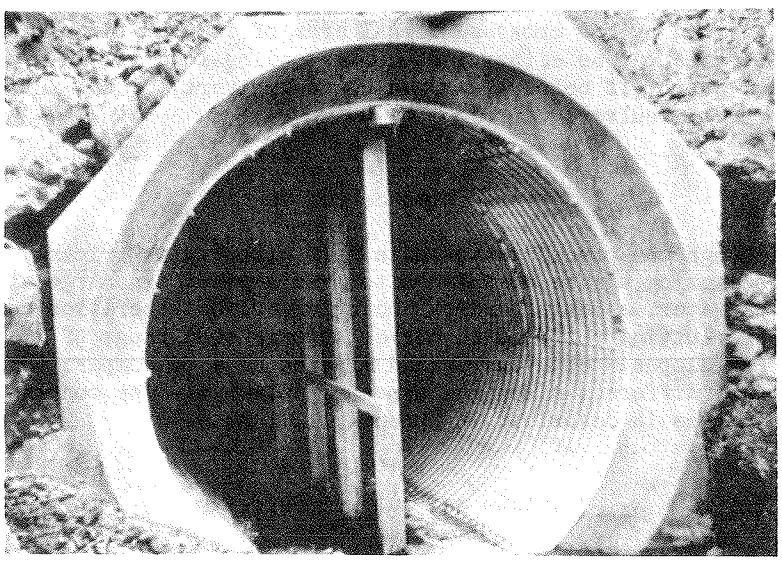


Photo -- Courtesy of Oregon State Highway Department

Figure 6

REFERENCES

1. "Computation of Uniform and Nonuniform Flow in Prismatic Conduits," by P. N. Zelensky, U.S. Department of Transportation, Federal Highway Administration. For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

2. "Handbook of Hydraulics," by H. W. King, McGraw-Hill Book Company, New York City.

3. "Design Charts for Open-Channel Flow," U.S. Department of Transportation, Federal Highway Administration. For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

4. "Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets," by B. P. Fletcher, J. L. Grace, Jr., U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi 39180.

5. "Hydraulic Design of Stilling Basins and Energy Dissipators," by A. J. Peterka, U.S. Department of Interior, Bureau of Reclamation, 1964. For sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, or the Chief Engineer, Bureau of Reclamation, Attention 841, Denver Federal Center, Denver, Colorado 80225.

6. "Hydraulic Design of Improved Inlets for Culverts," by L. J. Harrison, J. L. Morris, J. M. Normann, and F. L. Johnson, Hydraulic Engineering Circular No. 13, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., August 1972.

Procedure for Selection of Culvert Size

Step 1: List design data. (See suggested tabulation form, figure 7, p. 5-18.)

a. Design discharge Q, in cfs., with average return period. (i.e. Q_{25} or Q_{50} etc.)

b. Approximate length L of culvert, in feet.

c. Slope of culvert. (If grade is given in percent, convert to slope in ft. per ft.)

d. Allowable headwater depth, in feet, which is the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert.

e. Mean and maximum flood velocities in natural stream.

f. Type of culvert for first trial selection, including barrel material, barrel cross-sectional shape and entrance type.

Step 2: Determine the first trial size culvert.

Since the procedure given is one of trial and error, the initial trial size can be determined in several ways:

a. By arbitrary selection.

b. By using an approximating equation such as $\frac{Q}{10} = A$ from which the trial culvert dimensions are determined.

c. By using inlet control nomographs (Charts 1-7) for the culvert type selected. If this method is used an $\frac{HW}{D}$ must be assumed, say $\frac{HW}{D} = 1.5$, and using the given Q a trial size is determined.

If any trial size is too large in dimension because of limited height of embankment or availability of size, multiple culverts may be used by dividing the discharge equally between the number of barrels used. Raising the embankment height or the use of pipe arch and box culverts with width greater than height should be considered. Final selection should be based on an economic analysis.

Step 3: **Find headwater depth for trial size culvert.**

a. Assuming INLET CONTROL

(1) Using the trial size from step 2, find the headwater depth HW by use of the appropriate inlet control nomograph (Charts 1-7). Tailwater TW conditions are to be neglected in this determination. HW in this case is found by multiplying $\frac{HW}{D}$ obtained from the nomographs by the height of culvert D.

(2) If HW is greater or less than allowable, try another trial size until HW is acceptable for inlet control before computing HW for outlet control.

b. Assuming OUTLET CONTROL

(1) Approximate the depth of tailwater TW, in feet, above the invert at the outlet for the design flood condition in the outlet channel. (See general discussion on tailwater, p. 5-11.)

(2) For tailwater TW elevation equal to or greater than the top of the culvert at the outlet set h_o equal to TW and find HW by the following equation (equation 3).

$$HW = H + h_o - LS_o$$

where

HW = vertical distance in feet from culvert invert (flow line) at entrance to the pool surface.

H = head loss in feet as determined from the appropriate nomograph (Charts 8-14)

h_o = vertical distance in feet from culvert invert at outlet to the hydraulic grade line (In this case h_o equals TW, measured in feet above the culvert invert.)

S_o = slope of barrel in ft./ft.

L = culvert length in ft.

(3) For tailwater TW elevations less than the top of the culvert at the outlet, find headwater HW by equation 3 as in b(2) above except that

$$h_o = \frac{d_c + D}{2} \text{ or TW, whichever is the greater.}$$

where

d_c = critical depth in ft. (Charts 15 through 20) Note: d_c cannot exceed D

D = height of culvert opening in ft.

Note: Headwater depth determined in b(3) becomes increasingly less accurate as the headwater computed by this method falls below the value $D + (1 + k_e)\frac{V^2}{2g}$. (See discussion under "Culvert Flowing Full with Outlet Control", p. 5-9.)

c. Compare the headwaters found in Step 3a and Step 3b (Inlet Control and Outlet Control). The higher headwater governs and indicates the flow control existing under the given conditions for the trial size selected.

d. If outlet control governs and the HW is higher than is acceptable, select a larger trial size and find HW as instructed under Step 3b. (Inlet control need not be checked, since the smaller size was satisfactory for this control as determined under Step 3a.)

Step 4: Try a culvert of another type or shape and determine size and HW by the above procedure.

Step 5: Compute outlet velocities for size and types to be considered in selection and determine need for channel protection.

108 a. If outlet control governs in Step 3c above, outlet velocity equals $\frac{Q}{A_o}$, where A_o is the cross-sectional area of flow in the culvert barrel at the outlet. If d_c or TW is less than the height of the culvert barrel use A_o corresponding to d_c or TW depth, whichever gives the greater area of flow. A_o should not exceed the total cross-sectional area A of the culvert barrel.

b. If inlet control governs in step 3c, outlet velocity can be assumed to equal mean velocity in open-channel flow in the barrel as computed by Manning's equation for the rate of flow, barrel size, roughness and slope of culvert selected.

Note: Charts and tables are helpful in computing outlet velocities. (See references p. 5-14.)

Step 6: Record final selection of culvert with size, type, required headwater, outlet velocity, and economic justification.

PROJECT: ______________

DESIGNER: ______________

DATE: ______________

HYDROLOGIC AND CHANNEL INFORMATION

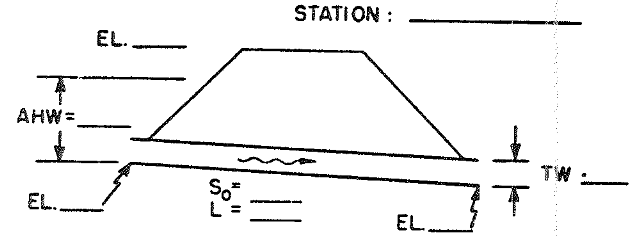
Q_1 = ________ TW_1 = ________

Q_2 = ________ TW_2 = ________

(Q_1 = DESIGN DISCHARGE , SAY Q_{25}
Q_2 = CHECK DISCHARGE , SAY Q_{50} OR Q_{100})

SKETCH

STATION : ______________



MEAN STREAM VELOCITY = ______

MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				

SUMMARY & RECOMMENDATIONS:

Figure 7

INLET-CONTROL NOMOGRAPHS

Charts 1 through 7

Instructions for Use

1. To determine headwater (HW), given Q, and size and type of culvert.

   a. Connect with a straightedge the given culvert diameter or height (D) and the discharge Q, or $\frac{Q}{B}$ for box culverts; mark intersection of straightedge on $\frac{HW}{D}$ scale marked (1).

   b. If $\frac{HW}{D}$ scale marked (1) represents entrance type used, read $\frac{HW}{D}$ on scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (a) horizontally to scale (2) or (3) and read $\frac{HW}{D}$.

   c. Compute HW by multiplying $\frac{HW}{D}$ by D.

2. To determine discharge (Q) per barrel, given HW, and size and type of culvert.

   a. Compute $\frac{HW}{D}$ for given conditions.

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point on $\frac{HW}{D}$ scale (1) as found in (b) above and the size of culvert on the left scale. Read Q or $\frac{Q}{B}$ on the discharge scale.

   d. If $\frac{Q}{B}$ is read in (c) multiply by B (span of box culvert) to find Q.

3. To determine culvert size, given Q, allowable HW and type of culvert.

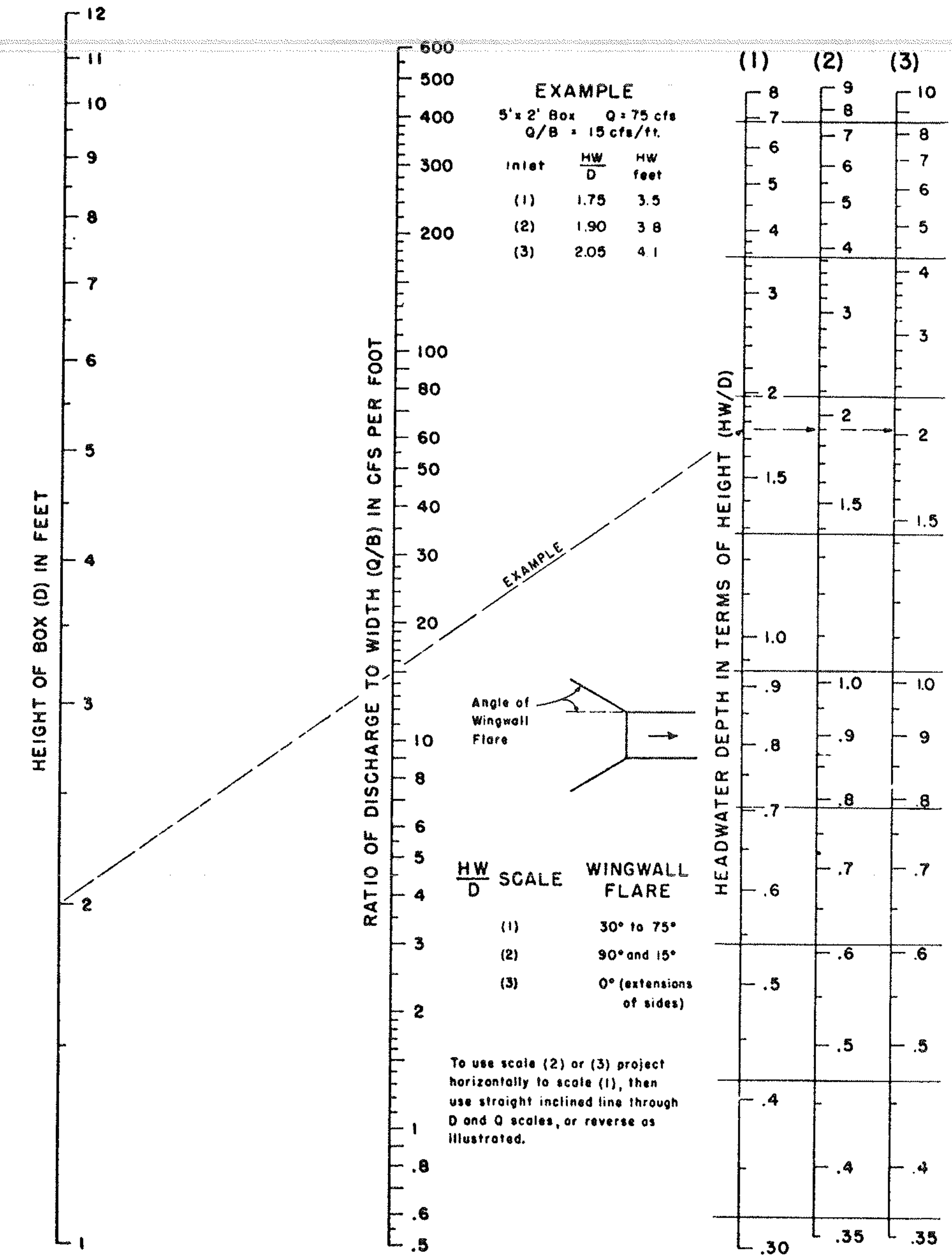
   a. Using a trial size, compute $\frac{HW}{D}$

   b. Locate $\frac{HW}{D}$ on scale for appropriate entrance type. If scale (2) or (3) is used, extend $\frac{HW}{D}$ point horizontally to scale (1).

   c. Connect point on $\frac{HW}{D}$ scale (1) as found in (b) above to given discharge and read diameter, height or size of culvert required for $\frac{HW}{D}$ value.

   d. If D is not that originally assumed, repeat procedure with a new D.

CHART I



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

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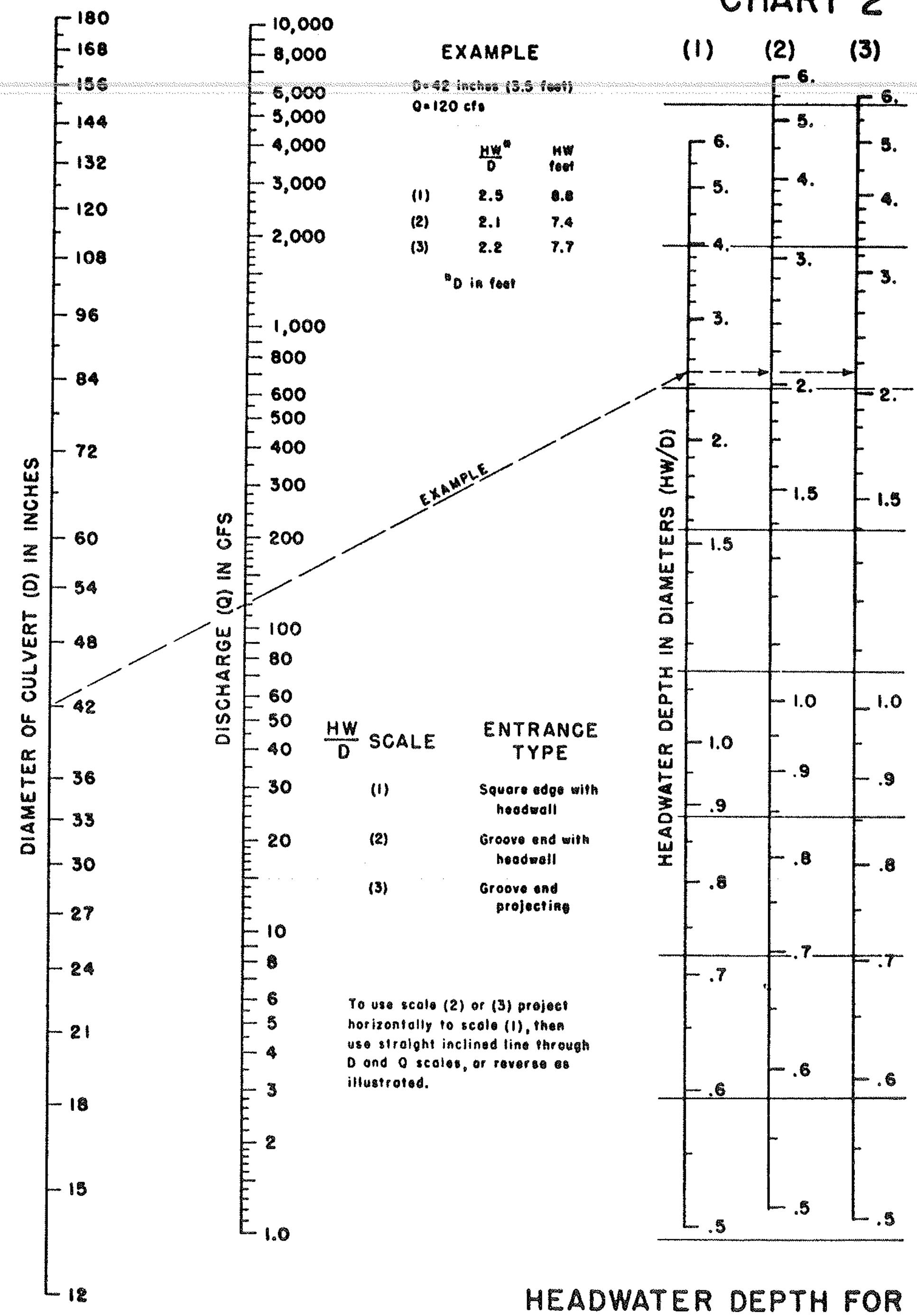
5-22

112

CHART 2



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3 REVISED MAY 1964

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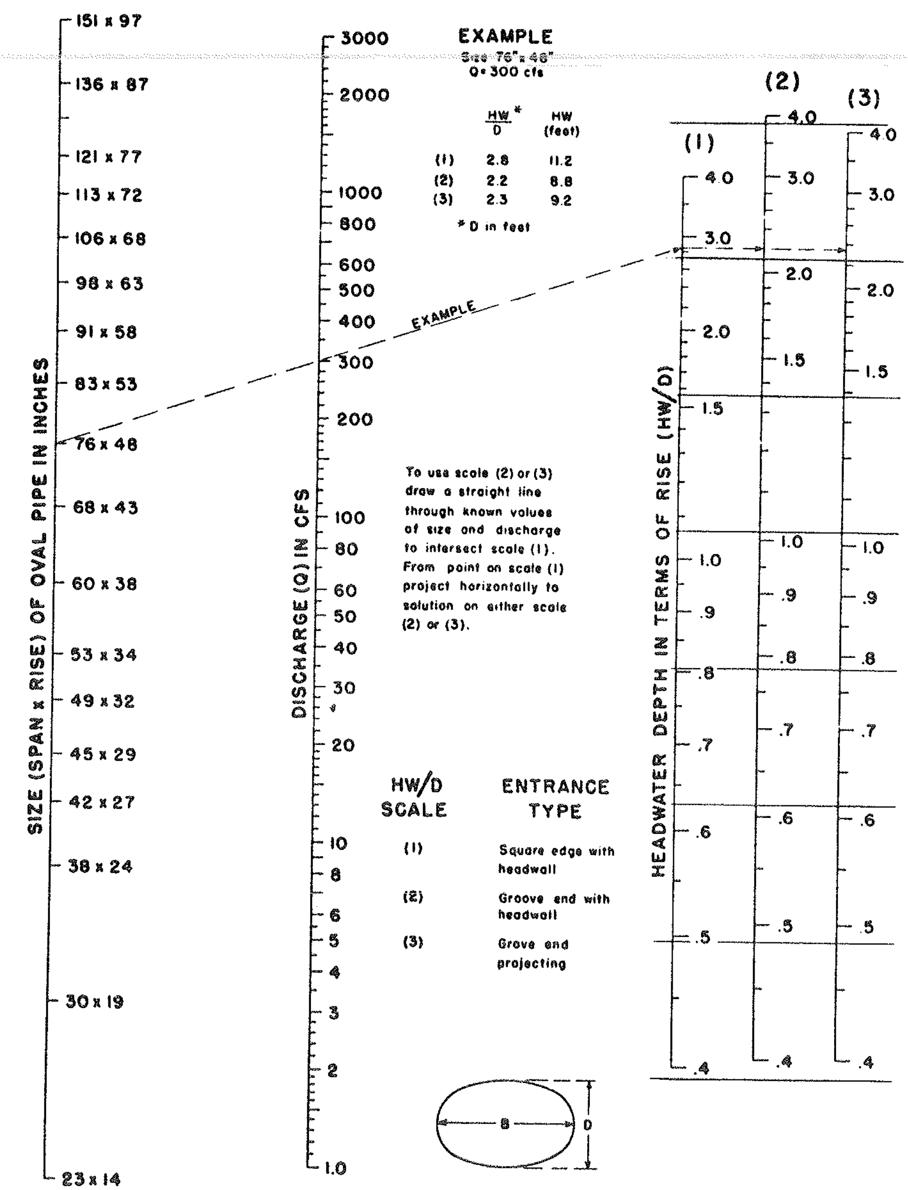
CHART 3



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

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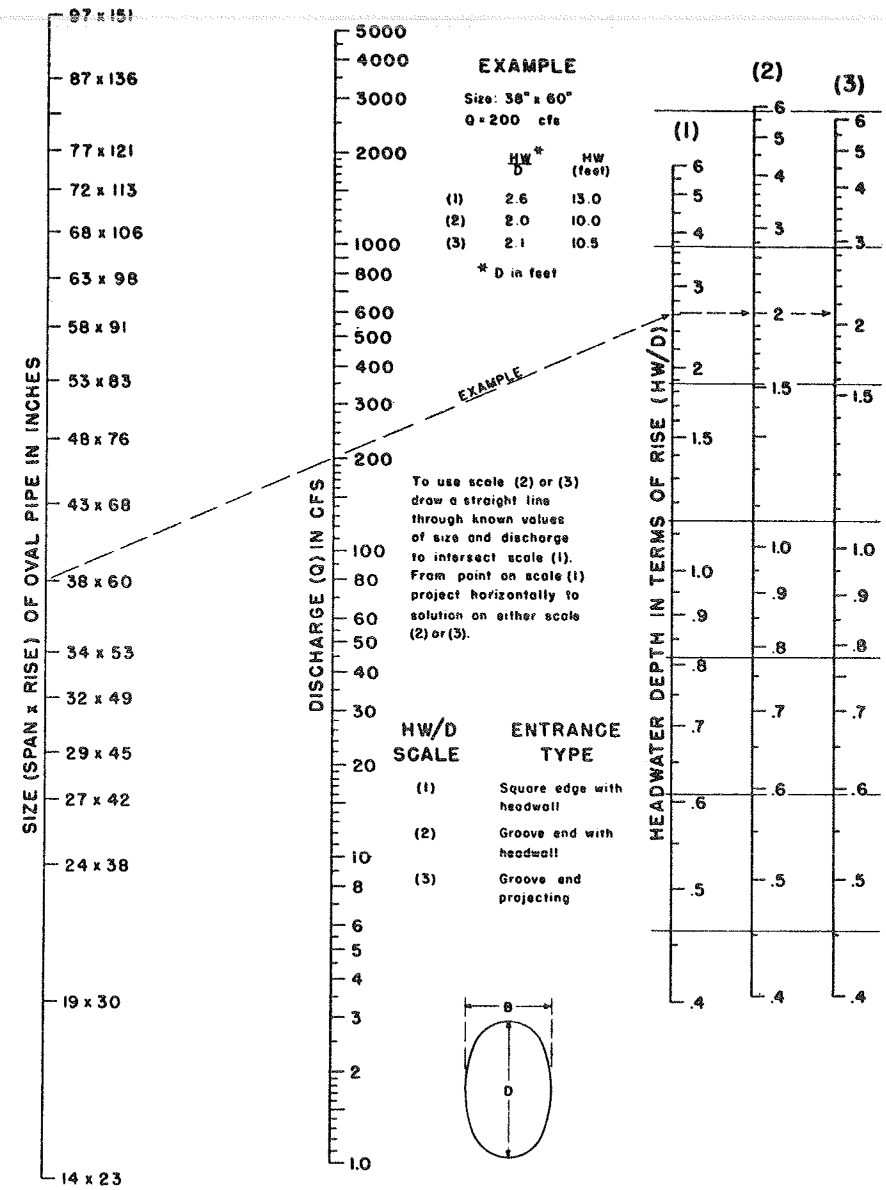
113

CHART 4



HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

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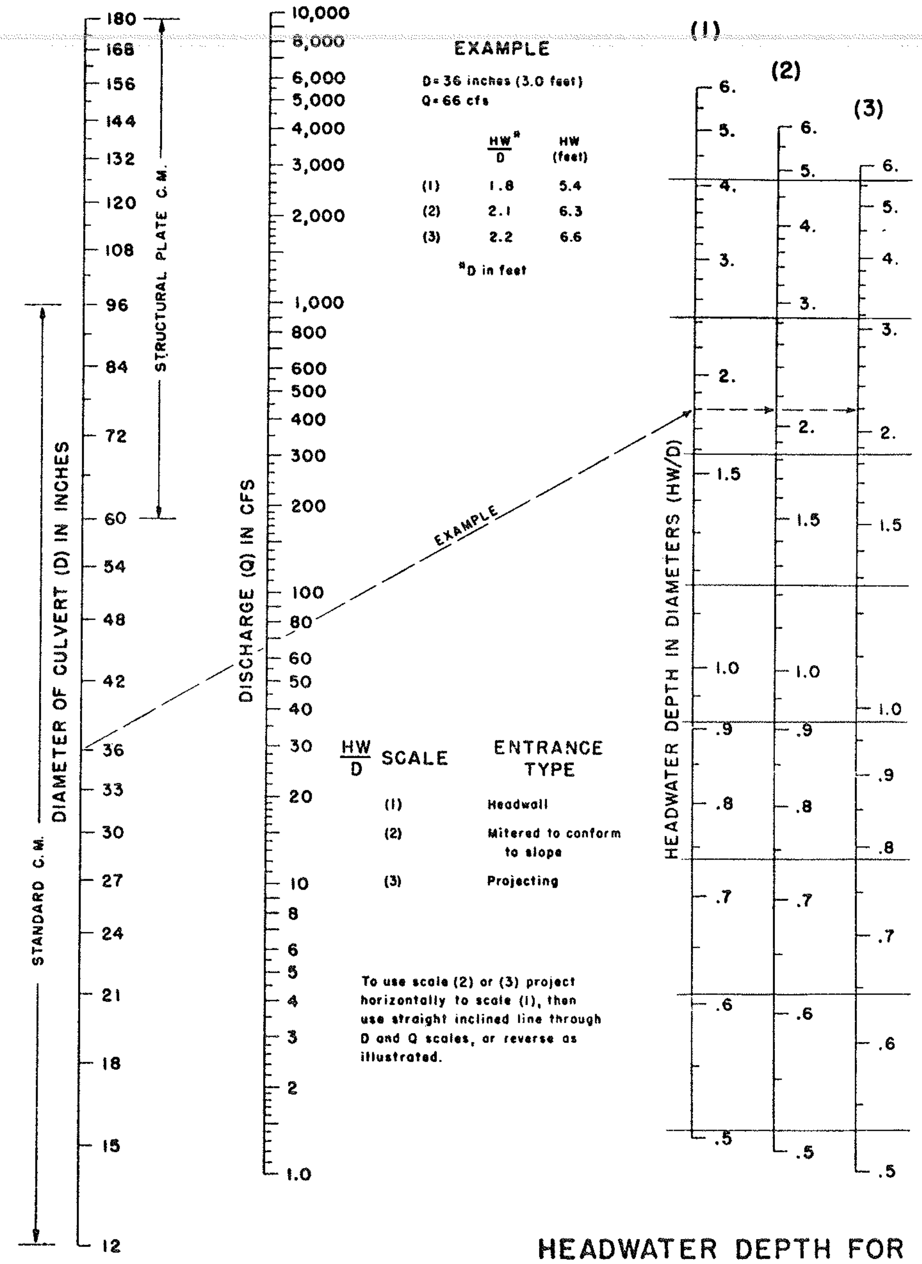
CHART 5



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

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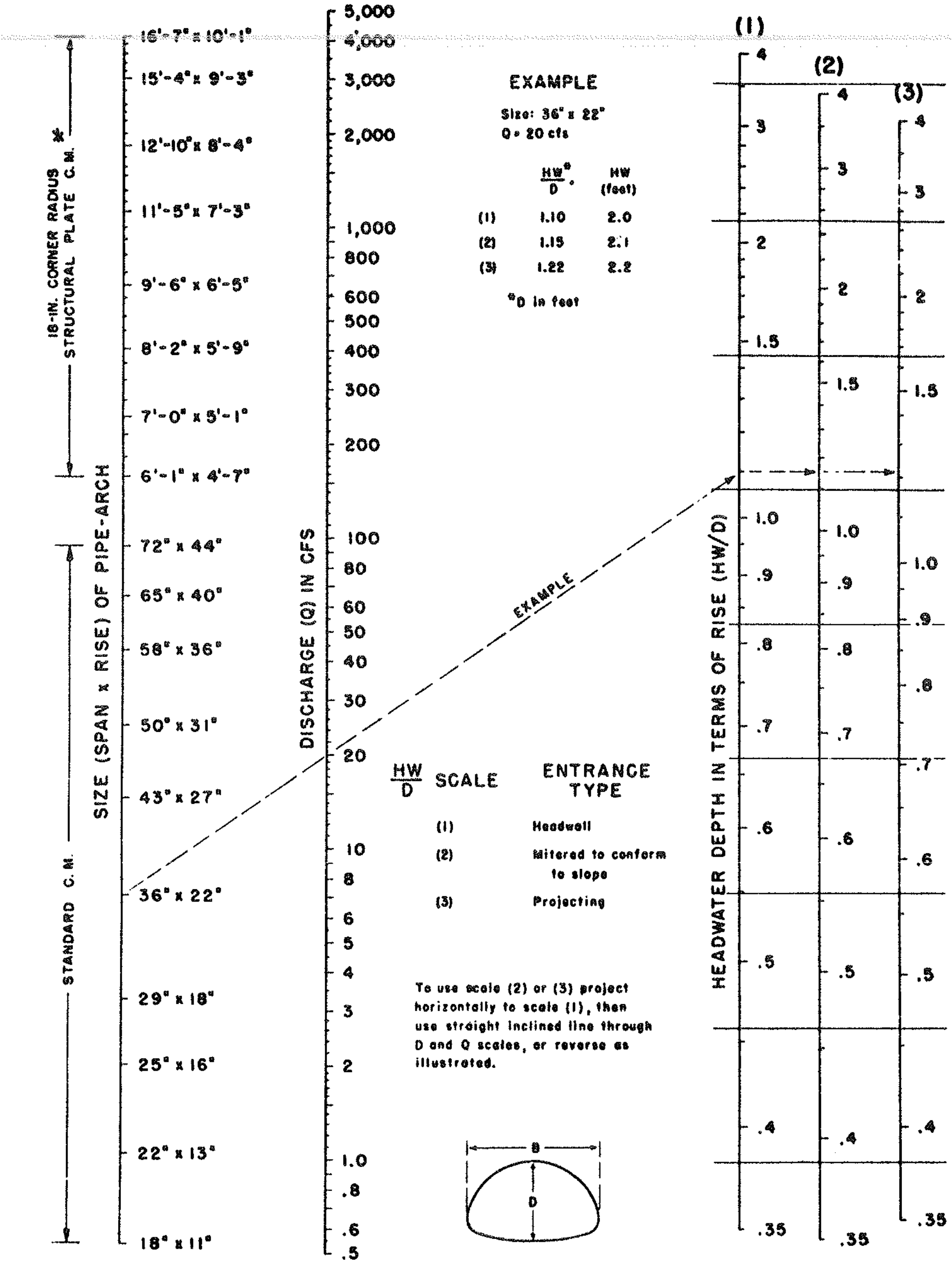
115

CHART 6



*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

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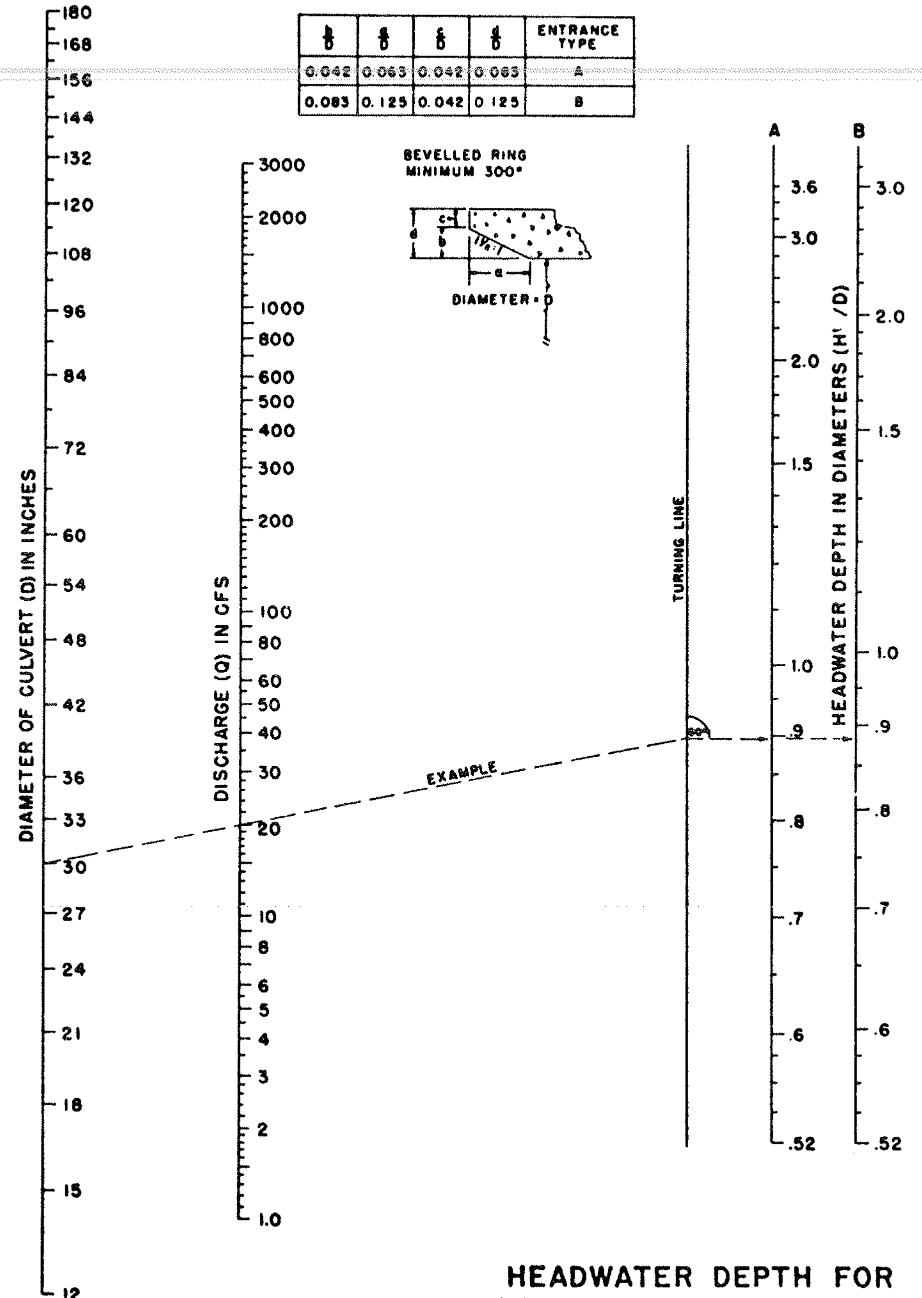
HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

116

Chart 7

b/D	a/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



FEDERAL HIGHWAY ADMINISTRATION
MAY 1973

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL

OUTLET-CONTROL NOMOGRAPHS

Charts 8 through 14

Instructions for Use

Outlet control nomographs solve equation 2, p. 5-6, for head H when the culvert barrel flows full for its entire length. They are also used to determine head H for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for finding headwater HW, since they only give H in equation 3, $HW = H + h_o - LS_o$. (See discussion for "Culverts Flowing with Outlet Control", p. 5-5.)

1. To determine head H for a given culvert and discharge Q.

   a. Locate appropriate nomograph for type of culvert selected. Find k_e for entrance type in Appendix B, Table 1, p. 5-49.

   b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales follow instructions below:

      (1) If the n value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper k_e and locate the starting point at the given culvert length. If a k_e curve is not shown for the selected k_e, see (2) below. If the n value for the culvert selected differs from that of the nomograph, see (3) below.

118

      (2) For the n of the nomograph and a k_e intermediate between the scales given, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the k_e values.

      (3) For a different roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the formula

      $$L_1 = L\left[\frac{n_1}{n}\right]^2$$ See instruction 2 for n values.

   c. Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line". See instruction 3 below for size considerations for rectangular box culvert.

   d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale. For values beyond the limit of the chart scales, find H by solving equation 2, p. 5-6.

2. Values of n for commonly used culvert materials.

Concrete

Pipe	Boxes
0.012	0.012

Corrugated Metal

	Small Corrugations (2 2/3" x 1/2")	Medium Corrugations (3" x 1")	Large Corrugations (6" x 2")
Unpaved	0.024	0.027	Varies*
25% paved	0.021	0.023	0.026
Fully paved	0.012	0.012	0.012

*Variation in n with diameter shown on charts. The various n values have been incorporated into the nomographs and no adjustment for culvert length is required as instructed in 1b(3).

3. To use the box culvert nomograph, chart 8, for full-flow for other than square boxes. 119

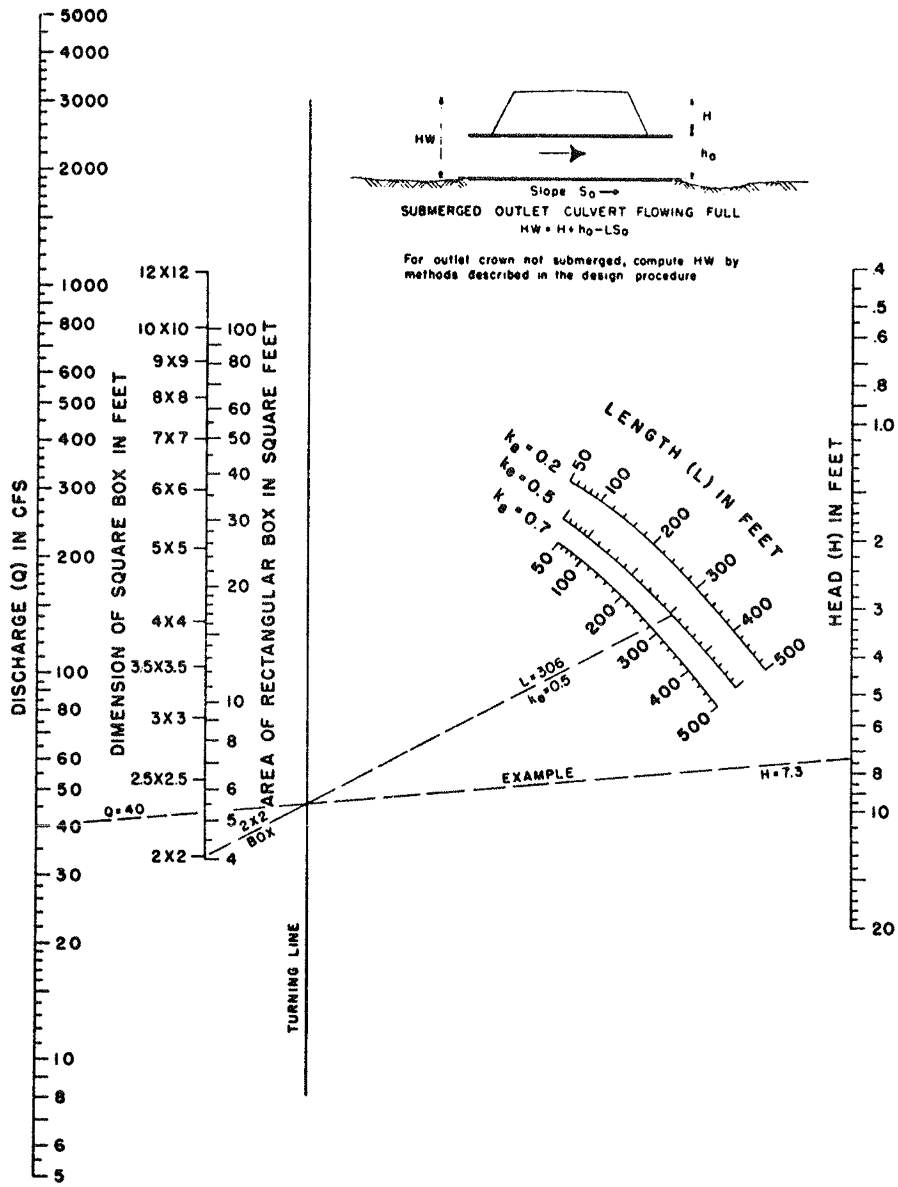
   a. Compute cross-sectional area of the rectangular box.

   b. Connect proper point (see instruction 1) on length scale to barrel area[3/] and mark point on turning line.

   c. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale.

3/ The area scale on the nomograph is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 1/2D. For other box proportions use equation 2 for more accurate results.

CHART 8



120

HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

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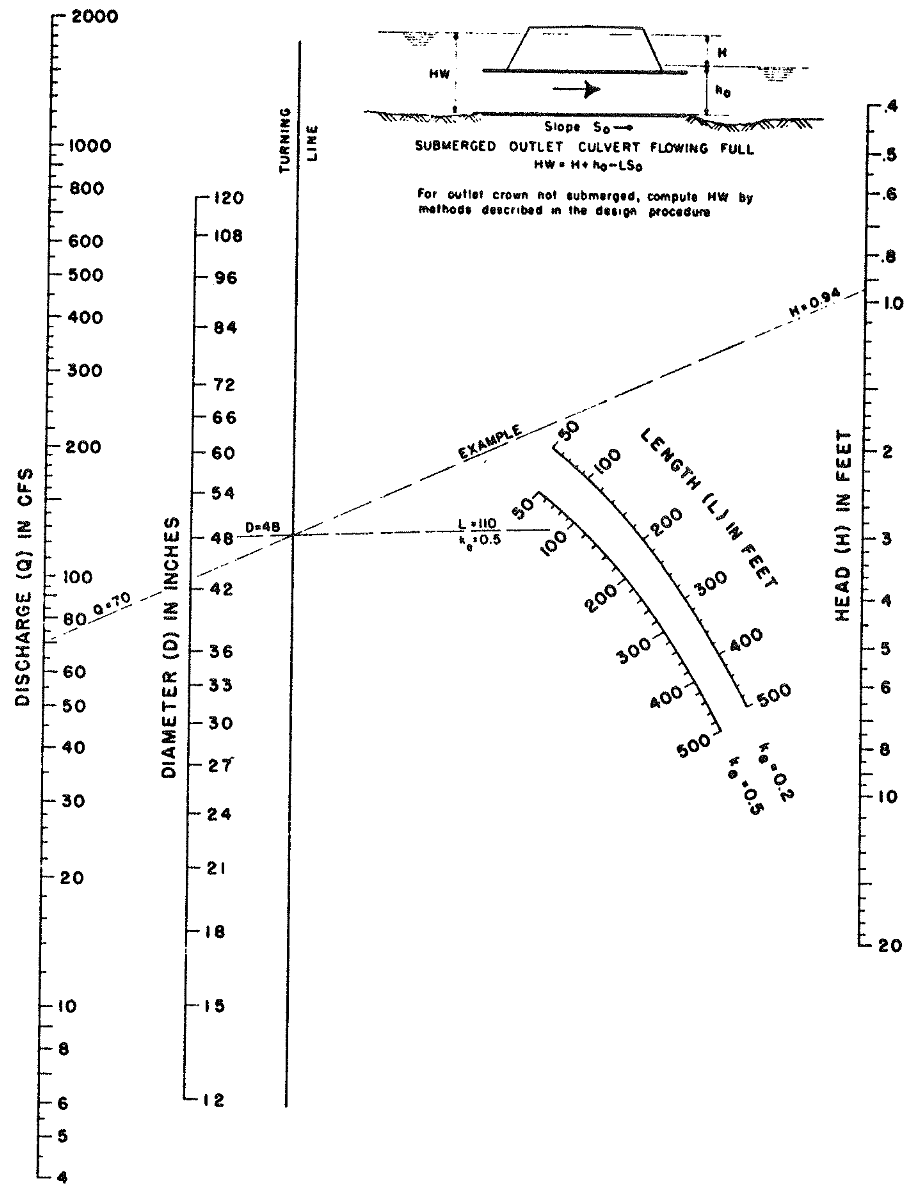
CHART 9



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
$n = 0.012$

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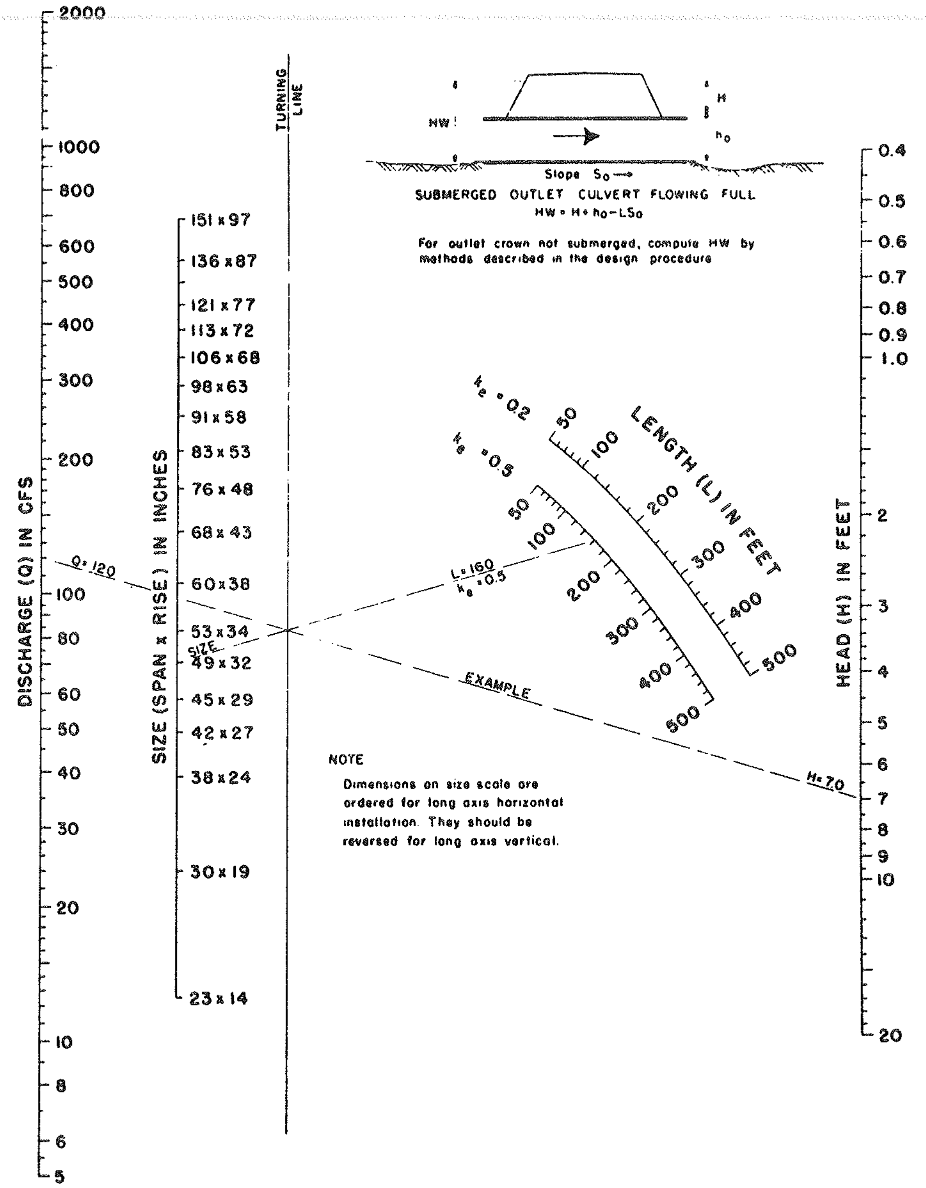
122

CHART 10



HEAD FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
$n = 0.012$

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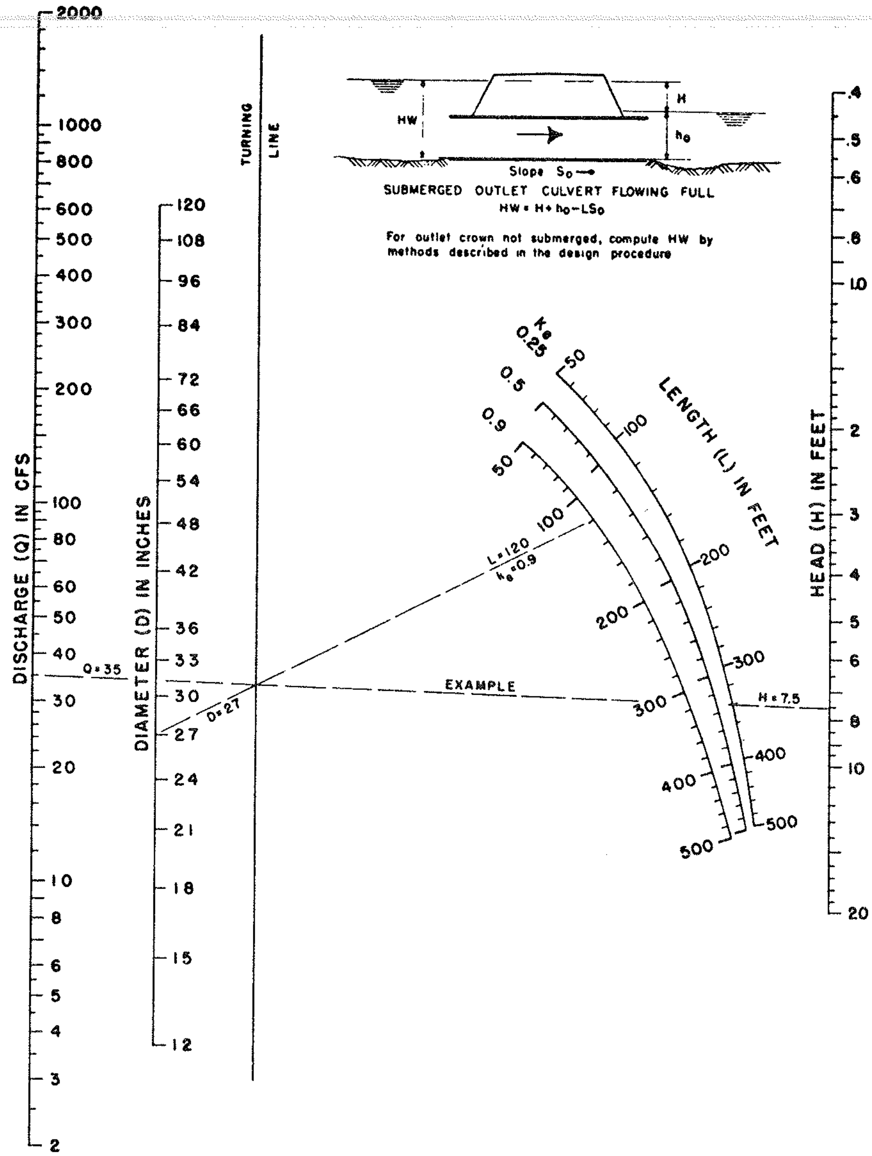
231-382 O - 77 - 3

CHART 11



HEAD FOR STANDARD C. M. PIPE CULVERTS FLOWING FULL n = 0.024

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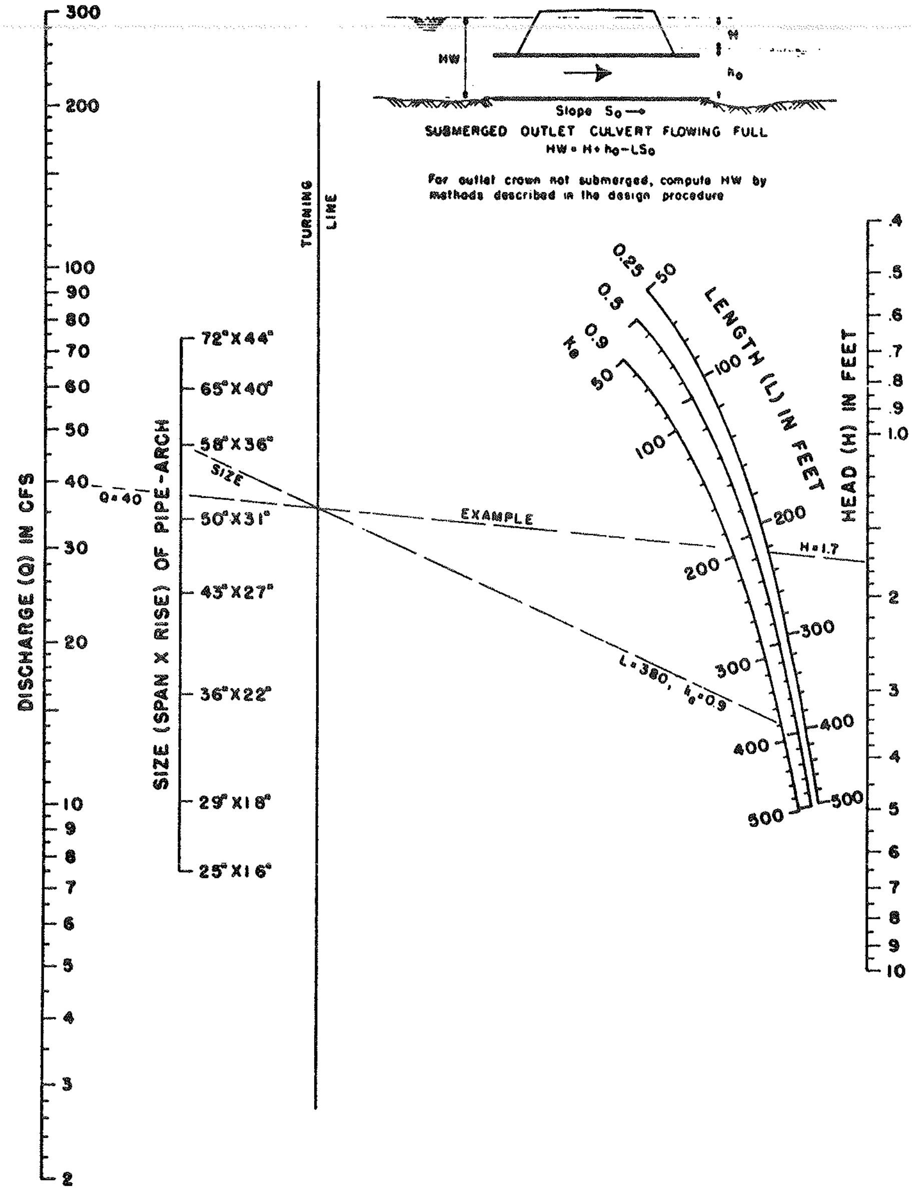
124

CHART 12



HEAD FOR
STANDARD C. M. PIPE-ARCH CULVERTS
FLOWING FULL
n=0.024

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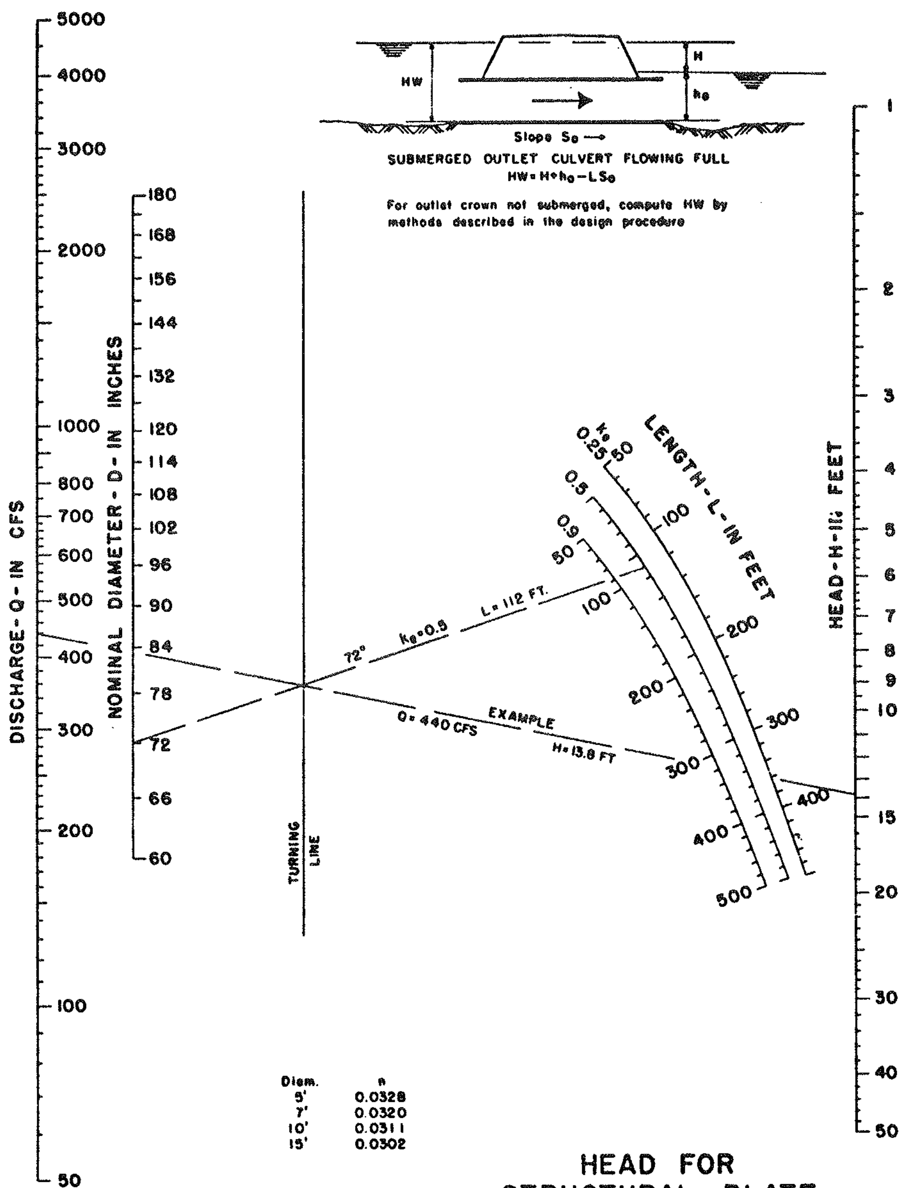
CHART 13



HEAD FOR
STRUCTURAL PLATE
CORR. METAL PIPE CULVERTS
FLOWING FULL
$n = 0.0328$ TO 0.0302

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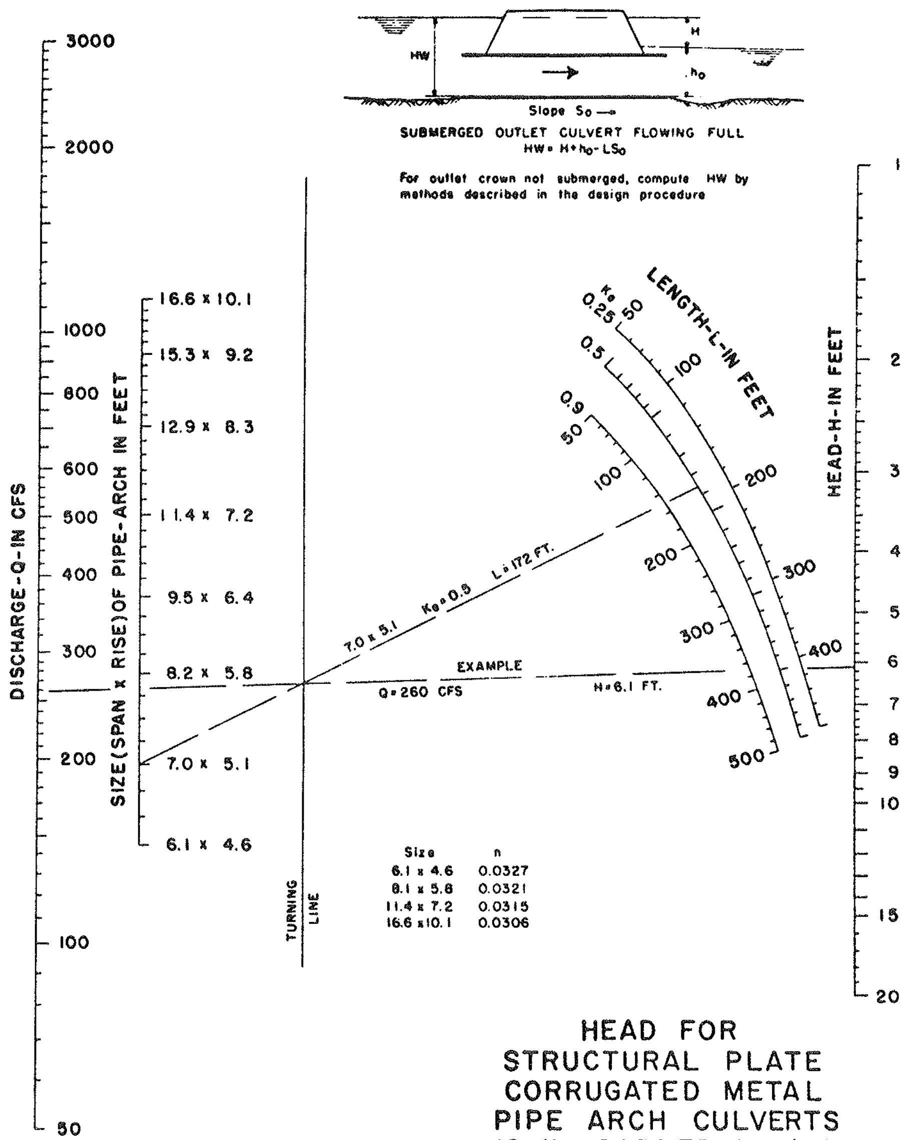
125

126

CHART 14



Size	n
6.1 x 4.6	0.0327
8.1 x 5.8	0.0321
11.4 x 7.2	0.0315
16.6 x 10.1	0.0306

HEAD FOR
STRUCTURAL PLATE
CORRUGATED METAL
PIPE ARCH CULVERTS
18 IN. CORNER RADIUS
FLOWING FULL
n=0.0327 TO 0.0306

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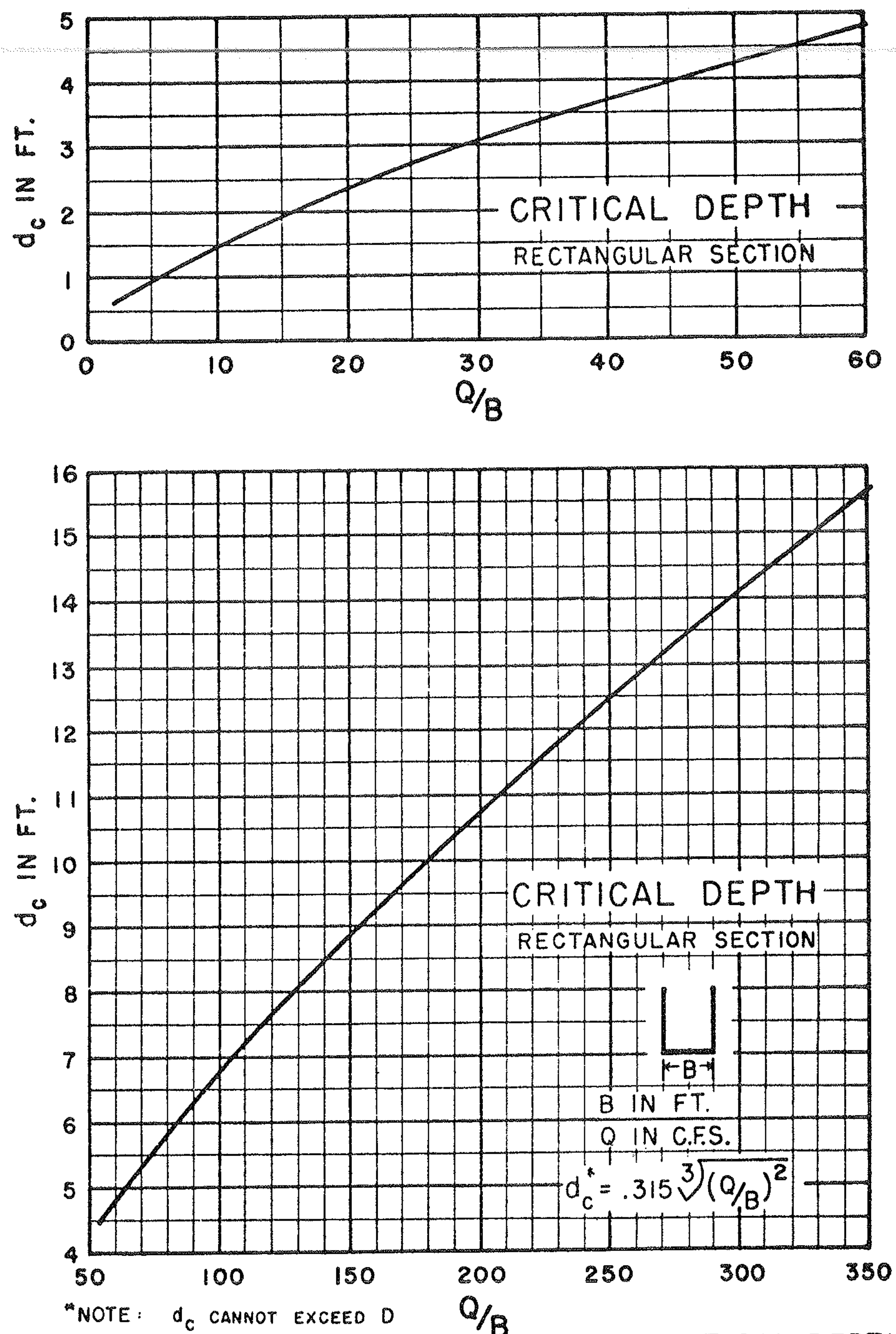
Chart 15



*NOTE: d_c CANNOT EXCEED D

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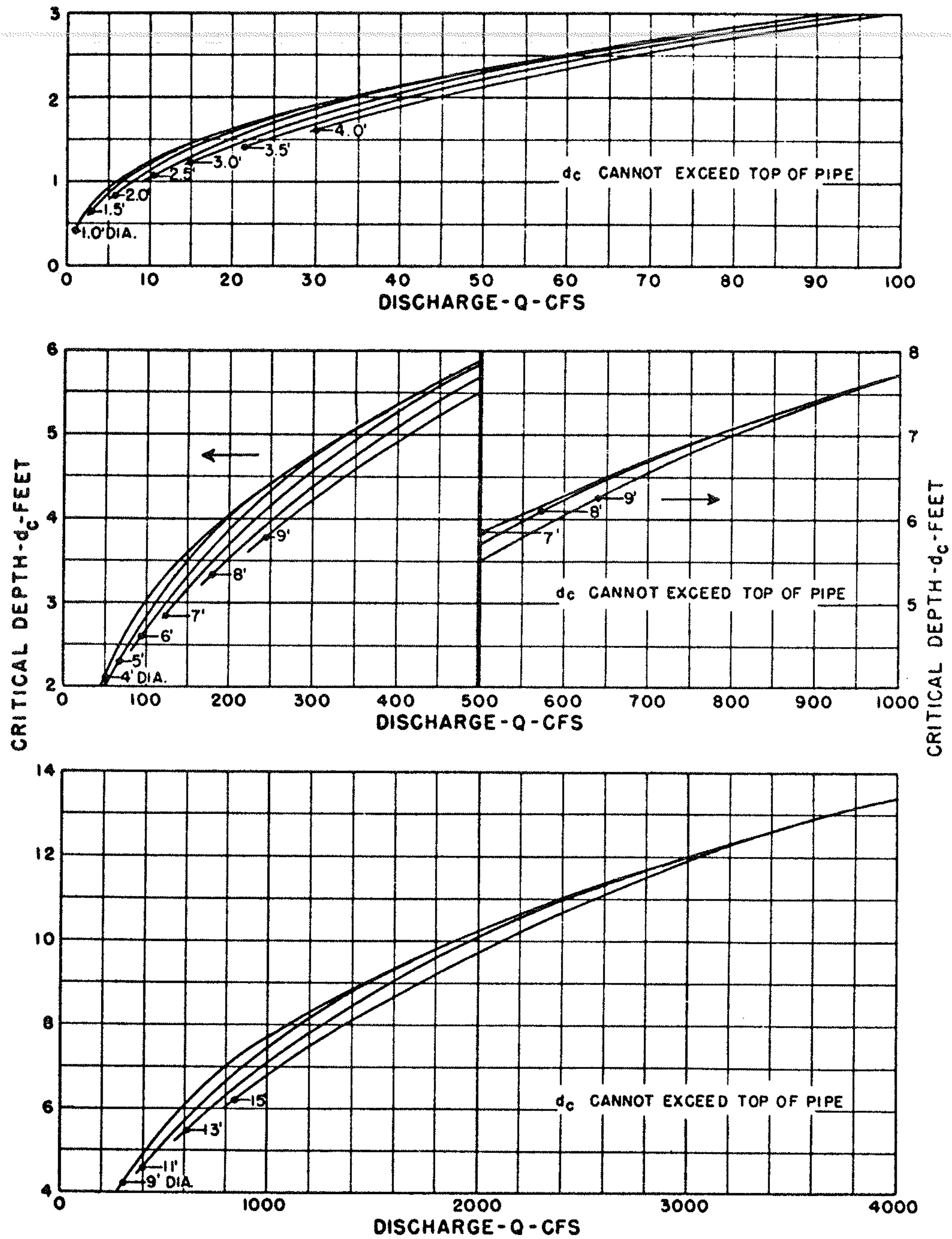
CRITICAL DEPTH
RECTANGULAR SECTION

CHART 16



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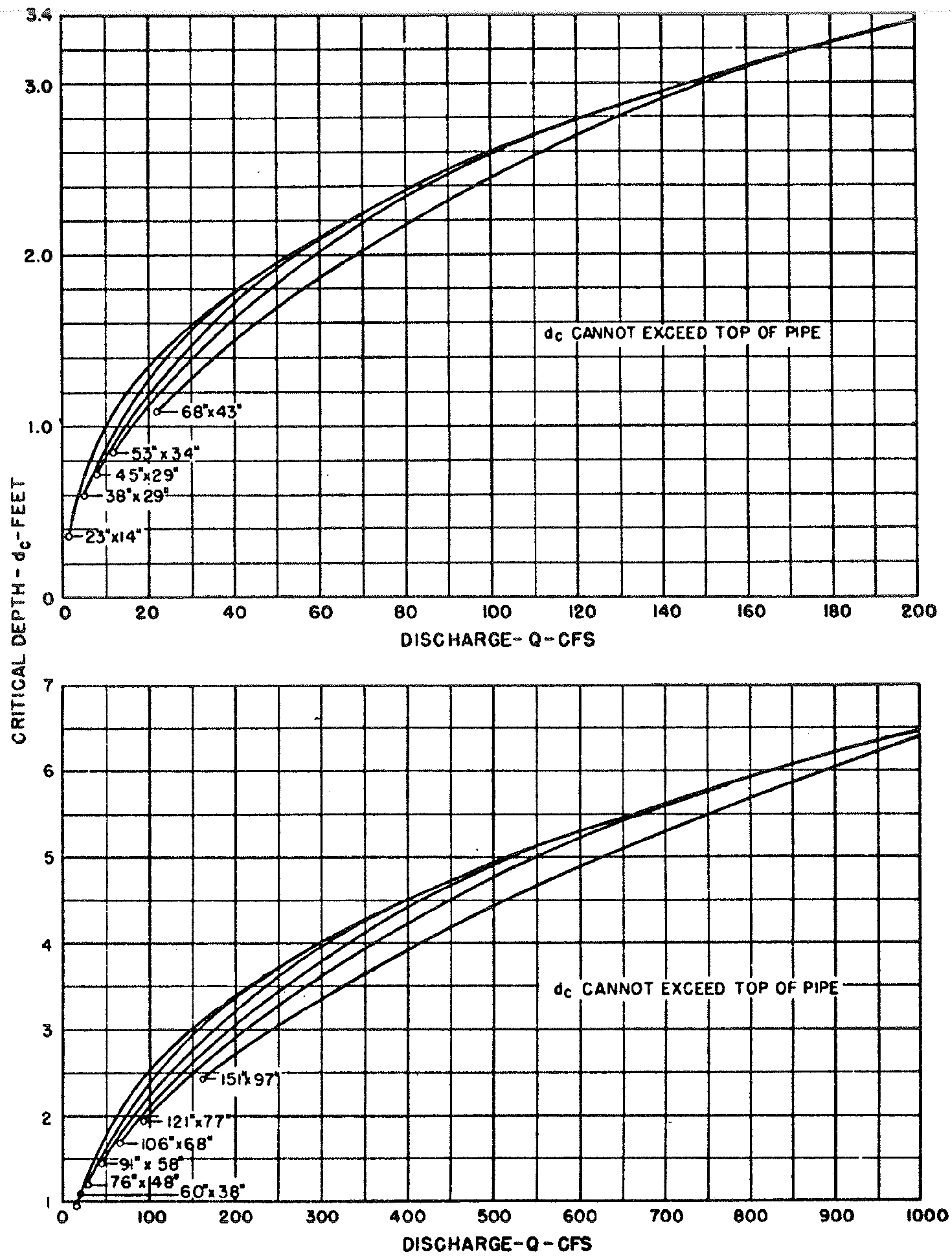
CRITICAL DEPTH
CIRCULAR PIPE

CHART 17



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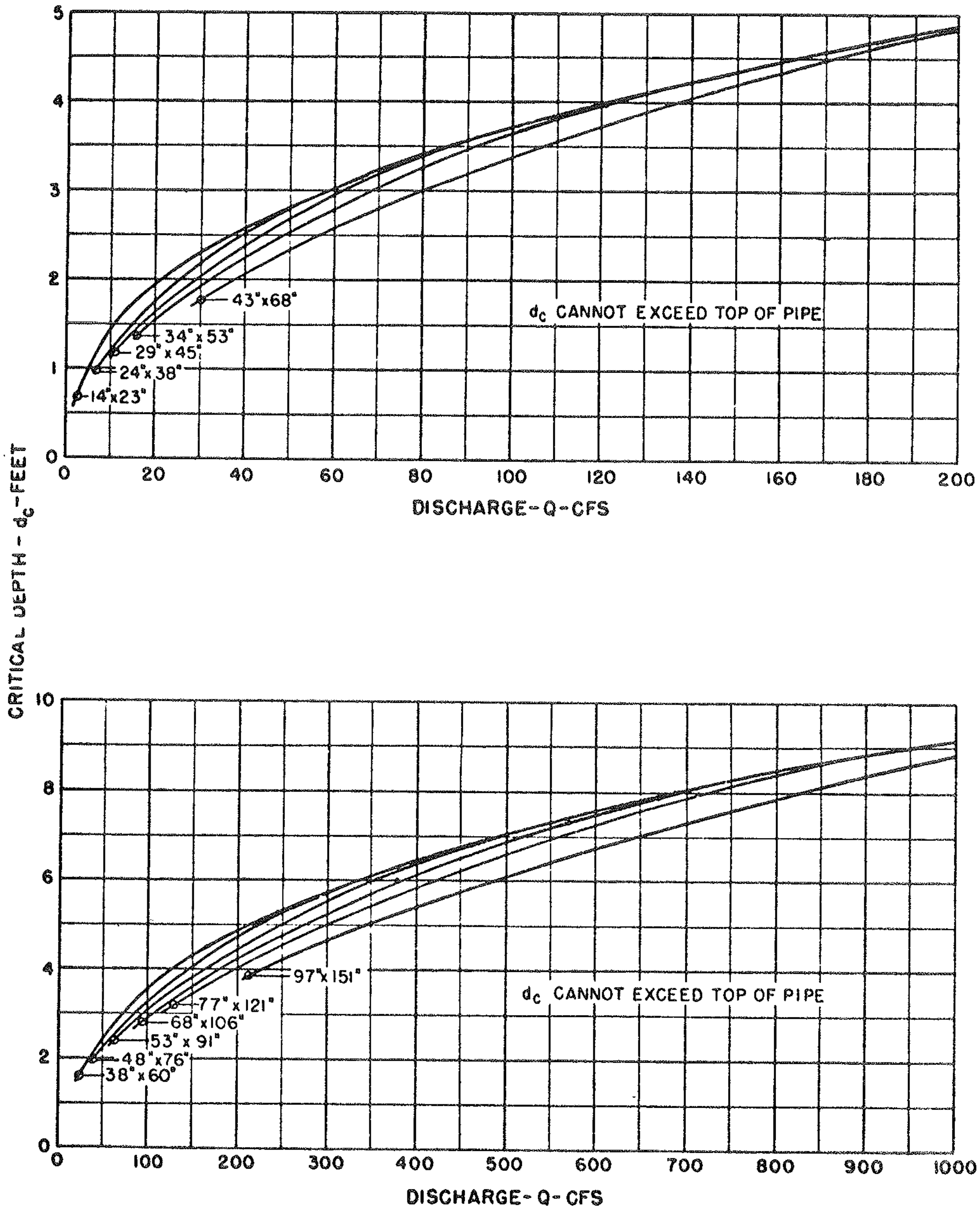
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL

CHART 18



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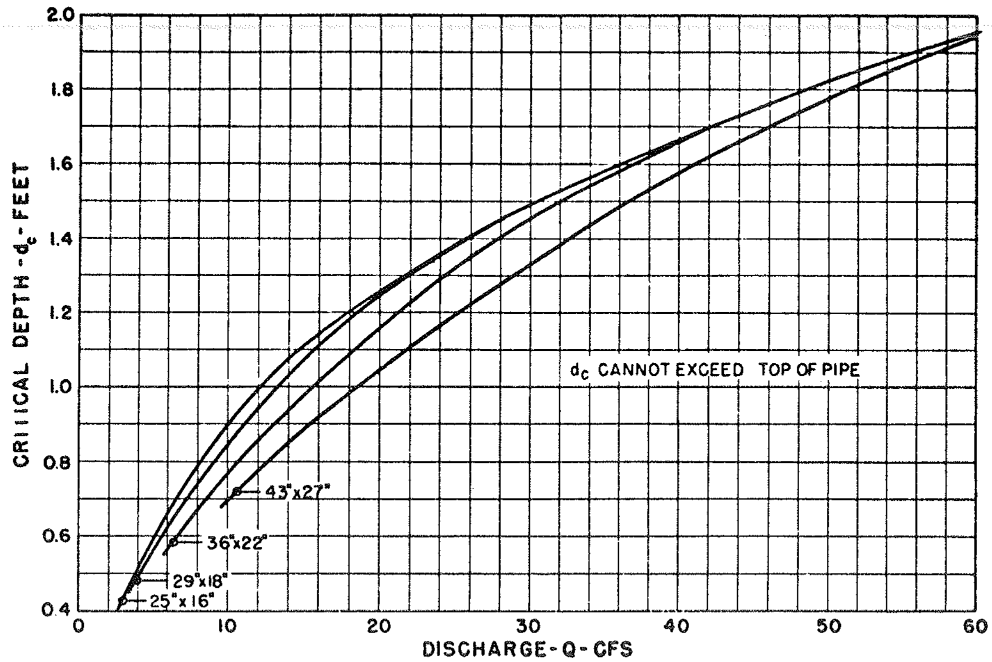
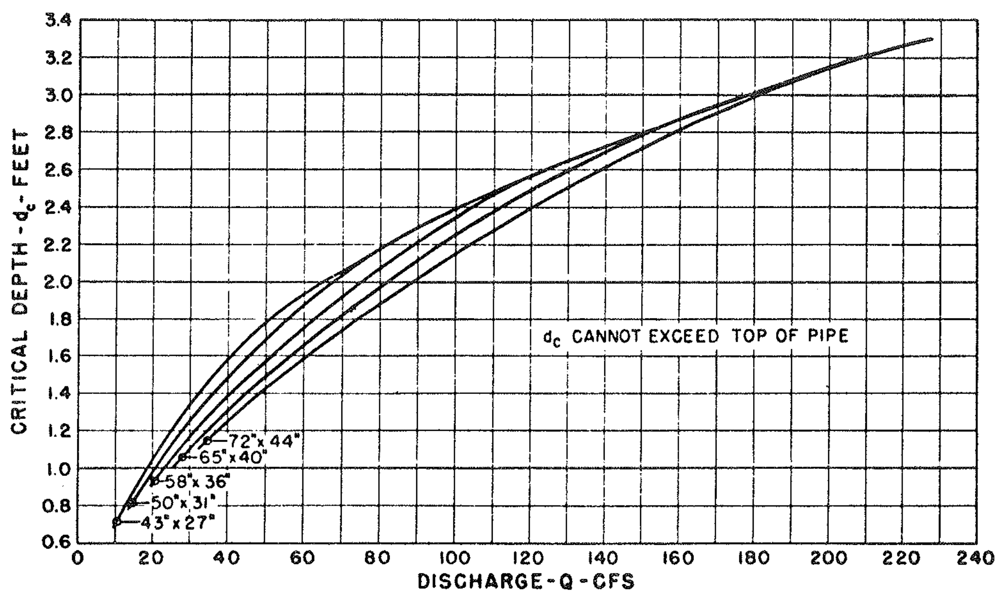
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL

CHART 19





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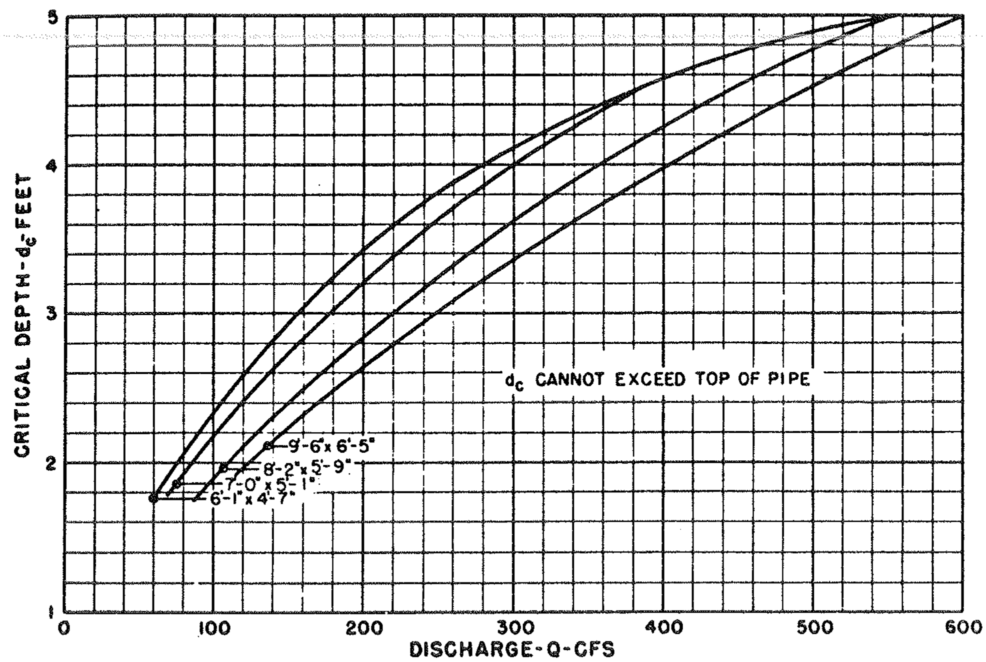
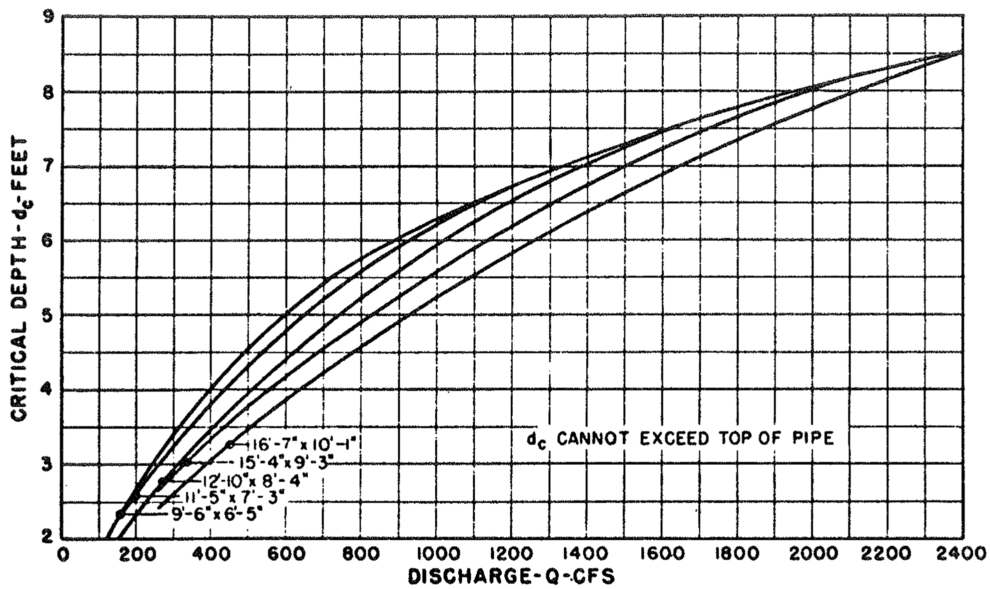
CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH

132

CHART 20





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CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH
18 INCH CORNER RADIUS

Appendix A - PERFORMANCE CURVES

The principal disadvantage in using nomographs for the selection of culvert sizes is that it requires the trial and error solution described in this circular. Some engineers who limit their selection to a relatively small number of types of culverts would find it advantageous to prepare performance curves such as shown in figure 8. These curves are applicable through a range of headwaters and discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes.

Figure 8 is plotted from the data shown in the following tabulations. These data were obtained from the nomographs contained in this circular. (Computer programs are available from Public Roads for making these computations.) The first tabulation is for the inlet-control curve on figure 8, and the second tabultation is for the outlet-control curves.

Data for Inlet-Control Curve

HW*/D (Assume)	Q* (Read)	HW = HW/D x 4
.5	21 c.f.s.	2.0 ft.
.6	29	2.4
.7	37	2.8
.8	46	3.2
.9	56	3.6
1.0	65	4.0
1.1	74	4.4
1.3	90	5.2
1.5	102	6.0
1.7	112	6.8
2.0	126	8.0
2.5	145	10.0
3.0	165	12.0

*From Chart 5 Projecting Inlet (3)

Data for Outlet-Control Curves

Q	d_c	$\frac{d_c + D}{2}$	H	HW for Various S_o				
(Assume)	Chart 16	(Compute)	Chart 11	0%	.5%	1%	1.5%	2.0%
20 cfs	1.3 ft.	2.6 ft.	.2* ft.	2.8 ft.	-	-	-	-
40	1.9	3.0	.8	3.8	2.8	1.8	.8	-
60	2.3	3.2	1.9	5.1	4.1	3.1	2.1	1.1
80	2.7	3.4	3.3	6.7	5.7	4.7	3.7	2.7
100	3.1	3.6	5.2	8.8	7.8	6.8	5.8	4.8
120	3.3	3.6	7.5	11.1	10.1	9.1	8.1	7.1
140	3.5	3.8	10.2	14.0	13.0	12.0	11.0	10.0
160	3.7	3.8	13.6	17.4	16.4	15.4	14.4	13.4

$HW = H + h_o - LS_o$ where $h_o = \frac{d_c + D}{2}$

*From Chart 11 - or by Equation 2.

The curves plotted apply only to the type and length of culvert shown. Culverts placed on grades steeper than about 2.5 percent will operate on the inlet control curve for the headwater-discharge range of this plot. If a free outfall condition does not exist a correction for tailwater should be made as instructed in Step 3b. p. 5-16 of "Procedure for Selection of Culvert Size".

134

HYDRAULIC PERFORMANCE CURVES 48-INCH C.M. PIPE CULVERT WITH PROJECTING INLET

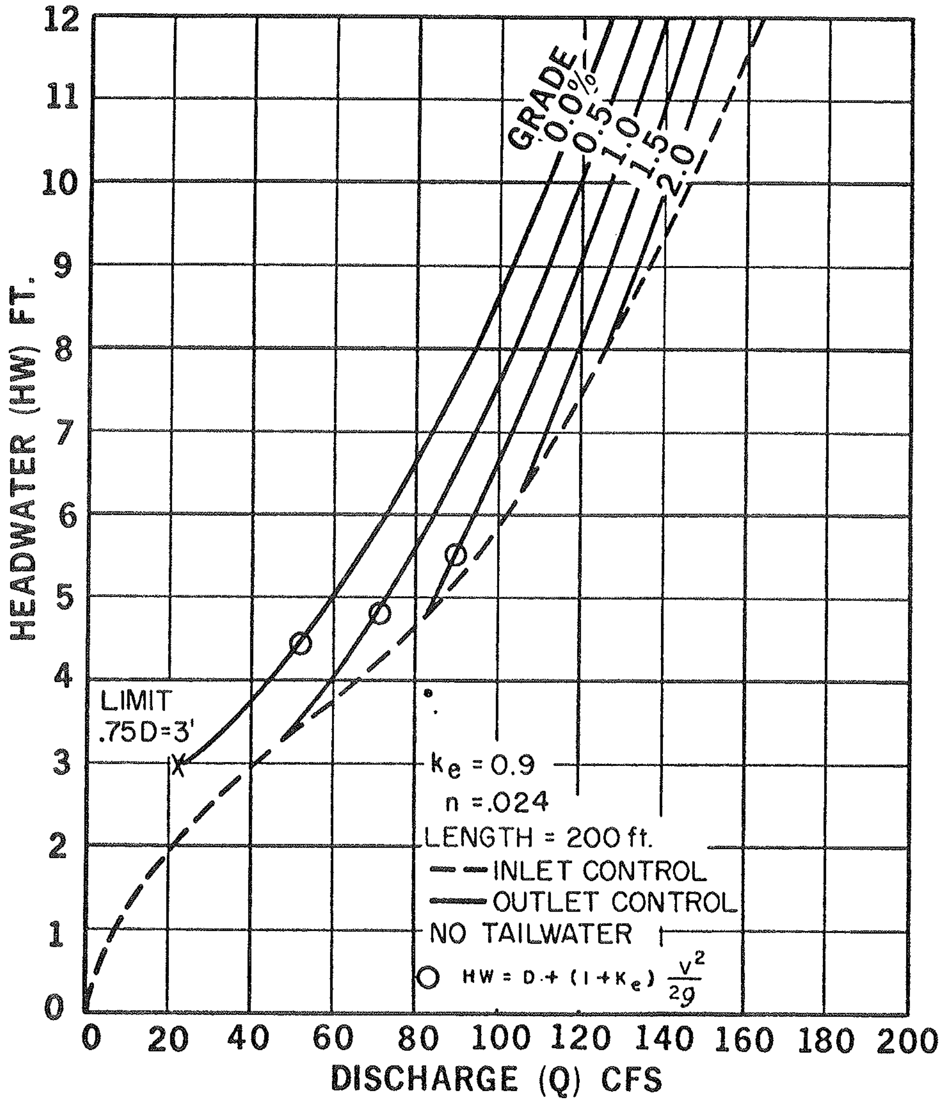


Figure 8

135

TABLE 1 - ENTRANCE LOSS COEFFICIENTS

Outlet Control, Full or Partly Full

Entrance head loss $H_e = k_e \frac{V^2}{2g}$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end) . . .	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge . .	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides . . .	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet, p. 5-13.

Table 2. - Manning's n for Natural Stream Channels[4/]
(Surface width at flood stage less than 100 ft.)

1. Fairly regular section:		
	a. Some grass and weeds, little or no brush	0.030--0.035
	b. Dense growth of weeds, depth of flow materially greater than weed height.	0.035--0.05
	c. Some weeds, light brush on banks	0.035--0.05
	d. Some weeds, heavy brush on banks	0.05 --0.07
	e. Some weeds, dense willows on banks	0.06 --0.08
	f. For trees within channel, with branches submerged at high stage, increase all above values by.	0.01 --0.02
2. Irregular sections, with pools, slight channel meander; increase values given above about		0.01 --0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:		
	a. Bottom of gravel, cobbles, and few boulders. . .	0.04 --0.05
	b. Bottom of cobbles, with large boulders	0.05 --0.07

4/ From "Design Charts for Open Channel Flow", (see p. 5-14).

Compendium 3

Appendix C - ILLUSTRATIVE PROBLEMS

5-51

PROJECT: I-46 (1)

DESIGNER: J. A. F.

DATE: 2-18-64

HYDROLOGIC AND CHANNEL INFORMATION

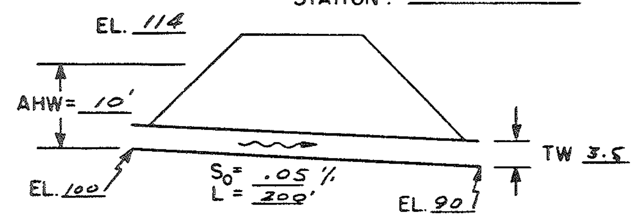
Q_1 = 180 cfs. = Q_{25} TW_1 = 3.5

Q_2 = 225 cfs. = Q_{50} TW_2 = 4.0

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 6+21



MEAN STREAM VELOCITY = 10'/sec

MAX. STREAM VELOCITY = 14'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				
CIRCULAR CMP PROJ. ENT.	180		ASSUME 1.5	7.5	D= 60"± Try Smaller Size AHW = 10'											
"	180	54"	2.2	9.9	.9	9.7	3.9	4.2	3.5	4.2	10.0	3.9	9.9	16.5		
"	225	54"	3.15	14.2	.9	15.3	4.2	4.4	4.0	4.4	10.0	9.7	14.2	17.0		HW high for Q_{50} - Try 60"
"	180	60"	1.51	7.55	.9	5.9	3.9	4.4	3.5	4.4	10.0	0.3	7.55	16.7		
"	225	60"	2.1	10.5	.9	9.3	4.2	4.6	4.0	4.6	10.0	3.9	10.5	17.5		

SUMMARY & RECOMMENDATIONS: VELOCITIES READ FROM CHART 46, 47 - "DESIGN CHARTS FOR OPEN CHANNEL FLOW". (SEE P. 5-14). OUTLET VELOCITIES ARE ABOUT THE SAME FOR EACH SIZE, INDICATING CHANGE IN SIZE HAS LITTLE EFFECT. SIZE SELECTED (60 OR 54-INCH) DEPENDS ON DESIGNER'S CONFIDENCE IN FLOOD ESTIMATE AND DAMAGE INCURRED IF A LARGER FLOOD SHOULD OCCUR. NOTE THAT HW MUST BE GREATER THAN 10.1' FOR OUTLET CONTROL TO GOVERN FOR THE 54" PIPE FLOWING 180 CFS. ACCURATE DETERMINATION OF TW DEPTHS IS UNNECESSARY IN MOST CASES.

Text 4

PROJECT: 142 B

DESIGNER: L.D.K

DATE: 2-18-64

HYDROLOGIC AND CHANNEL INFORMATION

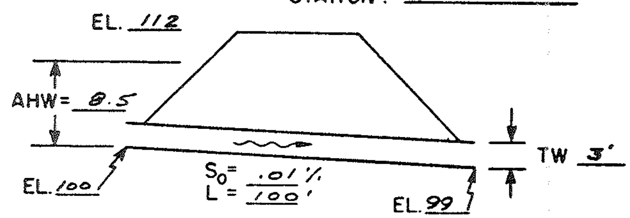
Q_1 = 160 cfs = Q_{50} TW_1 = 3.0'

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 321+14



MEAN STREAM VELOCITY = 8'/sec
MAX. STREAM VELOCITY = 10'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL HW=H + h_o − LS_o											
			HW/D	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				
CMP (cir.) Headwall	160	Assume 54"	1.56	7.0												Hw less than 8.5'-Try 48"
"	160	48"	2.25	9.0	.5	8.5	3.7	3.8	3	3.8	1.0	11.1	11.1	13.2'/sec		Hw High Try 54"
"	160	54"	1.56	7.0	.5	4.7	3.6	4.1	3	4.1	1.0	7.8	7.8	11.1'/sec		velocity at d_c size o.k.
Concrete (cir) Sq. Edge-Hdwl	160	48"	2.35	9.4	.5	4.7	3.7	3.8	3	3.8	1.0	7.5	9.4	14'/sec.		Hw high Try 54"
"	160	54"	1.6	7.2	.5	2.9	3.6	4.1	3	4.1	1.0	6.0	7.2	14.7'/sec		Hw ok. Vel > CMP Try 48 G. Edge
Concrete (cir) Groove end - Hdwl	160	48	1.95	7.8	.2	4.0	3.7	3.8	3	3.8	1.0	6.8	7.8	14.0'/sec		Hw ok Vel. high

SUMMARY & RECOMMENDATIONS:

The selection of a 54" CMP with headwall will keep the headwater below the AHW with a minimum outlet velocity. A 48" concrete pipe with groove edged entrance gives equal HW and slightly higher outlet velocity. Protection of outlet channel might be necessary in some locations.

5-52

Text 4

PROJECT: E 14-2(3)

DESIGNER: FPR

DATE: 2-20-64

HYDROLOGIC AND CHANNEL INFORMATION

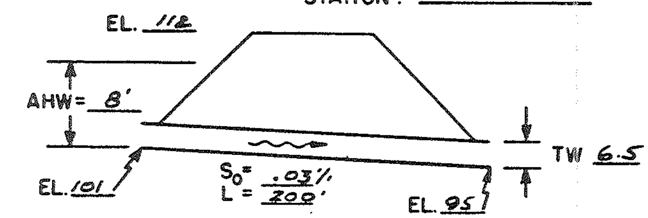
Q_1 = 400 cfs. Q_{50} TW_1 = 6.5'

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION : 8+61



MEAN STREAM VELOCITY = 8'/sec
MAX. STREAM VELOCITY = 12'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. HW/D	INLET CONT. HW	OUTLET CONTROL $HW=H+h_o-LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
Concrete (C/R) Gr. End Proj	400		Assume 1.5			Find	D =	78"								HW = 9.1 Too high - Try 86"
"	400	84"	1.18	8.3												HW High Try 90"
"	400	90"	1.05	7.9	.2	1.9	5.2	6.3	6.5	6.5	6.0	2.4	7.9	28'/sec I.C. 10'/sec O.C.		If too large Try 2 pipes
Same type 2 pipes	200	54"	1.85	8.3												Too small
"	200	60"	1.38	6.9	.2	3.4	4.0	4.5	6.5	6.5	6.0	3.9	6.9	23'/sec I.C. 10'/sec O.C.		Use See Comments
Cir. CMP Bevel B (chart 7)	200	60"	1.34	6.7	.25	6.2	4.0	4.5	6.5	6.5	6.0	6.7	6.7	14'/sec I.C. 10'/sec O.C.		Use. Bevel A can be used here
		Might try single concrete oval or metal arches														

SUMMARY & RECOMMENDATIONS: PROBLEM TO ILLUSTRATE USE OF DOUBLE PIPES IF ONE PIPE IS TOO HIGH OR NOT AVAILABLE. INLET CONTROL GOVERNS. TW SUBMERGES CULVERT OUTLET FOR ALL DOUBLE BARRELS. VELOCITIES ARE COMPUTED FOR BOTH INLET CONTROL AND FOR FULL FLOW AT OUTLET CAUSED BY TW. TWO 60-INCH CONCRETE PIPES OR TWO 60-INCH CMP WITH INLETS SHOWN SATISFY HEADWATER LIMITATIONS. CONCRETE PIPE WILL GIVE CONSIDERABLY HIGHER OUTLET VELOCITIES IF TAILWATER IS NOT EFFECTIVE IN CAUSING THE CULVERT TO FILL AT THE OUTLET.

5-54

PROJECT: I 85-2

DESIGNER: L. A. H.

DATE: 2-23-64

HYDROLOGIC AND CHANNEL INFORMATION

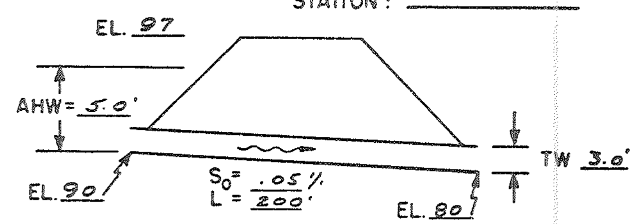
Q_1 = 120 cfs. = Q_{25} TW_1 = 3.0'

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: 314+10



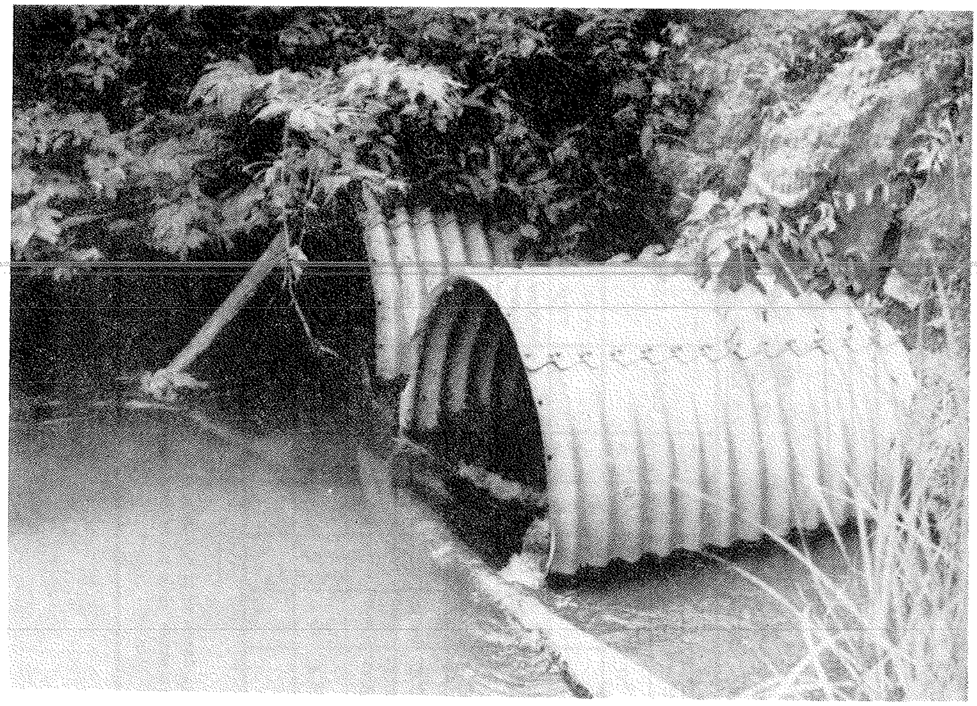
MEAN STREAM VELOCITY = 12'/sec
MAX. STREAM VELOCITY = 15'/sec

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW = H + h_o - LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CMP (Cir.) Mitered	120	Assume 54"	1.25	5.6'								Outlet Control does not Govern.		Not Computed - Take Channel		HW high Try 60"
"	120	60"	.97	4.9	.7	2.5	3.0	4.0	3.0	4.0	10.0		4.9			Need more cover - Try arch
CMP Arch Mitered	120	72" x 44"	1.24	4.6	.7	3.4	2.4	3.0	3.0	3.0	10.0		4.6			Check Box Culvert
Concrete Box 30° W.W.	120	4' x 4'	1.23	4.9	.4	2.0	3.1	3.5	3.0	3.5	10.0		4.9			
Concrete Oval Gr. End Proj.	120	60" x 38"	1.51	4.8	.2	2.9	2.7	2.9	3.0	3.0	10.0		4.8			
Concrete Cir Groove End Proj	120	54"	1.11	5.0	.2	1.7	3.1	3.8	3.0	3.8	10.0		5.0			

SUMMARY & RECOMMENDATIONS:

IN-PLACE COST, AVAILABILITY, LOCATION, COVER REQUIREMENTS, ETC., SHOULD BE CONSIDERED BY THE DESIGNER IN SELECTING CULVERT. CM PIPE ARCH CULVERTS OR CONCRETE OVAL PIPES MIGHT BE A SOLUTION WHERE COVER IS LIMITED.

U.S. GOVERNMENT PRINTING OFFICE : 1977 O-231-382



Debris begins to build up at entrance to a dual 1.80-m culvert—Brazil.

Debris-Control Structures

Hydraulic Engineering Circular No. 9 March 1971 *

Prepared by the Hydraulics Branch, Bridge Division, Office of Engineering, Federal Highway Administration, Washington, D. C. 20591

CONTENTS

* This Circular is a minor revision of the February 1964 Edition.

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

DEBRIS-CONTROL STRUCTURES

Prepared by
G. Reihsen, Highway Engineer, Region 7
and
L. J. Harrison, Highway Engineer
Hydraulics Branch, Bridge Division

PREFACE

The original edition of this Circular was prepared in 1964 in cooperation with the Region 7 and Washington offices of the Federal Highway Administration (formerly Bureau of Public Roads) and the California Division of Highways. This revision incorporates comments received from users and an additional remark on safety.

The Circular is based principally on California practice and experience since the publication of the chapter on debris-control in California Culvert Practice 1/. The authors are indebted to Messrs. Kenneth Fenwick and Walter Whitnack of the California Division of Highways for their cooperation in furnishing standard plans and photographs and in arranging for field inspections. Particular credit is due to the hydraulics and maintenance engineers of California Highway Districts I, II, IV, VII, and VIII for relating their experiences with these structures. Permission to use plans prepared by the highway departments of California, Washington, and Hawaii is acknowledged. Special recognition is given to Mr. J. Kieley of Region 7, Federal Highway Administration, for his review and helpful suggestions in the preparation of the manuscript.

INTRODUCTION

General

Water-borne debris problems and structures used for controlling debris are discussed in this Circular. An accumulation of debris at inlets of highway drainage structures is a frequent cause of unsatisfactory performance or malfunction. This accumulation may result in failure of the drainage structure or overtopping of the roadway by

1/ State of California Division of Highways, California Culvert Practice, Sacramento, California, 2nd Edition, pp. 13-31, 1955.

flood waters and possible damage to the roadway and other property. Consideration of the need for debris-control structures should be an essential part of all hydraulic structure designs. The emphasis in this publication is on culverts because their relatively limited waterway area is subject to clogging by retention of debris at the inlet.

Debris can be controlled by three methods: (a) intercepting the debris at or above the inlet; (b) deflecting the debris for detention near the inlet; or (c) passing the debris through the structure. In some locations, it may be desirable to provide a relief opening either in the culvert itself or by installing a separate, smaller pipe with the inlet higher than the principal culvert inlet. The choice of method depends upon the size, quantity and type of debris, the potential hazard to life and property, the costs involved and maintenance proposed. The debris-control structure selected to meet the needs of the site must be compatible with the need for a forgiving roadside for errant vehicles. Some examples shown herein do not fully meet this criterion but were selected to illustrate the control device only.

Often the waterway opening is arbitrarily increased in an attempt to pass debris through the culvert. The additional cost of such an approach is usually greater than that for a device installed to control debris. On the other hand, when debris from the drainage basin can be passed through the structure without clogging, maintenance costs will be less than when debris is intercepted and subsequently requires removal.

A debris-control structure may have several of the following advantages:

(a) Prevents traffic delays due to an accumulation of drift on the roadway or washouts caused by clogged culverts.

(b) Allows for planned maintenance rather than emergency maintenance during floods when other situations arise which also require immediate attention.

(c) Avoids providing a "safety factor" in sizing a culvert to accommodate debris.

(d) Provides a safeguard against damaging buoyant forces when an accumulation of drift at the culvert entrances causes part-full flow.

(e) Gives maintenance forces a method for correcting drift problems at existing culverts.

In this publication a system of classifying the type of debris expected from any drainage basin is followed by a list of types of debris-control structures. The basis for choosing the type of control structure is given and details of design are discussed.

Classification of Debris

Flood flow reaching a culvert nearly always carries debris which may be either floating material, material heavier than water, or a combination of both. Debris concerns the highway engineer because it can be deposited at the culvert entrance or in the culvert, thus impairing its operation. A thorough study of the extent and type of the debris originating in the drainage basin is essential for proper design of a culvert.

As an aid in selecting an appropriate debris-control structure, the debris from the drainage basin should be classified. A convenient classification system is that of the California Division of Highways which follows:

1. Very Light Floating Debris or No Debris.
2. Light Floating Debris - Small limbs or sticks, orchard prunings, tules and refuse.
3. Medium Floating Debris - Limbs or large sticks.
4. Heavy Floating Debris - Logs or trees.
5. Flowing Debris - Heterogeneous fluid mass of clay, silt, sand, gravel, rock, refuse or sticks.
6. Fine Detritus - Fairly uniform bedload of silt, sand, gravel more or less devoid of floating debris, tending to deposit upon diminution of velocity.
7. Coarse Detritus - Coarse gravel or rock fragments.
8. Boulders - Large boulders and large rock fragments carried as a bedload of flood stage.

146

Types of Debris-Control Structures

Debris-control structures can have many shapes and can be constructed of a variety of materials. These structures will be divided into the following general types:

1. Debris Deflectors - (figs. 1-13) - Structures placed at the culvert inlet to deflect the major portion of the debris away from the culvert entrance. They are normally "V"-shaped in plan with the apex upstream.

2. Debris Racks - (figs. 14-27) - Structures placed across the stream channel to collect the debris before it reaches the culvert entrance. Debris racks are usually vertical and at right angles to the streamflow, but they may be skewed with the flow or inclined with the vertical.

3. Debris Risers - (figs. 28-34) - A closed-type structure placed directly over the culvert inlet to cause deposition of flowing debris and fine detritus before it reaches the culvert inlet. Risers are usually built of metal pipe. Risers are also used as relief devices in the event the entrance becomes plugged with debris (figs. 33, 34, 43, 45, and 51).

4. Debris Cribs - (figs. 35-39) - Open crib-type structures placed vertically over the culvert inlet in log-cabin fashion to prevent inflow of coarse bedload and light floating debris.

5. Debris Fins - (figs. 40-45) - Walls built in the stream channel upstream of the culvert. Their purpose is to aline debris, such as logs, with the axis of the culvert so that the debris will pass through the culvert barrel without clogging the inlet. They are sometimes used on bridge piers to deflect drift.

6. Debris Dams and Basins - (figs. 46-51) - Structures placed across well-defined channels to form basins which impede the streamflow and provide storage space for deposits of detritus and debris.

7. Floating Drift Boom - Logs or timbers which float on the water surface to collect floating drift. Drift booms require guides or stays to hold them in place laterally. They are limited in use and will not be discussed further.

8. Combination Devices - (figs. 28, 29, 43, 45, 48, and 51) - A combination of two or more of the preceding debris-control structures at one site to handle more than one type of debris and to provide additional insurance against a clogged culvert inlet.

148

DESIGN OF DEBRIS-CONTROL STRUCTURES

Preliminary Field Studies

Proper design of a debris-control structure must be preceded by a field study of the debris problem. Among the factors to be considered are possible future changes in the type of debris that might result from new industry or changes in land use within the drainage basin. As an example, logging in a previously virgin area could change the nature of the debris problem from one of "medium floating debris" to "heavy floating debris." Fire also could change the type and quantity of debris reaching culverts making it necessary to take remedial action for debris control.

Culverts located at the end of urban drainage channels are often clogged by refuse dumped into the channel or by trash washed off the city streets. Under such conditions, a rack can usually be installed at low cost to prevent clogging. However, urban locations require careful design since malfunction of the debris-control structure will often cause flooding and damage to adjacent property.

An estimate of the quantity as well as the type of debris is needed by the designer so that an adequate debris storage area can be provided immediately upstream from the control structure. Information on the types and quantities of debris resulting from past floods are an invaluable guide in selecting the type of debris-control structure. Such information could be secured from maintenance personnel, from inhabitants of the immediate area or by personal observation. Access to the debris storage area is needed for periodic removal of debris.

Determining the allowable headwater and the height of embankment above the invert of the culvert at the inlet is also necessary in selecting the type of control structure best suited to the particular problem. Damage that would result from a plugged culvert should be estimated to evaluate the need for a debris-control structure.

To summarize, the field survey data should include:

(1) Classification of the expected debris as to type.

(2) Quantity of expected debris.

(3) Future changes in debris type or quantity due to potential changes in land use.

(4) Information from which the designer can estimate streamflow velocities in the vicinity of the culvert.

(5) Topographic map or cross sections of the area available for storage of debris at the site, accessibility of the storage area for debris removal and the probable frequency of clean-out.

(6) Possible damage that would result from debris clogging the drainage structure.

Selecting Type of Structure

The safety of highway traffic should be an overriding consideration in the selection of the type of debris-control device. The culvert end and the debris-control structure should be located beyond the usual recovery area for errant vehicles or the debris-control structure should be designed to enhance the drivers chance of recovery 2/. At existing sites where modifications cannot be made to meet this objective, an appropriate vehicle restraining device or an impact attenuating device should be provided on the roadside.

In order for a debris-control structure to perform its intended function, the type of debris must be anticipated and the appropriate device selected to prevent the culvert entrance from clogging. Table 1, based on experience with different types of structures, provides a guide for selecting control structures for various debris classifications. Suitable devices for each debris classification are shown by "X". When the expected debris is not all of one classification, the table also provides guidance for selecting a combination of control devices.

2/ Highway Research Board, Traffic-Safe and Hydraulically Efficient Drainage Practice, NCHRP Synthesis of Highway Practice No. 3, Washington, D. C., 38p., 1969.

TABLE 1 - Guide for selecting type of structures suitable for various debris classifications

Debris Classification \ Type of Structure	Deflector	Rack	Riser	Crib	Fin	Dam and Basin	Boom
Light Floating Debris		X		X			X
Medium Floating Debris	X	X					X
Heavy Floating Debris	X				X		
Flowing Debris			X			X	
Fine Detritus			X			X	
Coarse Detritus			X	X		X	
Boulders	X						

Debris Deflectors

The function of a debris deflector (figs. 1-13) is to divert medium and heavy floating debris and large rocks from the culvert inlet for accumulation in a storage area where it can be removed after the flood subsides. The storage area provided must be adequate to retain the anticipated type and quantity of debris expected to be accumulated during any one storm or between cleanouts. The deflector should be built at the culvert entrance and aligned with the stream rather than the culvert so that accumulated debris will not tend to block the channel.

Single deflectors can be built over batteries of pipe culverts (fig. 6) or individual deflectors can be built over each pipe of a battery (fig. 11). Their structural stability and orientation with the flow make deflectors particularly suitable for large culverts, high velocity flow, and with debris such as heavy logs, stumps, or large boulders.

Plates I and II show general dimensional details of debris deflectors. The angle at the apex of the deflector should be between 15^{o} and 25^{o}, and the total area of the two sides of the deflector should be at least 10 times the cross-sectional area of the culvert. Spacing between vertical members should not be greater than the minimum culvert dimension nor less than 1/2 the minimum dimension. A spacing of 2/3 the minimum dimension is commonly used. The base width and height of the deflector should be at least 1.1 times the respective dimensions of the culvert. Where headwater from the design flood is expected to be above the top elevation of the deflector and floating debris is anticipated, horizontal members should be placed across the top. The spacing of horizontal members on the top should be no greater than 1/2 the smallest dimension of the culvert opening. The upstream member is vertical on most existing installations. However, a sloping member at the apex (sloping downstream from bottom of member) would reduce the impact of heavy floating debris and boulders, and probably prevent debris from gathering at that point. Deflectors with a sloping member at the apex are highly recommended by maintenance personnel.

Debris deflectors are usually built of heavy rail or steel sections (figs. 1-11), although timber (figs. 12, 13) and steel pipe are sometimes used for light debris. For economy salvaged railroad rails may be used if available. Figure 10 and Plate II show a deflector that uses a cable as its lower longitudinal member. This modification has proved superior in locations where heavy boulders damage rigid members. 151
Wire and post debris deflectors (fig. 9) have been used for light floating debris.

Debris Racks

A debris rack (figs. 14-27) is essentially a barrier across the stream channel which stops debris that is too large to pass through the culvert. Debris racks vary greatly in size and in the material used in their construction. Height of racks should allow some freeboard above the expected depth of flow in the upstream channel for the design flood. Racks 10 to 20 feet high have been constructed. The rack may be vertical or inclined and may be placed over the culvert inlet (figs. 14, 15, 19, 22, 23, 24, 26, 27, and 29) or upstream from the culvert (figs. 16, 18, 21, and 25). Figure 20 shows a rack protecting the inlet of a down drain. Racks should not be placed in the plane of the culvert entrance, since they induce plugging when thus positioned. Access to the rack is necessary for maintenance.

The rack should be placed well upstream from the culvert entrance in those locations where a well-defined channel exists. However, they should not be placed so far upstream that debris enters the channel between the rack and the culvert inlet. If a large debris storage area exists at the rack location, the frequency of maintenance is reduced and added safety is provided against overtopping the installation during a single storm. Some racks have not required maintenance for several years.

Plates III through VI, inclusive, show the general dimensional details of debris racks. The total straining area of a rack should be at least ten times the cross-sectional area of the culvert being protected. Vertical bars are generally spaced from 1/2 to 2/3 the minimum culvert dimension. This spacing permits the lighter debris to pass through the rack and the culvert. In urban areas, (fig. 19) bar spacing of racks should be a maximum of 6 inches and tied to the culvert headwall by top bars to prevent entrance of children. Under these conditions it is preferable to hold the lowest edge of rack about six inches above the flow line of the ditch, permitting some debris to pass under the rack during low flows. The close spacing of the bars creates a debris trap and increases the maintenance required.

Generally, racks do not have top or horizontal members extending from the rack to the culvert headwall although there are exceptions (fig. 15). The overall dimensions of the rack should be a function of the amount of debris expected per storm, the frequency of storms, and the schedule of expected cleanouts. When a rack is installed at the upstream end of the wingwalls, it should be at least as high as the culvert parapet.

Since vertical racks receive the full impact of floating debris and boulders, their structural design should incorporate brace members set in concrete. Inclined racks and rubber tires (fig. 17) have been used to help reduce the impact of heavy debris striking at high velocity.

Chain-link fence has been used for removal of light debris where stream velocities are low. The fence barrier has a particular advantage in tidal areas where the functioning of flap or check gates is hampered by light debris gathering on gate seats and thereby blocking complete closure of the gates.

Debris Risers

Debris risers (figs. 28-34) generally consist of a vertical culvert pipe and are usually suitable for culvert installations of less than 54-inch diameter. This type of debris-control structure is used where considerable height of embankment is available and where debris consists of flowing masses of clay, silt, sand, sticks, or medium floating debris without boulders. Risers are seldom structurally stable under high-velocity flow conditions because of their vulnerability to damage by impact.

Risers placed above the streambed at the bottom of steep, narrow draws cause ponding with a reduction in velocity and deposition of sediment. The resulting flat-bottom basin gives maintenance personnel a place to work when either culvert cleanout or debris removal is necessary. This basin also produces deposition of heavier debris upstream at the entrance to the basin where the debris cannot clog the drainage structure. To avoid vibration of the riser pipe and unstable flow conditions, the riser diameter should be about 1 foot larger than the culvert diameter.

Plates VII through X, inclusive, show the general dimensional details of debris risers. The riser should be covered by a grate or cage to prevent clogging of the culvert. The grate bars can be reinforcing steel or other such material with vertical spacing not greater than 1/2 the diameter of the culvert. Slots or holes are placed in the sides of the riser to carry low flow (fig. 32). It is preferable to have these holes punched before galvanizing to avoid deterioration by rust. The holes are considered to have no hydraulic capacity under peak flow conditions because of the likelihood of their becoming plugged by light floating debris and silt. It is good practice to build riser pipes at least 36 inches in diameter to provide an area large enough for maintenance access. It is also desirable to connect the grate bars to a coupling band, rather than directly to the riser pipe, so the grate can be removed should cleaning be required. If the embankment is of sufficient height, provisions should be made to extend the riser vertically if necessary. This can be accomplished by means of standard coupling bands in the case of corrugated metal pipe risers.

Installations have been built with the riser pipe at an angle between vertical and the stream grade (fig. 28). This reduces the impact of debris at the elbow and assists in moving debris through the culvert. A corrugated metal pipe reducing elbow can be used to connect risers to the culvert inlet, although damage to the metal elbow from falling rocks may occur. Occasionally, concrete is placed inside the elbow to prevent the metal from wearing through by this abrasive action. A solution for extremely severe conditions is to connect riser and culvert by a concrete junction box having the inside shaped as an elbow. A corrugated metal pipe riser usually costs less than a debris crib because of the labor involved in construction of the latter. Risers may be used as relief structures, either independent of the main culvert or in conjunction with it (figs. 33, 34, 43, 45, and 51).

Debris Cribs

A debris crib (figs. 35-39), often called a "bear trap," is particularly adapted to small-size culverts where a sharp change in stream grade or constriction of the channel causes deposition of detritus at the culvert inlet. The crib is usually placed directly over the culvert inlet and is generally built up in log-cabin fashion although other designs are sometimes used.

Plate XI shows the general dimensional details of a debris crib. Spacing between bars should be about 6 inches. A crib may be open (figs. 36-38) or covered (figs. 35, 39) with horizontal top members spaced equal to the crib members. Debris can almost envelop a crib without completely blocking the flow and plugging the culvert. When an open crib is used as a riser and an accumulation of detritus is expected to build up, provision can be made for increasing the heights as needed (figs. 36, 37). Cribs and risers are somewhat similar, but cribs are more appropriate than risers where the culvert has little cover and the detritus is coarse. Cribs have been built as high as 50 feet above a pipe invert with little change in the efficiency of the facility. Due to the debris type and site conditions associated with debris risers and cribs, field inspections of all types of existing debris-control structures have shown these two types to be most consistently successful in producing an efficient, maintenance-free installation.

Debris Fins

The debris fin is a thin wall of concrete, steel, or timber installed parallel with the flow (figs. 40-45). They have been used successfully with large culverts where the debris consists mostly of floating material that would pass through the culvert if oriented parallel with the culvert barrel. Material that is not aligned by the fin to pass through the culvert is retained at the front of the fin for later removal by maintenance personnel. If the fin is sloped upward toward the culvert, debris that does not pass through the culvert will be floated upward and prevented from blocking the culvert inlet. At bridge piers, long debris will generally ride up on the fin and fall off in an aligned position. Fins have also been successful in reducing ice clogging by displacing ice sheets upward along the sloping top surface.

Fins on culverts are usually concrete and located on the centerline of a single culvert (figs. 43-45) or as extensions of the interior walls of multiple box culverts (figs. 41, 42). The upstream end of the fin should be rounded and sloped upward toward the culvert, as shown in figures 40 and 41, to reduce impact, turbulence, and the probability of gathering debris, rather than vertical as shown in figures 42-45.

A debris fin is usually constructed to the height of the culvert; hence, its effectiveness is limited after the inlet becomes submerged. Based on experience, a fin length of 1½ to 2 times the culvert height is recommended. The leading edge would thus have a slope of from $1\frac{1}{2}$:1 to 2:1. Wall thickness should be the minimum needed to satisfy structural requirements in order to minimize disturbance to flow. Fins are generally not used on culverts with a minimum dimension of less than 4 feet.

Since depth of scour at bridge piers is proportional to the width of pier projected normal to the direction of flow, buildup of debris on piers often contributes to bridge failure by scour. Debris fins have been successfully used to align debris with the waterway opening and to avoid the accumulation of debris on bridge piers. When used for this purpose, however, fins should be carefully aligned with flow in order to avoid increasing the projected pier width and a corresponding greater depth of scour.

When used at bridge piers, debris fins are usually constructed of steel or treated timber piling and bracing. Pile penetration should be sufficient to withstand predicted scour depths.

Debris Dams and Basins

On streams carrying heavy sediment and debris loads it is often economically impracticable to provide a culvert large enough to carry surges of debris. If the height of embankment and storage area at the highway are not sufficient for a riser or crib, a debris dam and settling basin placed some distance upstream from the culvert might be feasible. These are sometimes used to trap heavy boulders or coarse gravel that would clog culverts, especially on low fills. In some locations debris dams have been built to provide the added benefit of ground water recharge resulting from ponded water.

Debris dams (figs. 46-51) can be built of precast concrete beams placed in crisscross or log-cabin fashion with rock dumped between the members (fig. 50). Other dams have been built of rock held in place by wire (figs. 47, 49).

The extent of preliminary investigation required for the design of a dam should be commensurate with the size and cost of the structure and the hazard created by failure of the culvert to carry the flow. Information is needed concerning watertightness of the reservoir, suitability of the foundations for supporting the dam, and the availability of construction materials.

Earth or rock fill dams are usually desirable. A spillway should be constructed as a channel outside the limits of the dam. A number of debris dams were built in Southern California and were found to have lower construction costs than the annual cost of removing the debris that otherwise would have been deposited adjacent to and within drainage structures.

Combined Debris Controls

Each drainage basin presents its own debris problem. Often more than one problem exists and two or more types of debris-control structures must be used. At some locations it may be preferable to remove the larger debris at a location upstream from the culvert and to remove the smaller material nearer the culvert inlet. At other locations it may be advisable to install two types of devices so that one will function if the other fails. For example, figure 33 shows a debris riser installed over the entrance of a culvert to provide the water access to the culvert in the event the culvert entrance becomes plugged. Figure 34 shows the same installation after a flood.

Figures 43 and 45 show a culvert protected by both a debris fin and a debris riser. Figure 51 shows an installation consisting of a debris dam and settling basin with a debris deflector at the inlet and a debris riser.

MAINTENANCE

156 The standard or frequency of maintenance must be considered in the design of a debris-control structure. Structures located on a primary highway may have a higher frequency of maintenance than those on a secondary highway. If a low standard of maintenance is to be provided, it may be desirable to use a different type debris-control structure requiring less attention or choose a larger culvert. This consideration may also determine the choice when two or more alternatives are available.

Provisions must be made for maintenance access to the debris-control structure site. A means of access is often difficult to provide, particularly where a high embankment exists. However, such installations usually require less maintenance because of the added debris storage available. If haul roads to debris-control installations are not practical, it may be necessary to provide an area where mechanical equipment such as a crane could be located for removing debris without disrupting highway traffic. Some debris barriers must be cleaned after each major storm.

Maintenance problems may require modifications in control device design. For example, positive debris control could become essential for an extremely long culvert necessitating reduction in the size of openings in the debris-control structure to remove all debris that might clog the culvert.

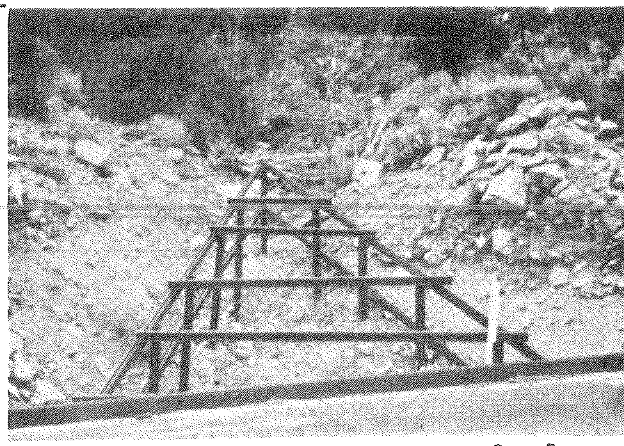


Figure 1. Steel rail debris deflector for large rock.

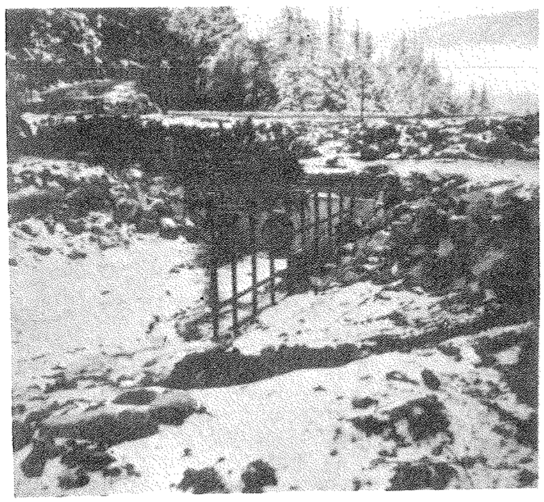


Figure 2. Steel rail debris deflector.

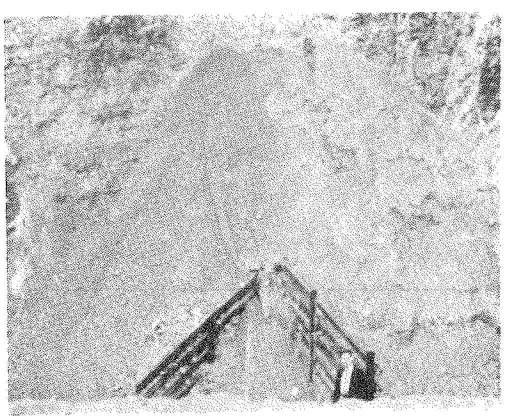


Figure 3. Steel rail debris deflector for fine detritus.

Figure 4. Steel rail debris deflector in area of heavy flowing debris.

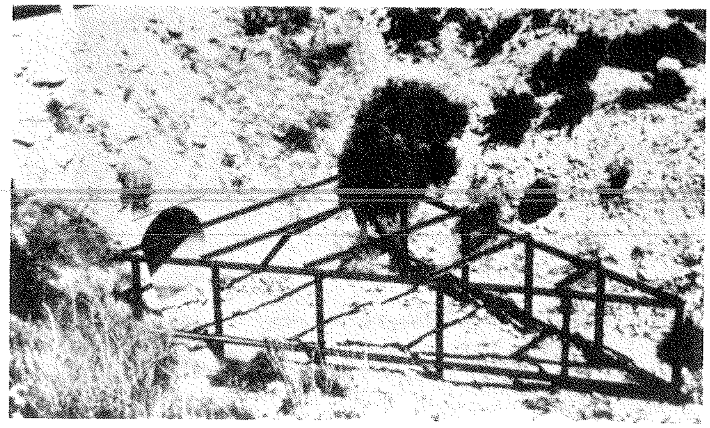


Figure 5. Steel rail debris deflector.

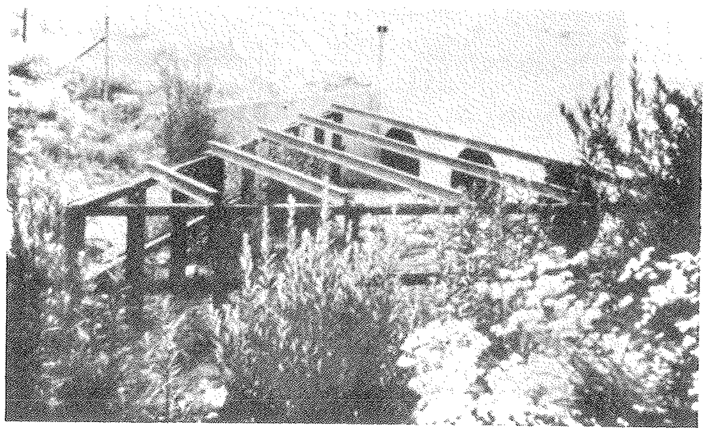


Figure 6. Steel rail debris deflector for battery of culverts (See Fig. 7).

158

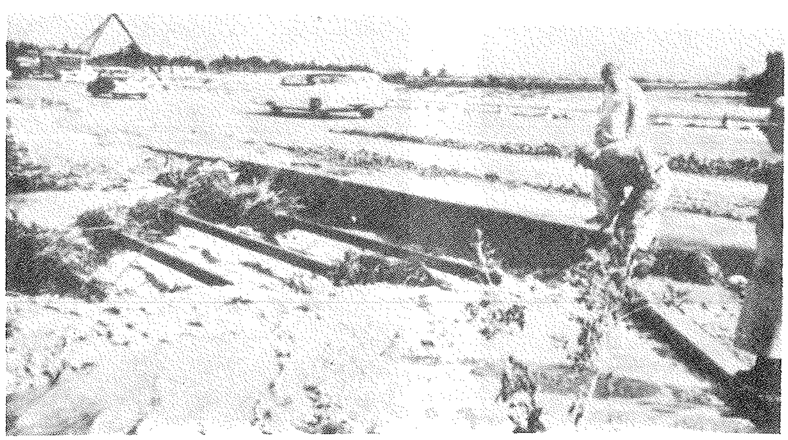


Figure 7. Installation shown in Figure 6 during flood; function well under heavy debris flow.

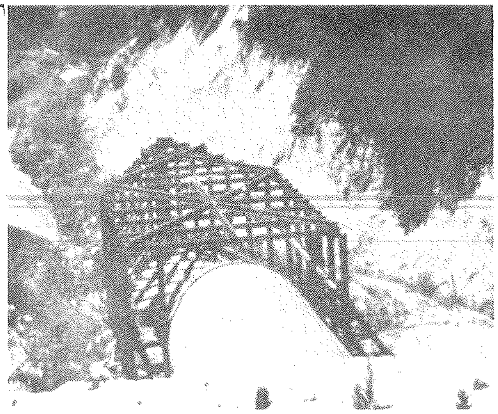


Figure 8. Steel rail debris deflector. Note storage area for debris resulting from culvert projection.

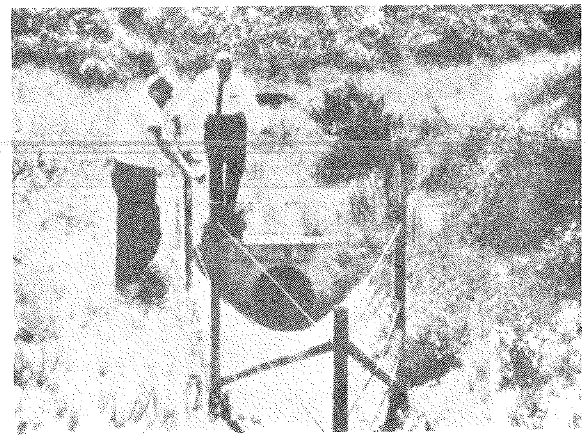


Figure 9. Wire and post debris deflector for light floating debris.

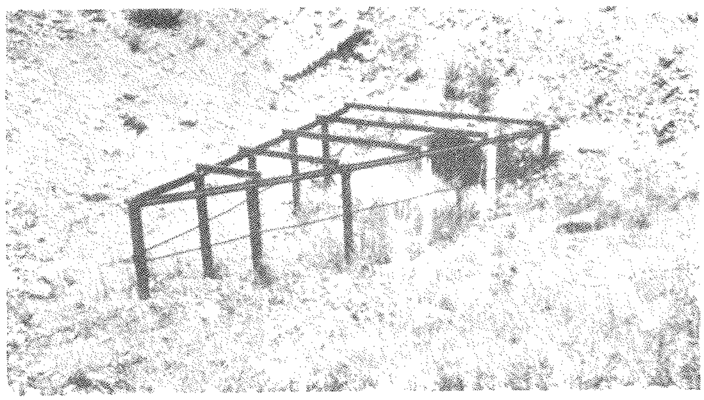


Figure 10. Steel rail and cable debris deflector. Cable's flexibility more desirable than rail's rigidity in boulder areas.

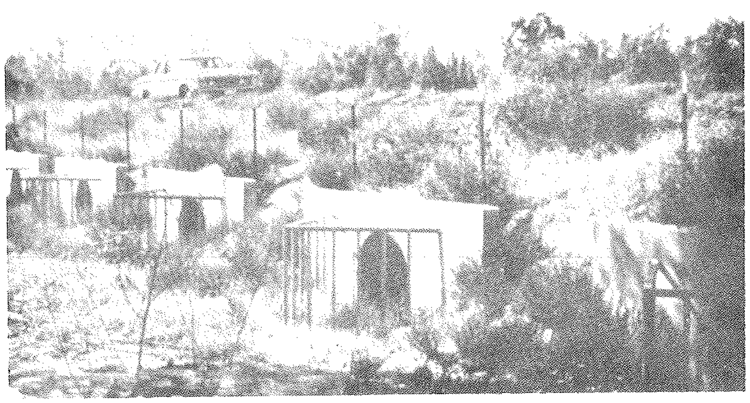


Figure 11. Steel debris deflectors installed at entrances to a battery of culverts.

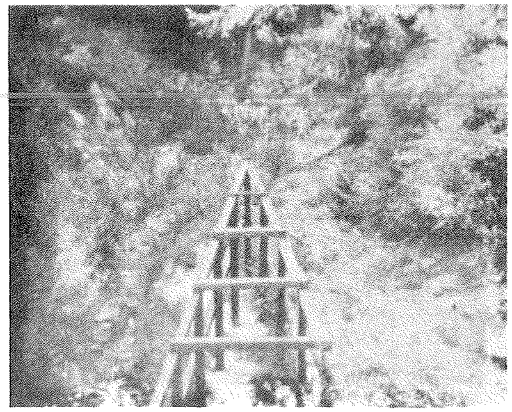


Figure 12. Timber pile debris deflector for boulders and heavy floating debris.

Figure 13. Timber pile debris deflector protected culvert during heavy floods. Nearby culverts without deflectors were plugged.

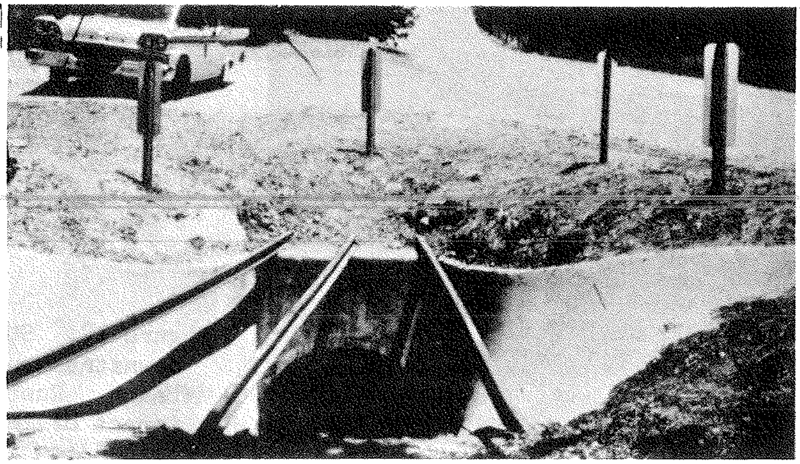


Figure 14. Rail debris rack over sloping inlet. Heavy debris and boulders ride over rack and leave flow to culvert unimpeded.

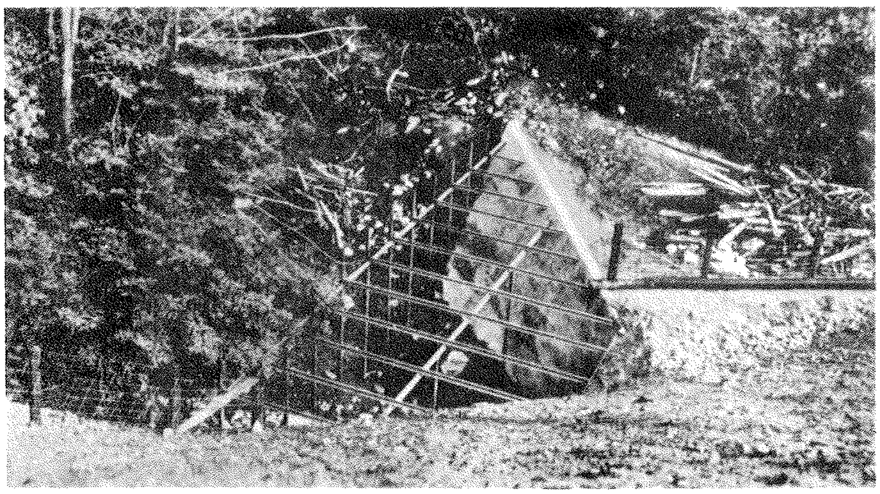


Figure 15. Rail debris rack with top members in area of logging operations.

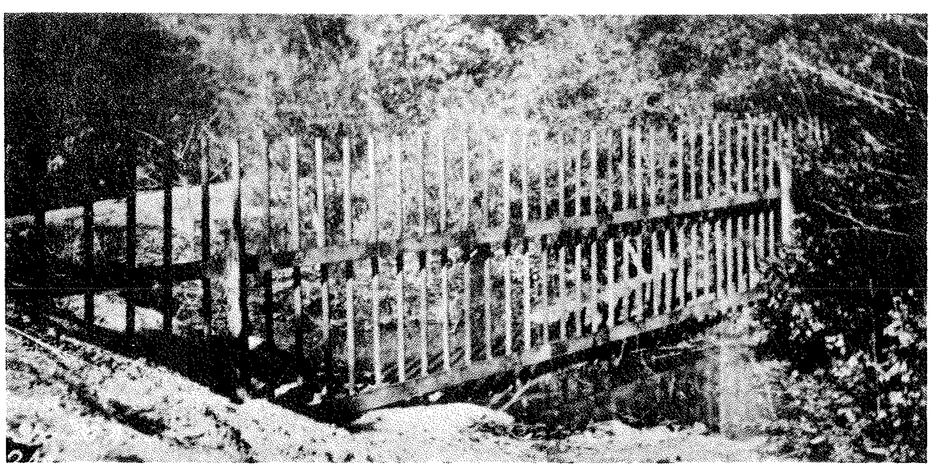


Figure 16. Post and rail debris rack, in place for 35 years, for light to medium floating debris installed 100' upstream of culvert.

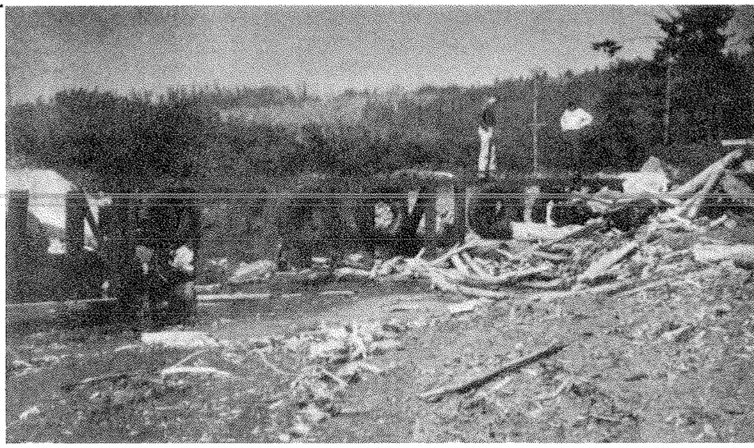
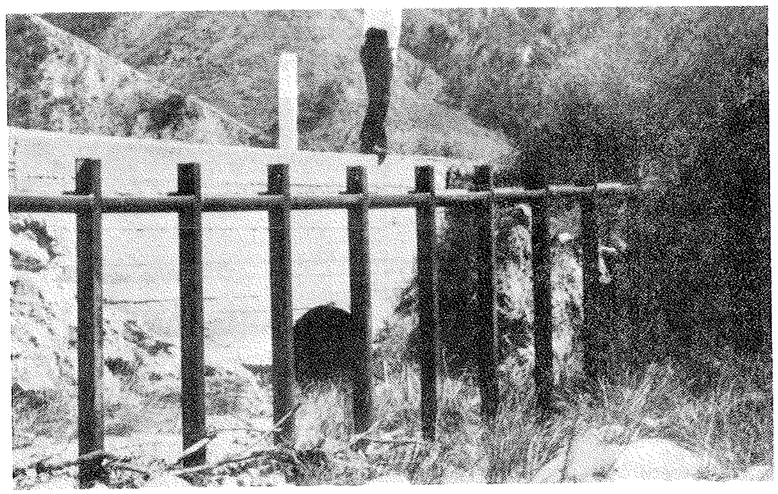


Figure 17. Steel debris rack downstream of culvert on beach. Rubber tires reduce impact of logs.

Figure 18. Rail debris rack. Note large straining area provided.



162

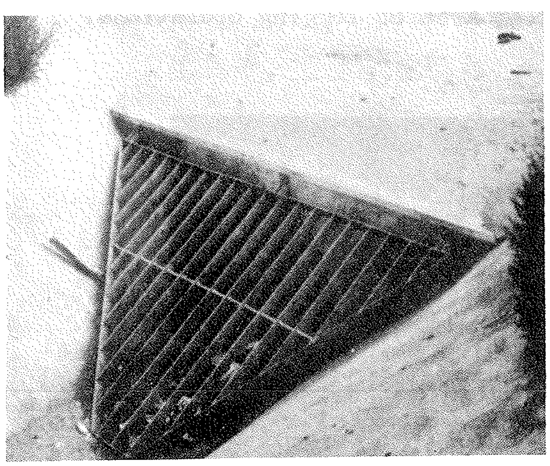


Figure 19. Hinged steel debris rack in urban area. Due to nature of debris and possible entry by children, bar spacing is close.

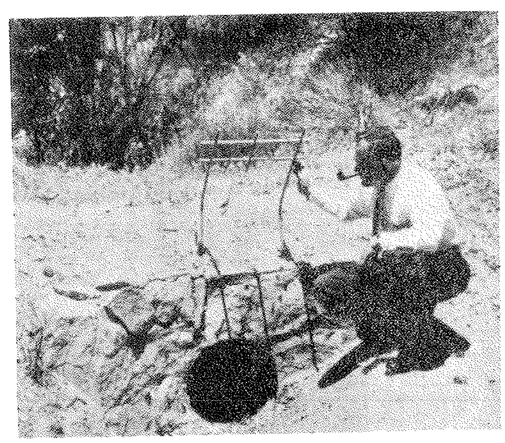


Figure 20. Debris control hinged installation of reinforcing steel at inlet to roadside downdrain.

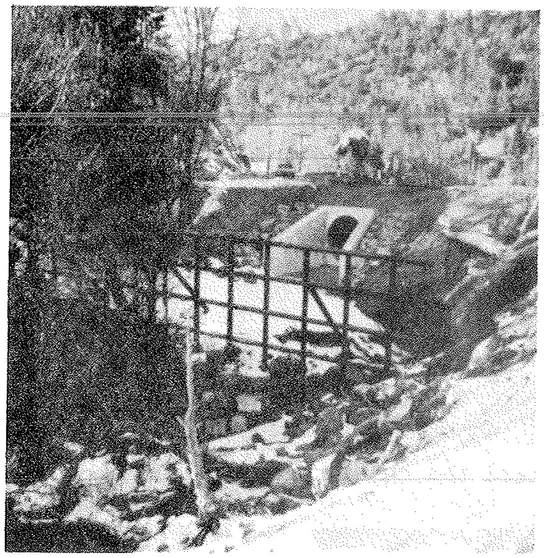


Figure 21. Steel debris rack.

Figure 22. Debris rack used in State of Washington. (See Plate III for design dimensions.)

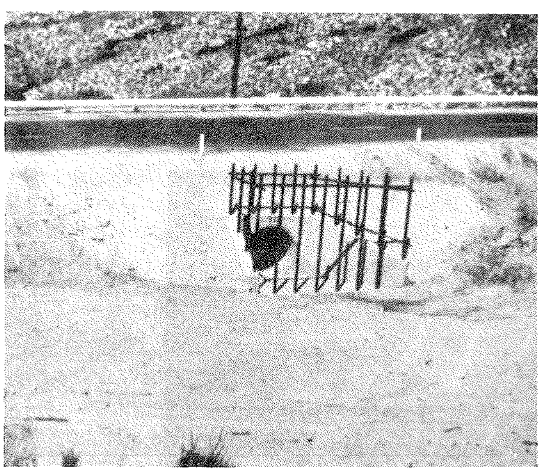


Figure 23. Rail debris rack in arid region. (See Fig. 24.)

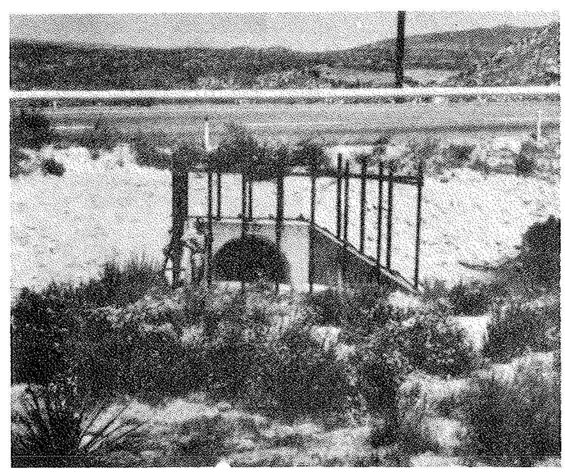


Figure 24. Installation shown in Fig. 23 after several years of fine silt deposition at entrance.

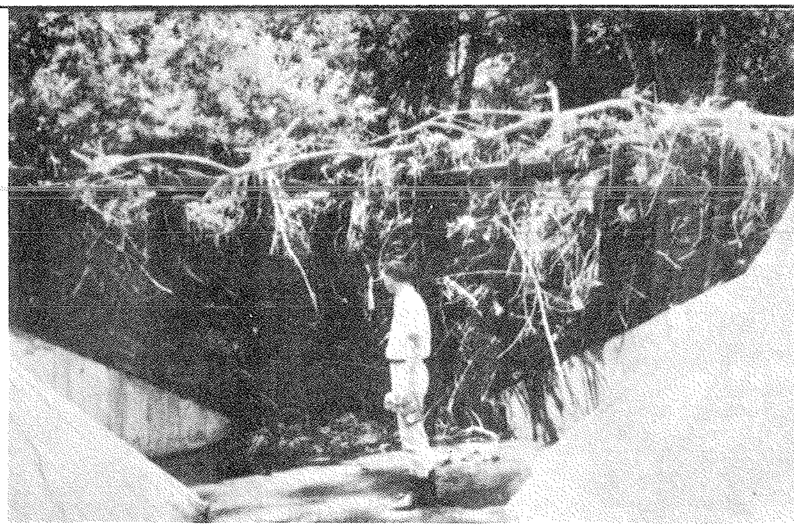


Figure 25. Steel debris rack probably saved the culvert from plugging.

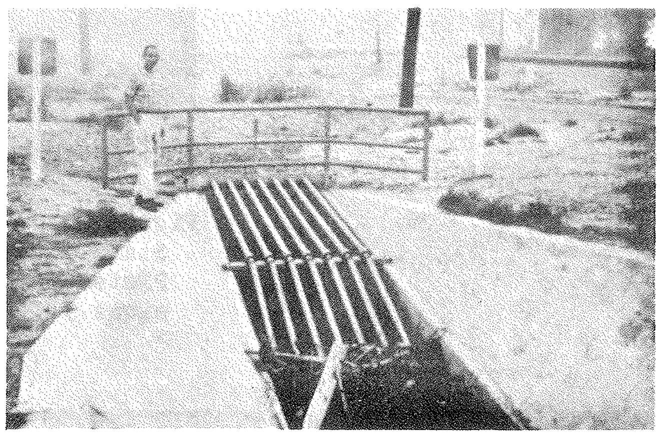


Figure 26. Pipe grill debris rack. Vertical fence at downstream end to prevent debris from spreading over ponding area.

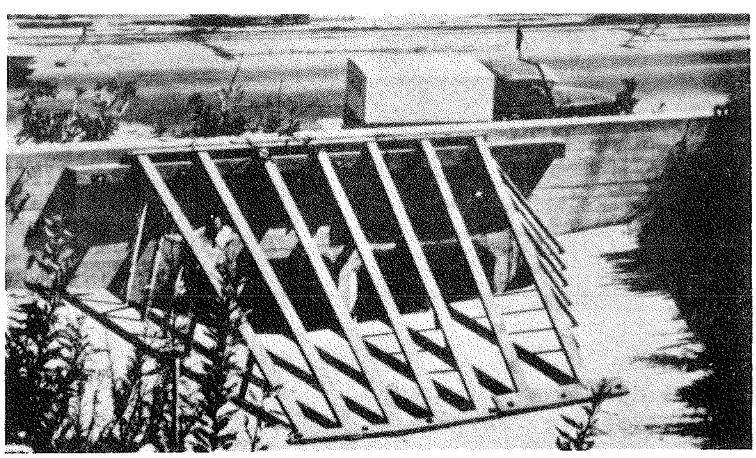


Figure 27. Steel grill debris rack with provision for cleanout afforded by concrete paved area in foreground.

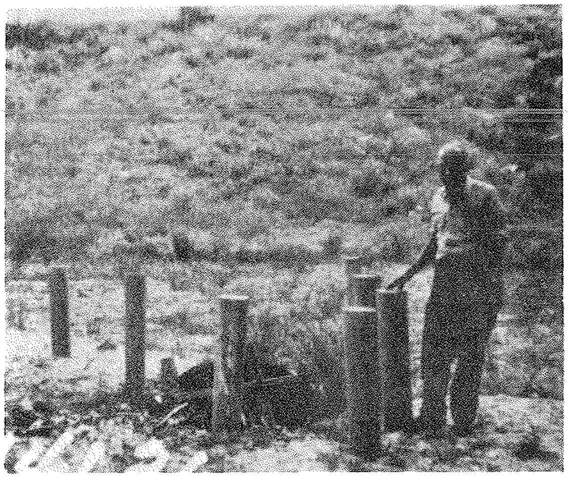


Figure 28. Metal pipe debris riser, with posts to deflect boulders, installed by maintenance forces on 45° angle to vertical.

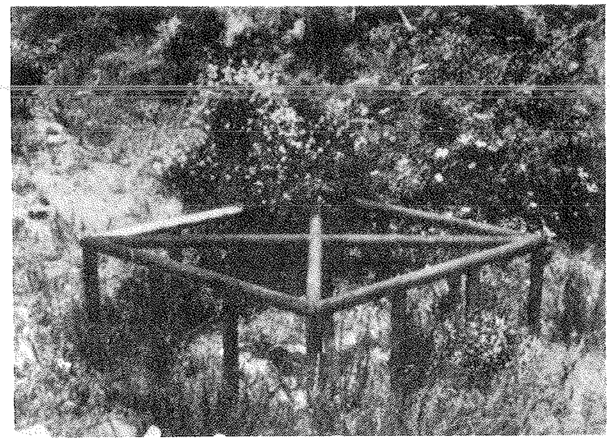


Figure 29. Post debris rack placed over entrance to metal pipe debris riser after latter had caused deposition.

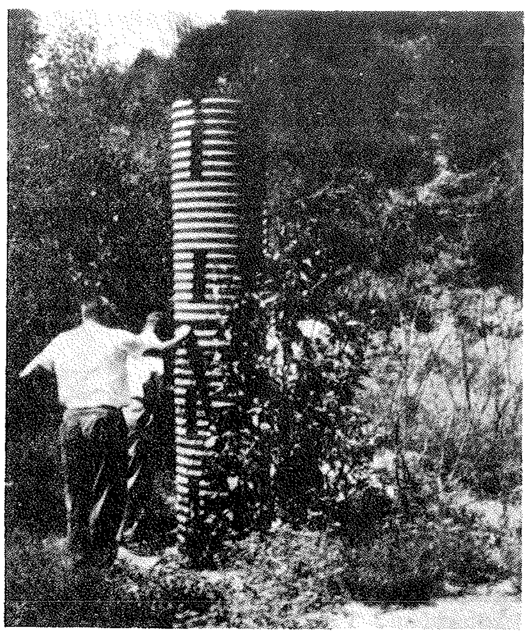


Figure 30. Metal pipe debris riser required little maintenance. Basin had built up 10'.

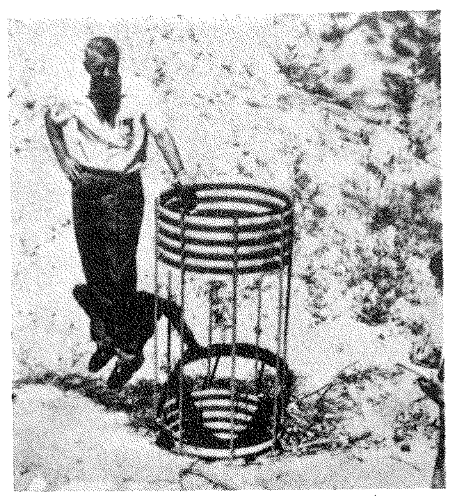


Figure 31. Metal pipe debris riser, in place for 25 years, operated well without vertical extension.

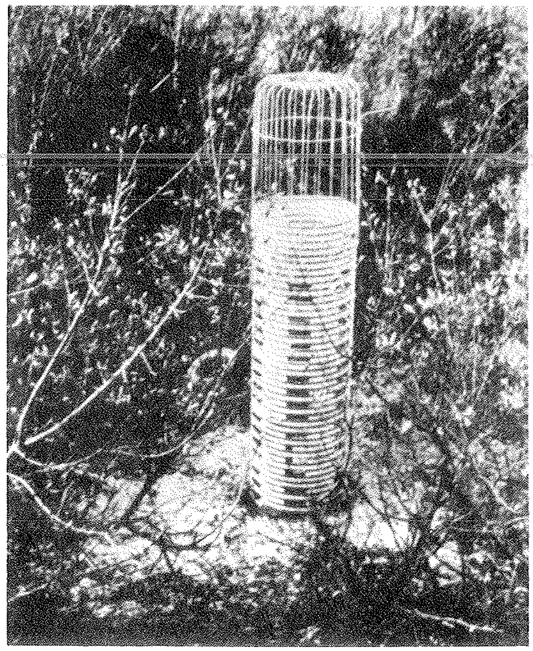


Figure 32. Metal pipe debris riser shows slots for low flows.

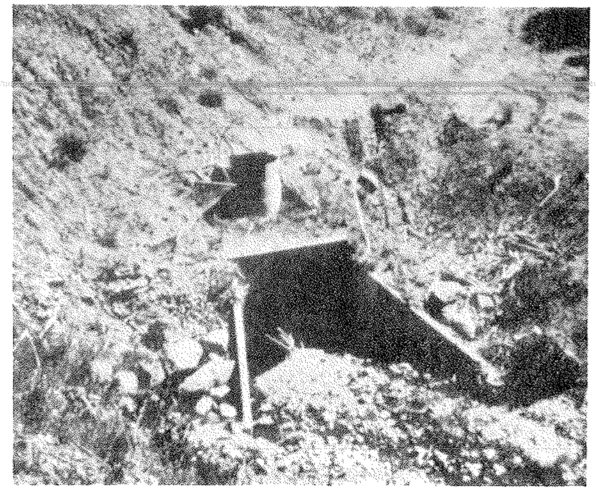


Figure 33. Metal pipe debris riser placed during initial construction of culvert provides relief in case the latter becomes plugged. (See Fig. 34.)

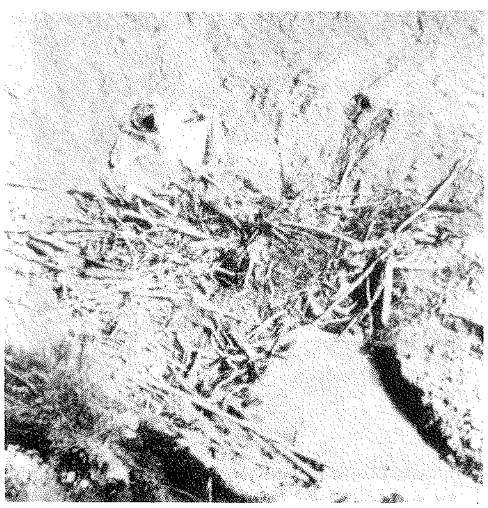


Figure 34. Installation shown in Fig. 33 after flood. Riser carried heavy flow during flood. Fence partially surrounding riser of no value for debris control. (Note man at center of photograph.)

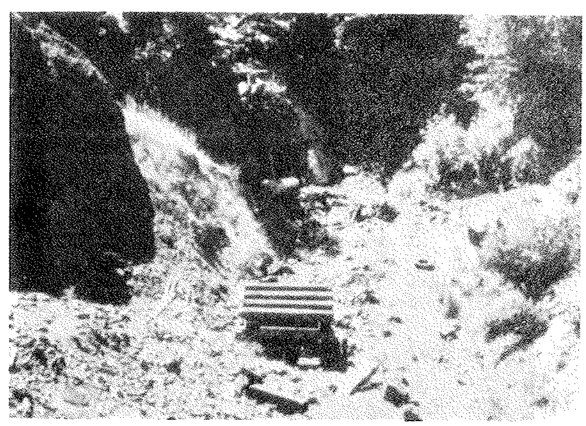


Figure 35. Timber debris crib in ideal location, i.e., high roadway embankment and large settling basin.

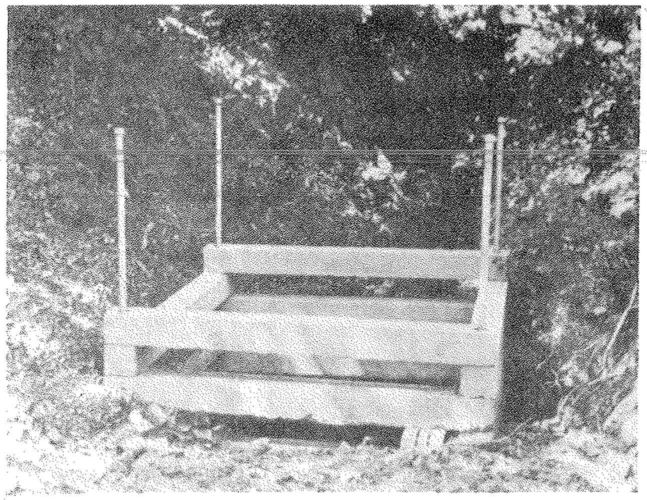


Figure 36. Debris crib of precast concrete sections and metal dowels. Height increased by extending dowels and adding more sections.

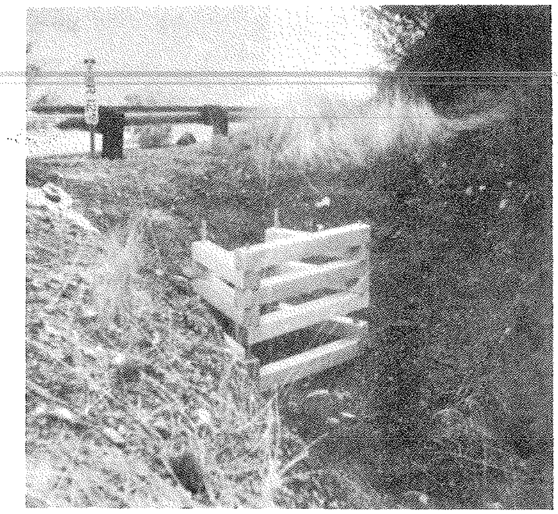


Figure 37. Debris crib of precast concrete sections and metal dowels.

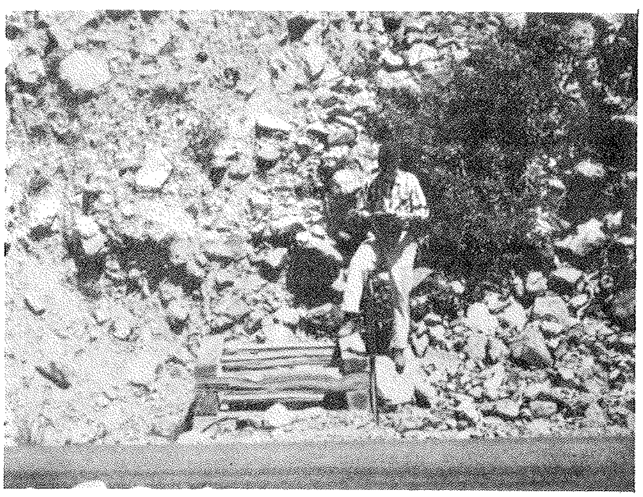


Figure 38. Timber debris crib of inexpensive local materials.

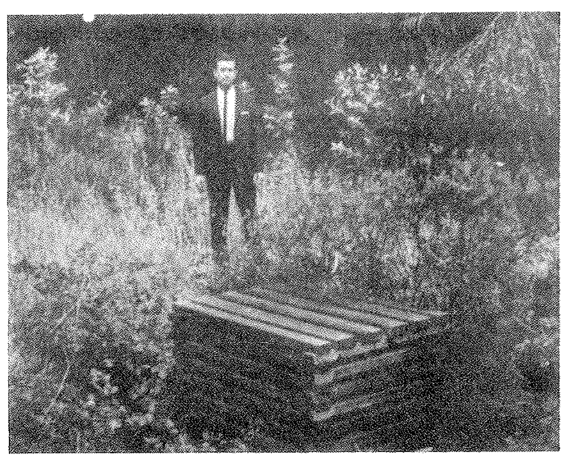


Figure 39. Redwood debris crib with spacing to prevent passage of fine material. Basin had built up 30'.

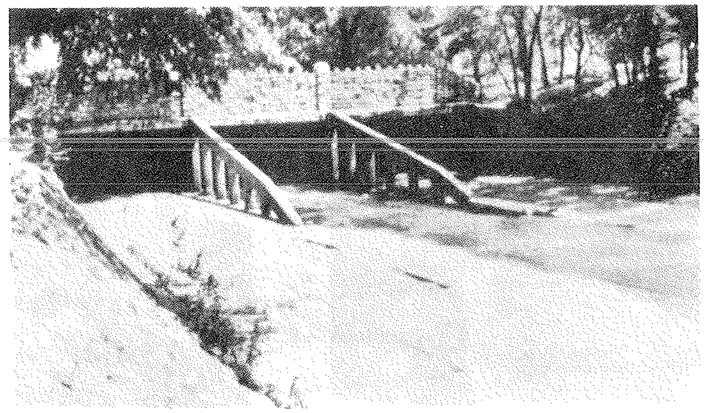


Figure 40. Concrete debris fins with sloping leading edges as extensions of culvert walls.

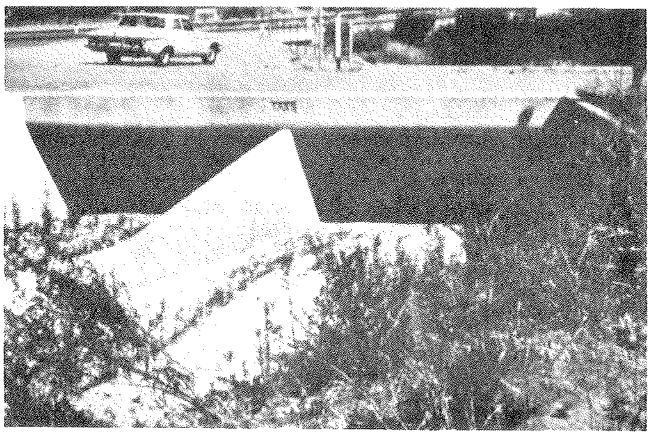


Figure 41. Concrete debris fin with sloping leading edge as extension of center wall.

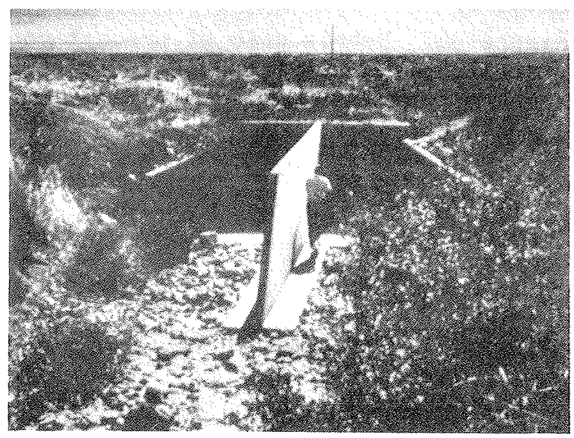


Figure 42. Concrete debris fin with rounded vertical leading edge as extension of culvert center wall.

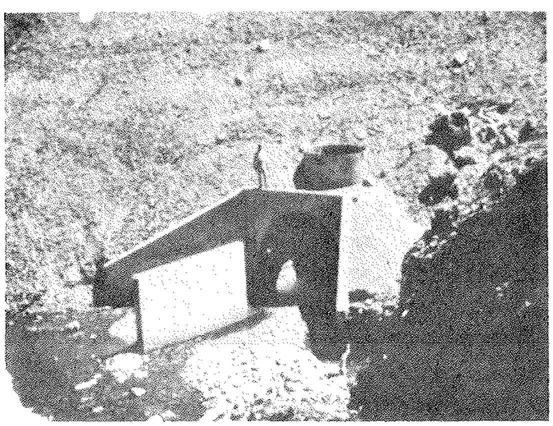


Figure 43. Concrete debris fin and metal pipe debris riser in conjunction with single corrugated metal pipe culvert.

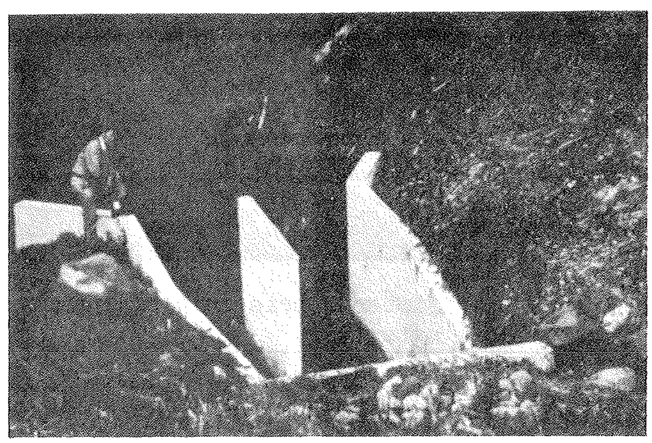


Figure 44. Concrete debris fin for single culvert. Preferable if more area existed between wingwalls and fin.

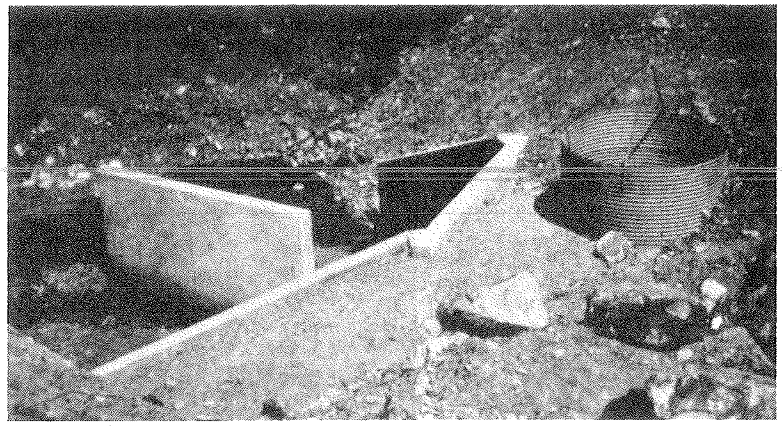


Figure 45. Debris fin and metal pipe debris riser in conjunction with single barrel culvert.

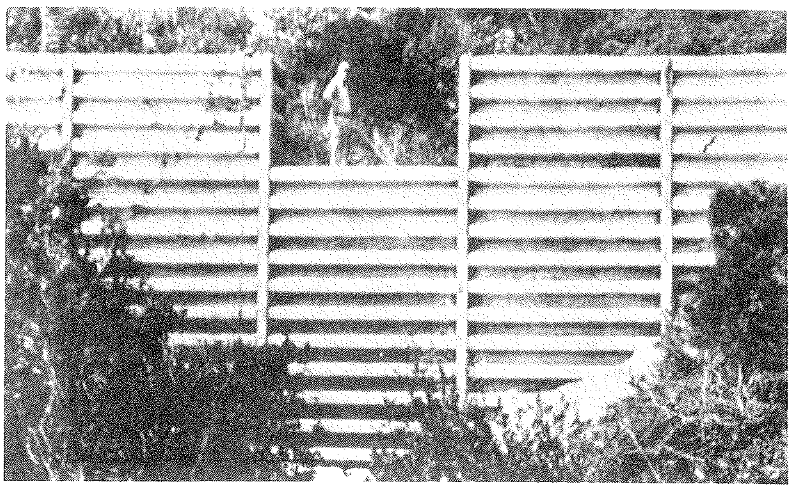


Figure 46. Metal bin type debris dam.

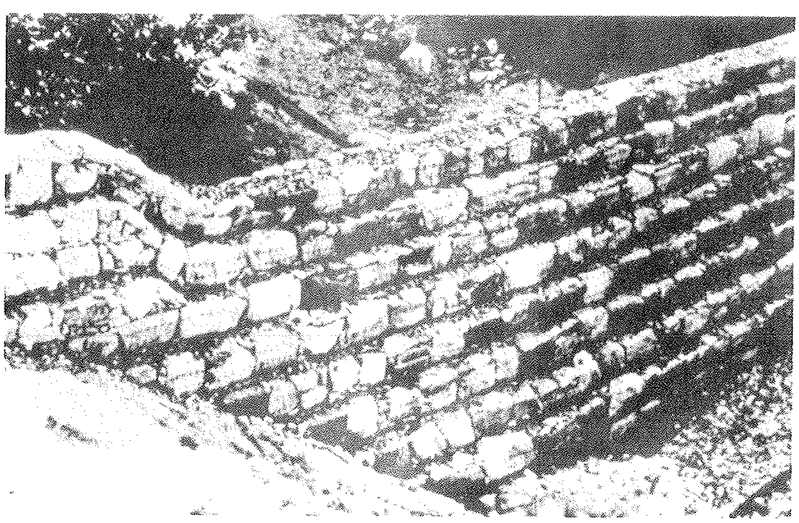


Figure 47. Debris dam of rock and wire.

Figure 48. Debris dam and basin in foreground and steel grill debris rack at culvert entrance in background. (See Fig. 49).

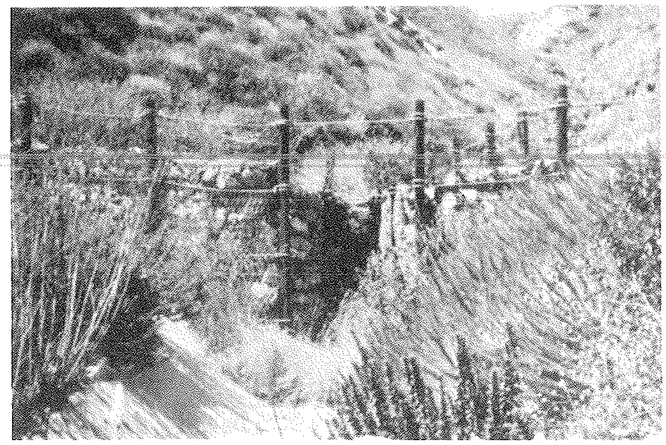


Figure 49. Debris dam of rock and wire shown in Fig. 48.

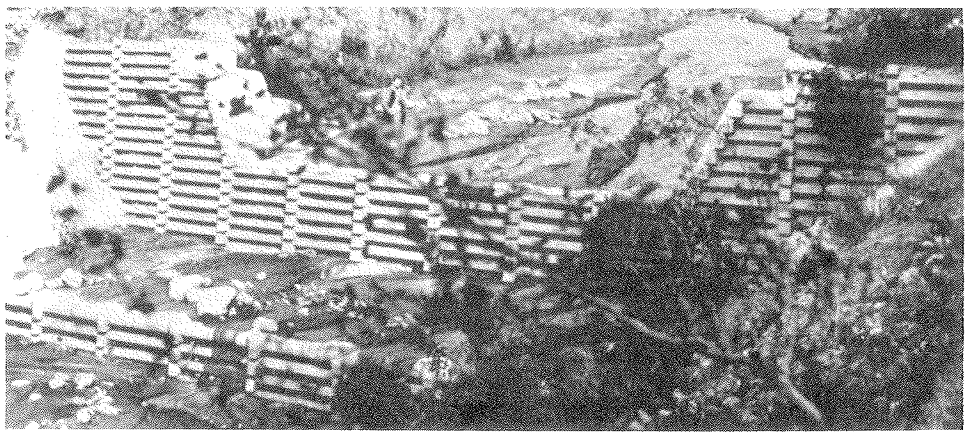


Figure 50. Debris dam of precast concrete sections fabricated to enable placement in interlocking fashion.

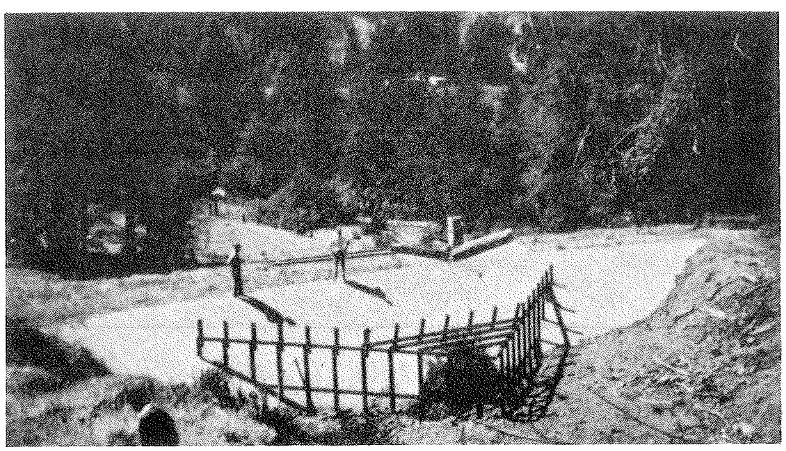
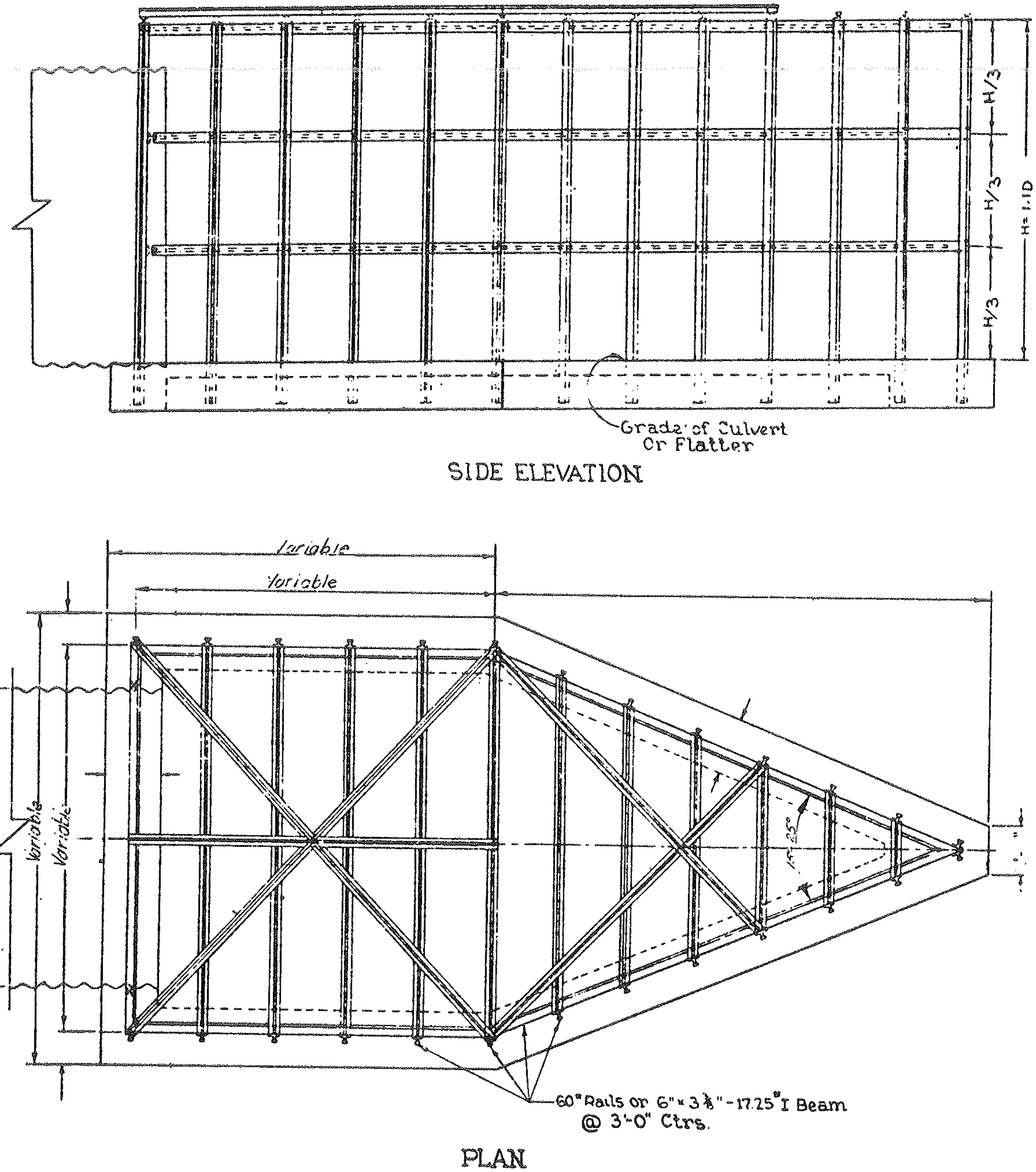
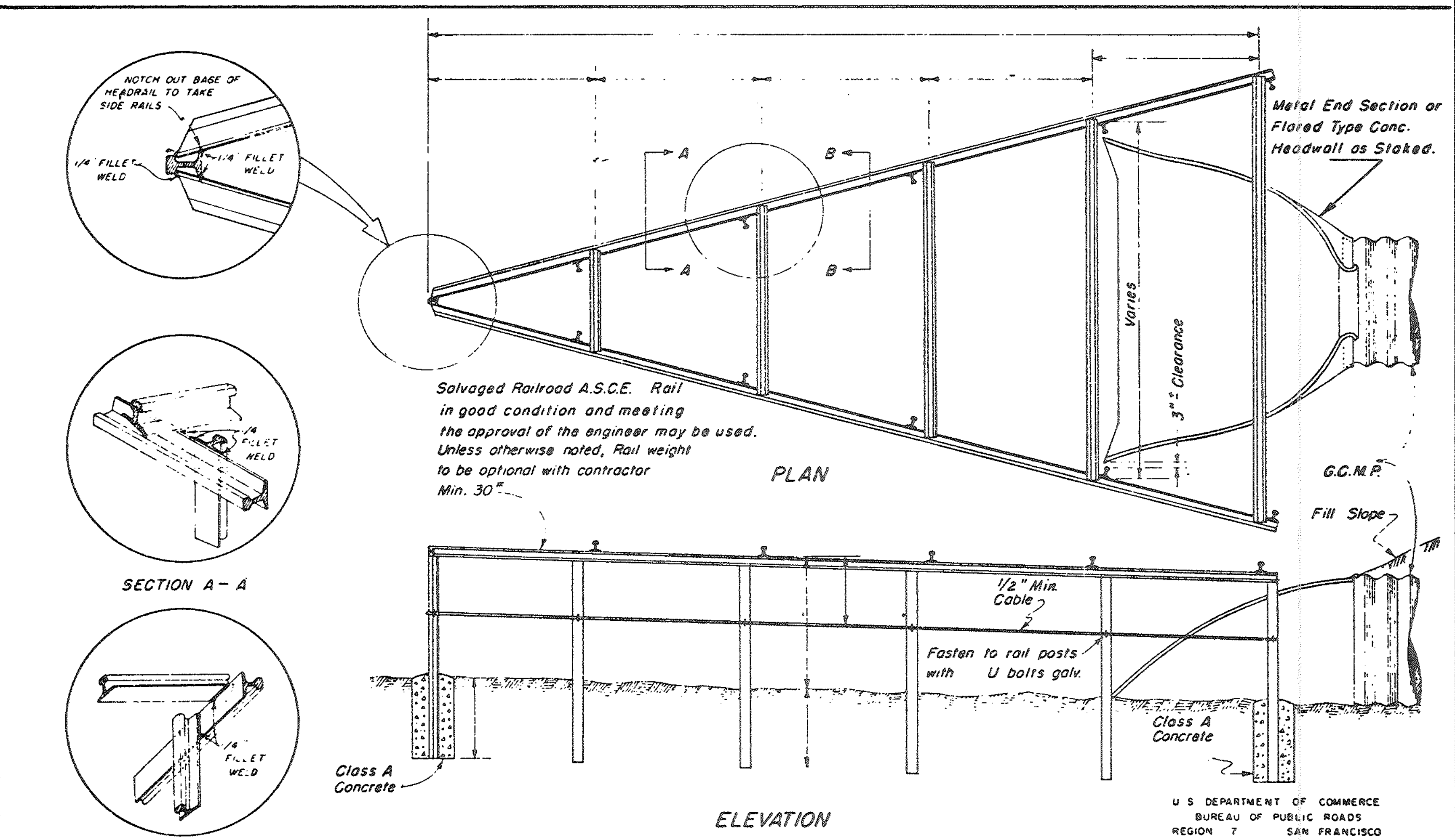


Figure 51. Debris dam and basin along with steel debris rack over culvert entrance in foreground. Metal pipe riser visible over the spillway. Roadway in background.



171

DEBRIS DEFLECTOR
CALIFORNIA DIVISION OF HIGHWAYS
DIVISION 8
PLATE I



TYPICAL UPPER JOINT DETAILS

PLATE II

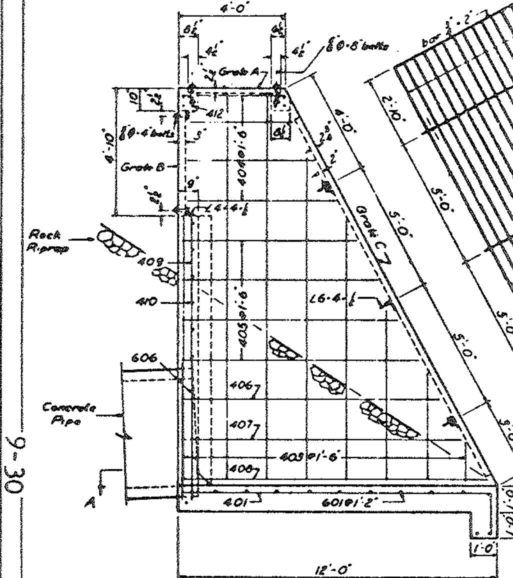
Note: Scales Variable

U S DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
REGION 7 SAN FRANCISCO

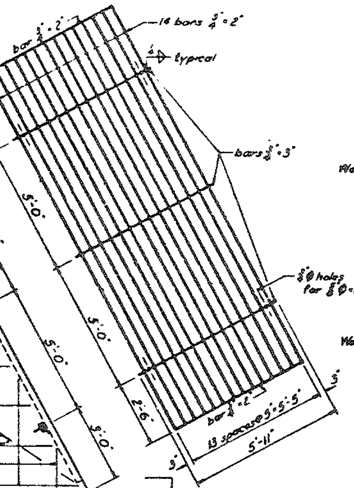
RAIL DEBRIS DEFLECTOR

GENERAL NOTES
Concrete shall be Class A (f'_c = 3600)
Reinforcing steel shall be deformed bars of intermediate grade conforming to ASTM Specification A15
Structural steel shall be A-36

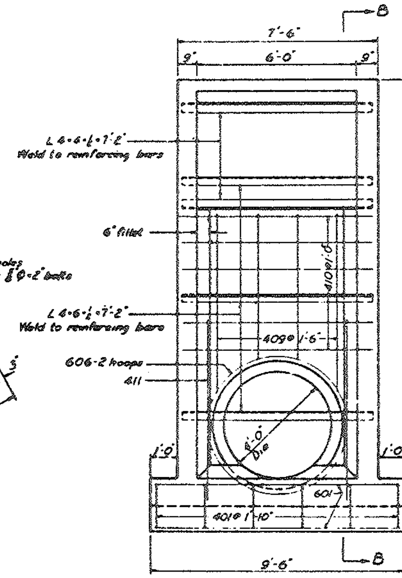
QUANTITIES	
Concrete	13 cu yds
Reinforcing Steel	1540 lbs
Structural Steel	2440 lbs



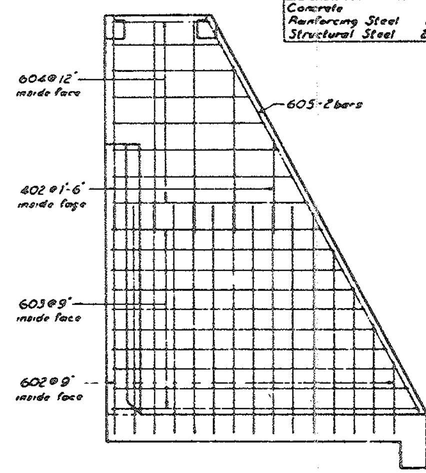
SIDE VIEW
outside face reinforcing shown



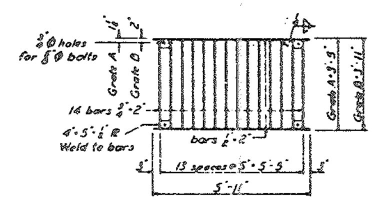
GRATE C



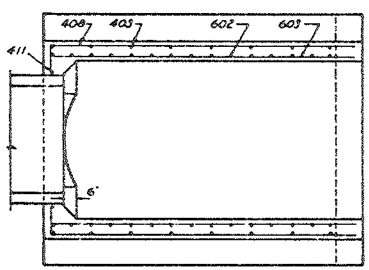
FRONT VIEW
grates not shown



SECTION B-B



GRATES A&B

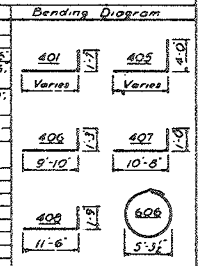


SECTION A-A

REINFORCING STEEL SCHEDULE

Mark	Location	No	Type	Size	Length	
601	Base	13	str	#6	9'-2"	
401	"	6	bent	4	Varies	2@12'-4", 4@13'-1"
602	Wall, inside face, vert.	30	str	6	Varies	20@8'-9", 2ea@8'-0", 6'-9", 5'-6", 4'-0", 2'-6"
603	" " " horiz	20	str	6	Varies	2ea@11'-6", 11'-1", 10'-8", 10'-3", 9'-10", 9'-5", 9'-0", 8'-7", 8'-2", 7'-9"
604	" " " horiz	16	str	6	Varies	2ea@3'-10", 4'-4", 4'-10", 5'-4", 5'-10", 6'-4", 6'-10", 7'-4"
402	" " " vert	10	str	4	Varies	6@8'-0", 2@6'-0", 2@3'-6"
605	Wall, front slope	4	str	6	16'-8"	
403	Wall, outside face, vert	16	str	4	Varies	6@14'-8", 2ea@13'-2", 10'-2", 7'-9", 4'-6", 2'-0"
404	" " " horiz	8	str	4	Varies	2ea@6'-0", 5'-2", 4'-5", 3'-8"
405	" " " horiz	8	bent	4	Varies	2ea@10'-9", 11'-6", 12'-3", 13'-0"
406	" " " horiz	2	bent	4	11'-1"	
407	" " " horiz	2	bent	4	11'-8"	
408	" " " horiz	2	bent	4	13'-5"	
606	Backwall, around pipe	2	bent	6	17'-7"	
409	" inside face, vert	4	str	4	Varies	2@5'-6", 2@6'-9"
410	" " " horiz	6	str	4	7'-2"	
411	" outside face, vert	2	str	4	7'-0"	
412	Strut	8	str	4	7'-2"	

Bending Diagram



All dimensions are out to out

SPECIAL INLET – DEBRIS RACK

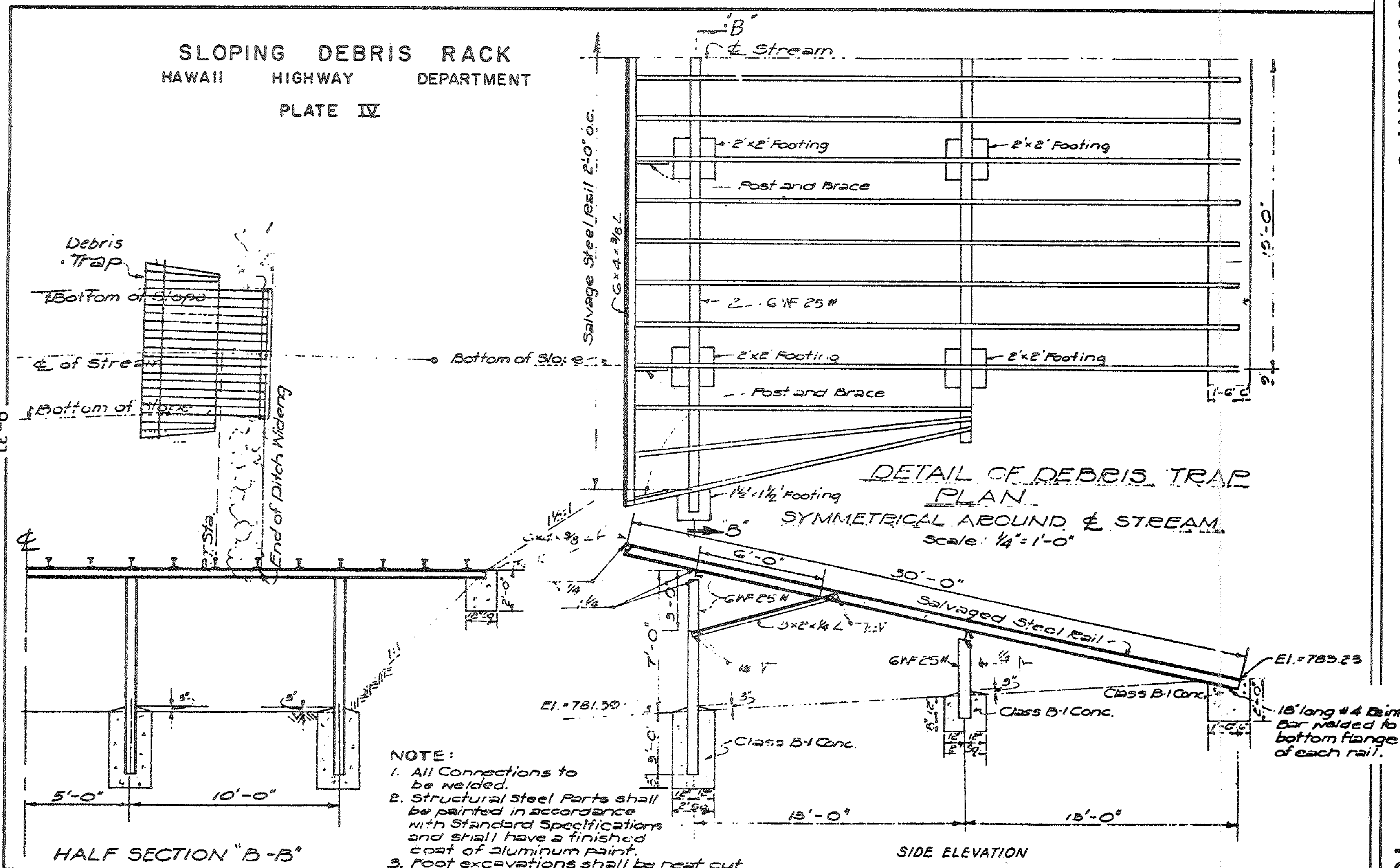
WASHINGTON STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
OLYMPIA, WASHINGTON

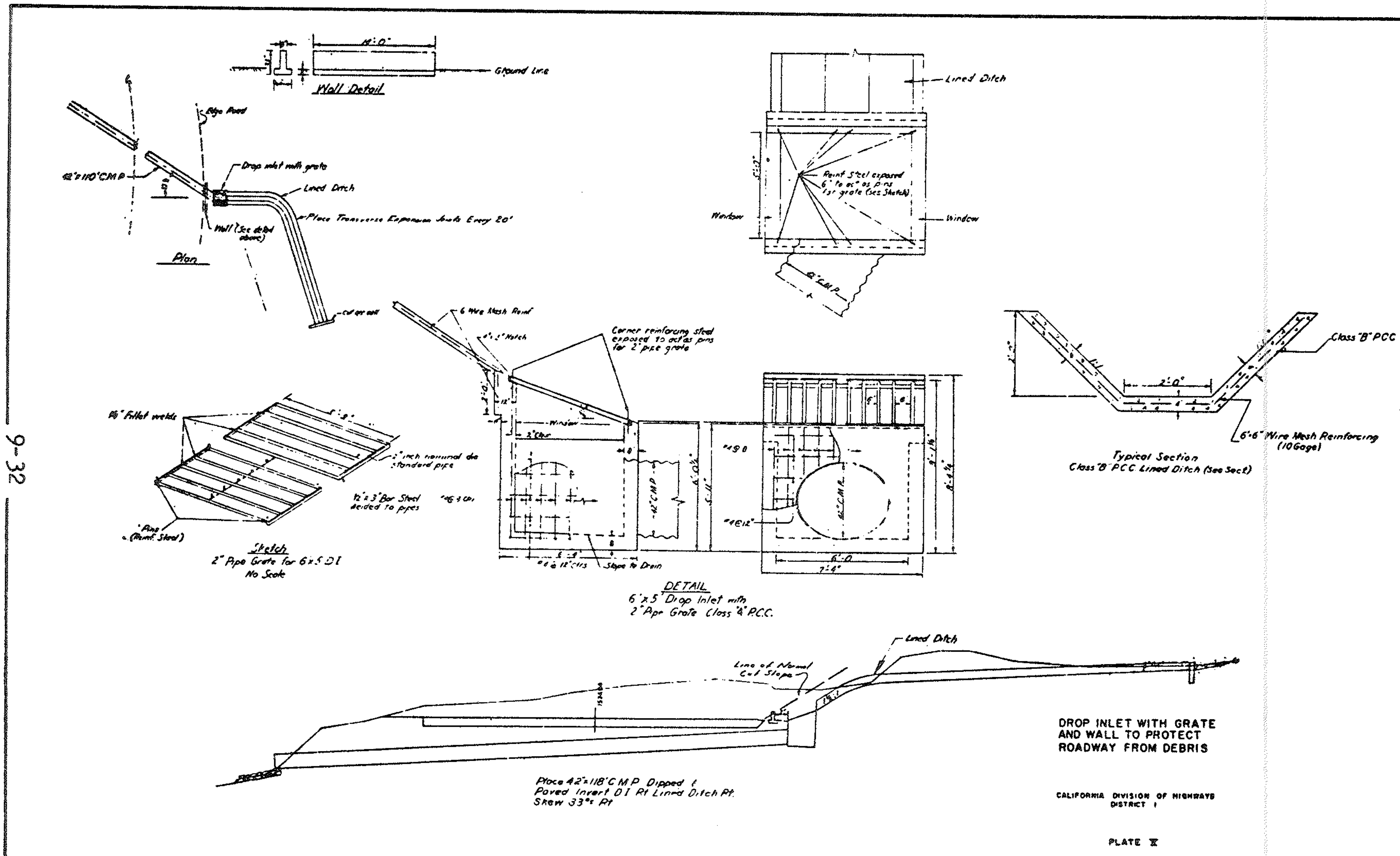
0 1 2 3
FEET

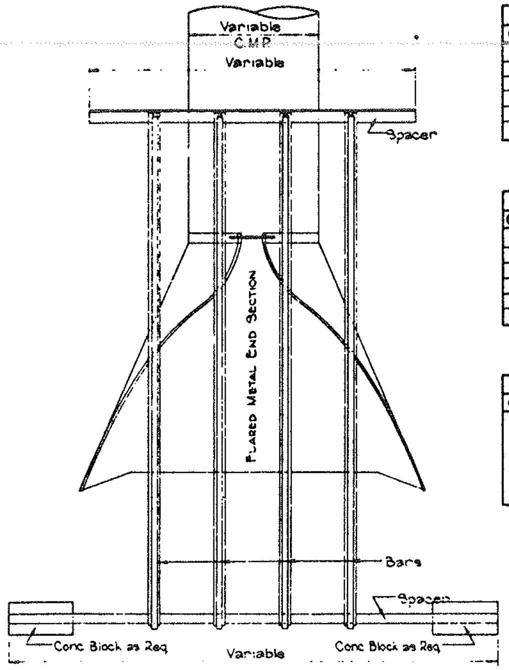
PLATE III

Compendium 3 Text 5

9-31




Wall Detail
Ground Line
Plan
Edge Road
42"x110' CMP
Drop inlet with grate
Lined Ditch
Place Transverse Expansion Joints Every 20'
Wall (See detail above)
Lined Ditch
Window
Window
Sketch
2" Pipe Grate for 6x5 D.I.
No Scale
Corner reinforcing steel exposed to act as pins for 2" pipe grate
DETAIL
6'x5' Drop Inlet with 2" Pipe Grate Class "A" P.C.C.
Class "B" PCC
2'-0"
6-6" Wire Mesh Reinforcing (10 Gage)
Typical Section
Class "B" PCC Lined Ditch (See Sect)
Lined Ditch
Line of Normal Cut Slope
Place 42"x118' C.M.P. Dipped & Paved Invert D.I. Rt Lined Ditch Rt. Skew 33° Rt
DROP INLET WITH GRATE AND WALL TO PROTECT ROADWAY FROM DEBRIS
CALIFORNIA DIVISION OF HIGHWAYS DISTRICT I
PLATE X



PLAN

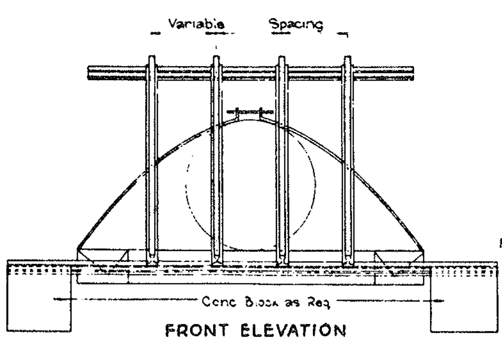
BAR SPACING FOR VARIOUS CULVERTS		
C.M.P.	BARS REQUIRED	BAR SPACING
18"	3	1'-0"
24"	4	1'-2"
30"	4	1'-4"
36"	4	1'-8"
42"	5	1'-8"
48"	5	1'-9"

REQUIRED LENGTH OF BARS			
C.M.P.	SLOPE OF BAR	BAR LENGTH	MATERIAL
18"	3:1	8'-3"	Std 3" Pipe or 25 to 60 Lb. Rail
24"	3:1	11'-0"	" " " " "
30"	3:1	12'-0"	" " " " "
36"	3:1	13'-6"	40" to 60" Rail or Steel I's
42"	3:1	15'-0	" " " " " " "
48"	3:1	16'-6"	" " " " " " "

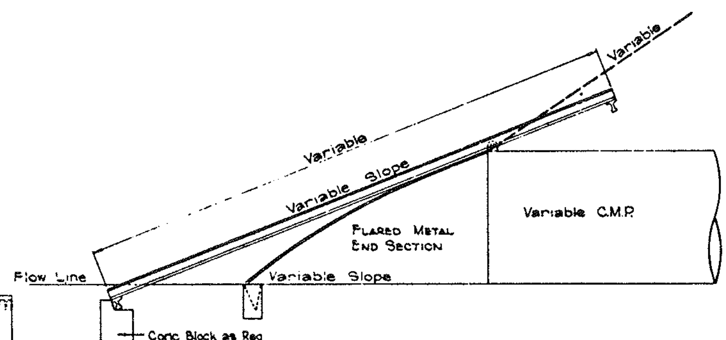
LENGTH OF SPACERS			
C.M.P.	TOP SPACER	BOTTOM SPACER	MATERIAL
18"	6'-0"	8'-0"	4"x4"x3/8" ∠s
24"	7'-0"	10'-0"	40 to 60 Lb.
30"	7'-6"	11'-0"	Rail or 3"
36"	8'-0"	12'-6"	Pipe
42"	9'-0"	13'-9"	
48"	10-0"	15-0"	

NOTE

- Special Treatment Required For Pipes Larger Than 48"
- Minimum Bar Spacing 0'-10"
- Maximum Bar Spacing 2'-0"
- Gradients Steeper Than 15% May Require Special Treatment
- Sizes Shown Are Minimums To Be Used.
- Heavier Sections Permitted In All Cases



FRONT ELEVATION



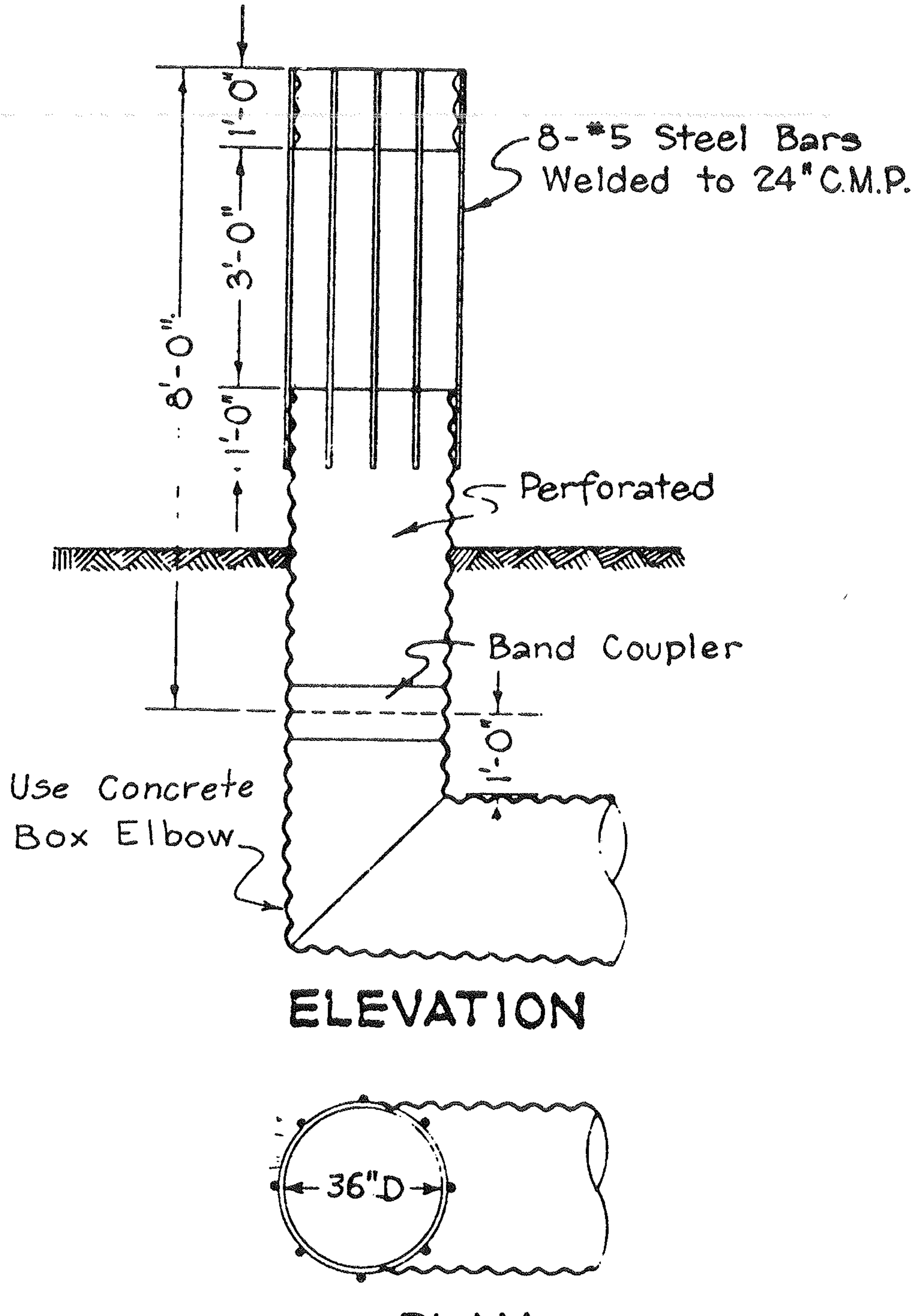
RIGHT SIDE ELEVATION

STANDARD DEBRIS RACK FOR 18" TO 48" C.M.P.'s

FOR USE WITH METAL END SECTION

CALIFORNIA DIVISION OF HIGHWAYS

PLATE VI



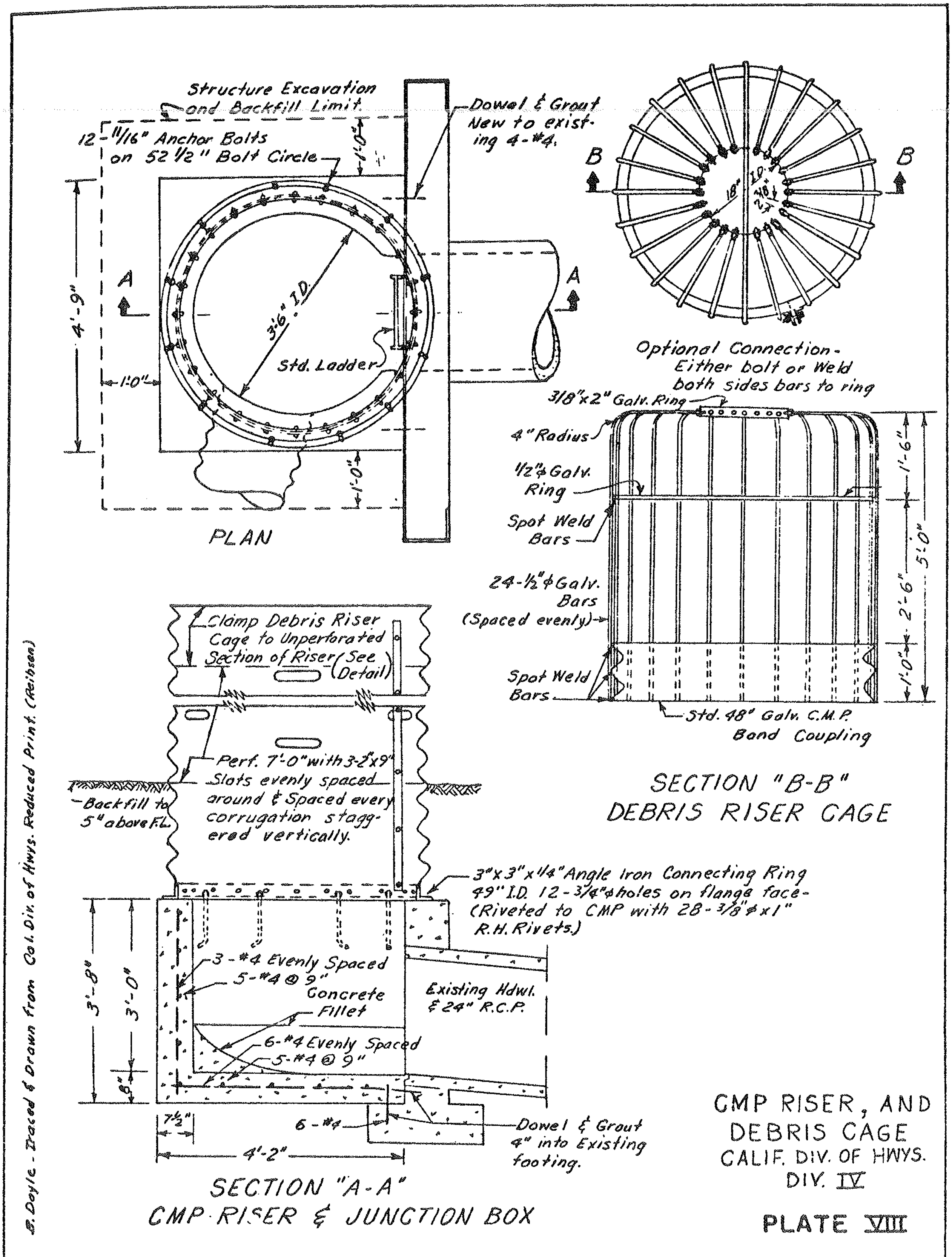
DEBRIS RISER

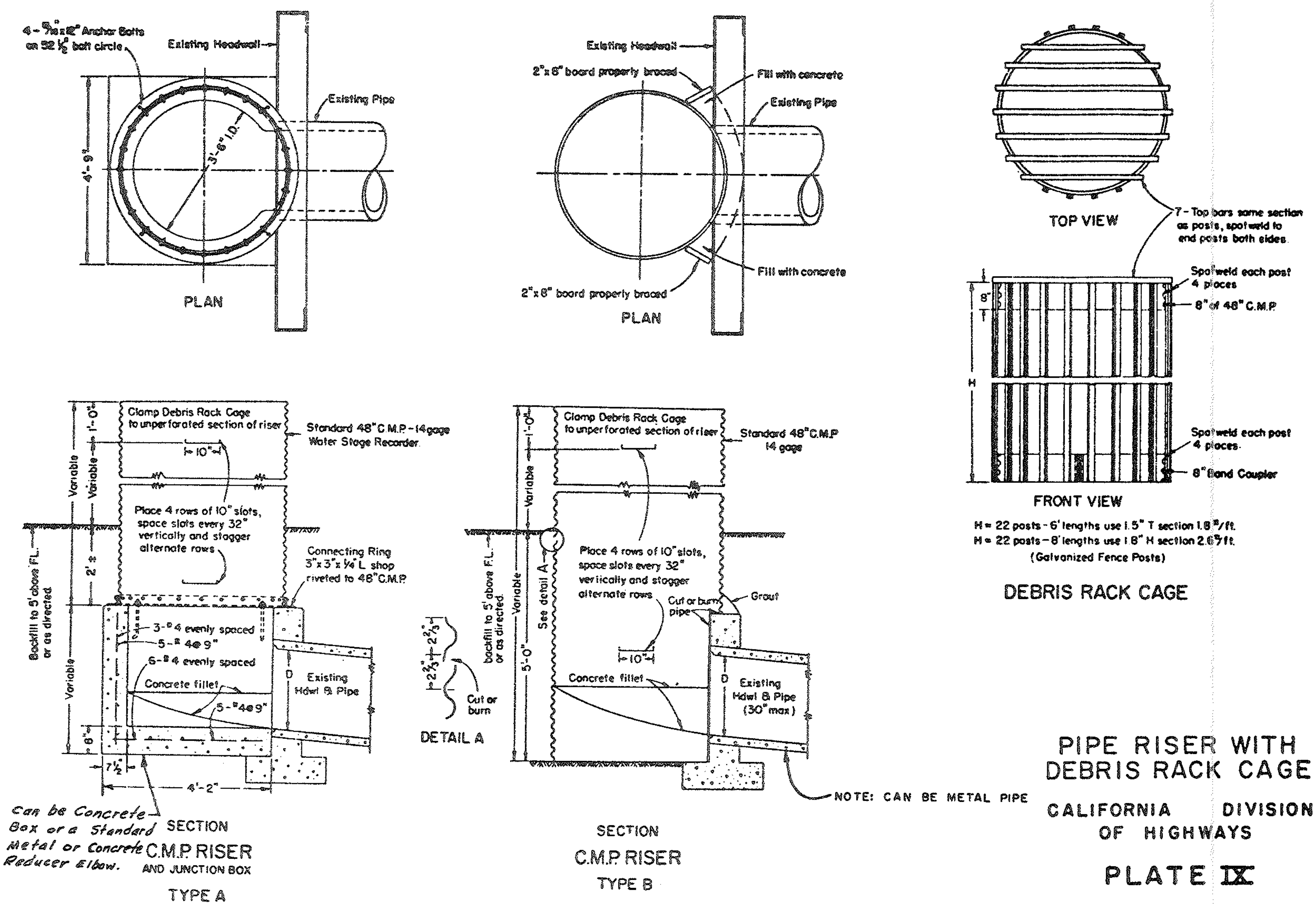
California Division of Highways

District II

PLATE VII

178


Structure Excavation and Backfill Limit.
12-11/16" Anchor Bolts on 52 1/2" Bolt Circle
Dowel & Grout New to existing 4-#4.
1'-0"
4'-9"
1'-0"
3'-6" I.D.
Std. Ladder
A
A
1'-0"
PLAN
B
B
18" I.D.
Optional Connection-Either bolt or Weld both sides bars to ring
3/8"x2" Galv. Ring
4" Radius
1/2"φ Galv. Ring
Spot Weld Bars
24-1/2"φ Galv. Bars (Spaced evenly)
Spot Weld Bars
Std. 48" Galv. C.M.P. Band Coupling
1'-6"
5'-0"
2'-6"
1'-0"
SECTION "B-B"
DEBRIS RISER CAGE
Clamp Debris Riser Cage to Unperforated Section of Riser (See Detail)
Perf. 7'-0" with 3-2"x9" Slots evenly spaced around & Spaced every corrugation staggered vertically.
Backfill to 5" above F.L.
3"x3"x1/4" Angle Iron Connecting Ring 49" I.D. 12-3/4"φ holes on flange face-(Riveted to CMP with 28-3/8"φx1" R.H. Rivets.)
3-#4 Evenly Spaced
5-#4 @ 9"
Concrete Fillet
6-#4 Evenly Spaced
5-#4 @ 9"
Existing Hdwl. & 24" R.C.P.
3'-8"
3'-0"
8"
7 1/2"
6-#4
4'-2"
Dowel & Grout 4" into Existing footing.
SECTION "A-A"
CMP RISER & JUNCTION BOX
B. Doyle - Traced & Drawn from Cal. Div. of Hwys. Reduced Print. (Reibsen)
CMP RISER, AND DEBRIS CAGE
CALIF. DIV. OF HWYS.
DIV. IV
PLATE VIII



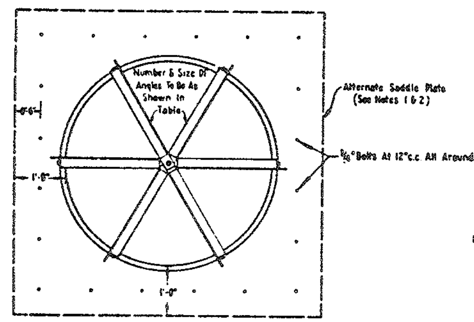
PIPE RISER WITH DEBRIS RACK CAGE

CALIFORNIA DIVISION OF HIGHWAYS

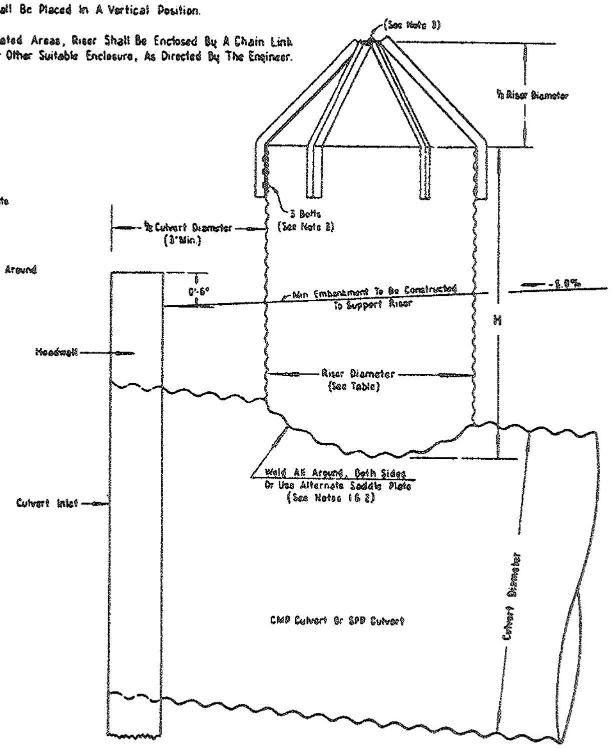
PLATE IX

NOTES:

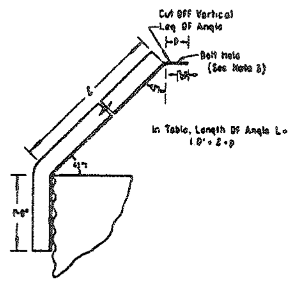
1. Riser To Be Fastened To Culvert By Welding Or Shop-Made Corrugated Metal Saddle Plate.
2. Saddle Plate Shall Be Same Gage Metal As Top Culvert Plates.
3. Cage Angles Shall Be Either Welded Or Bolted To Riser, And Top Joint Of Cage Angles Shall Be Either Welded Or Bolted. Bolt Diameter Shall Be Twice Angle Thickness.
4. All Angles, Nuts And Bolts Shall Be Galvanized.
5. All Welds On Galvanized Metal Shall Be Treated With Zinc Dust In Accordance With Section 66-1.02 G Of The Standard Specifications.
6. Riser Shall Be Placed In A Vertical Position.
7. In Populated Areas, Riser Shall Be Enclosed By A Chain Link Fence Or Other Suitable Enclosure, As Directed By The Engineer.



TOP VIEW
No Scale



ELEVATION
No Scale



CAGE ANGLE DETAIL
No Scale

0 1 2 3
SCALE IN FEET

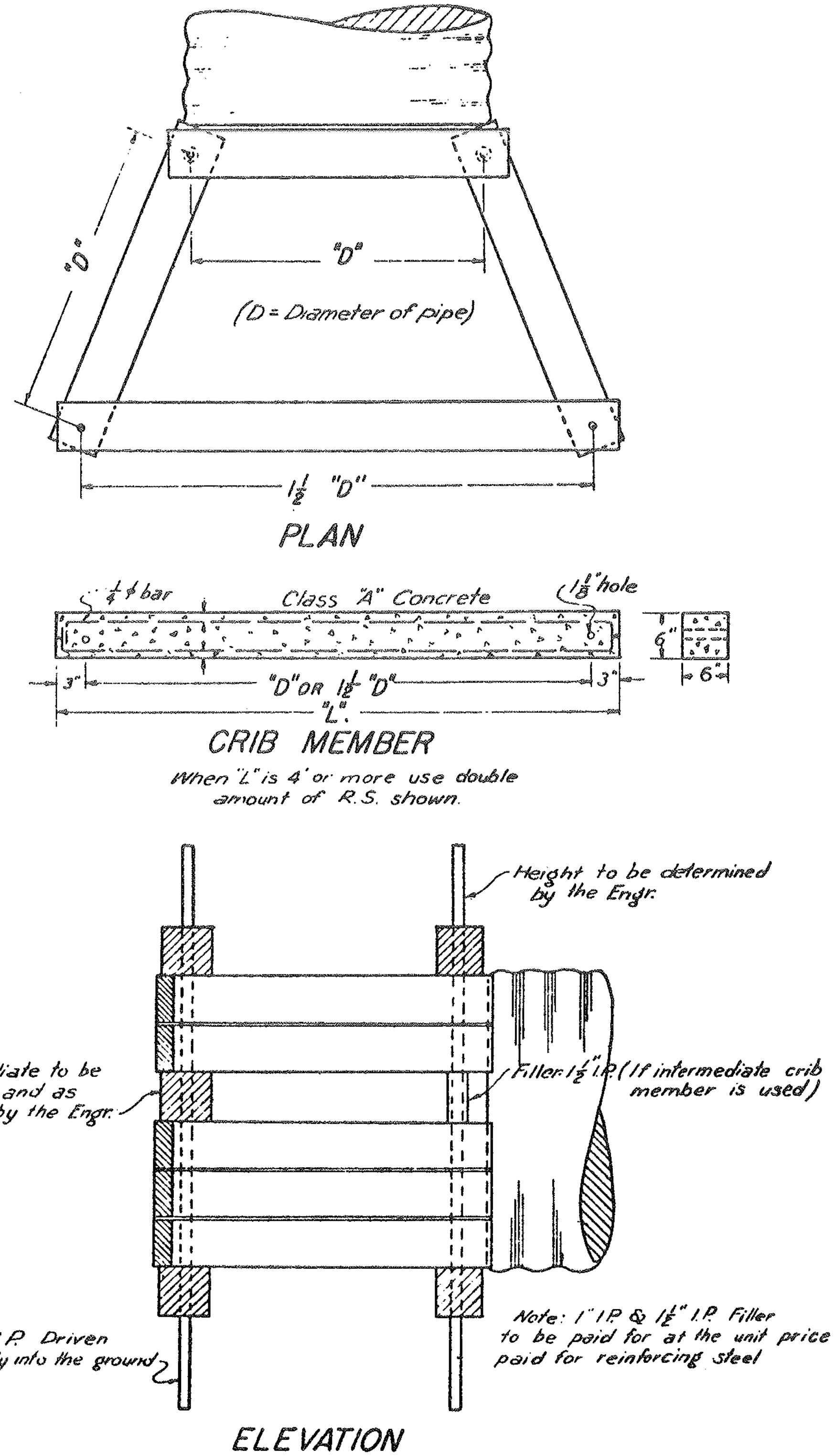
CULVERT DIAM.	RISER			RISER CAGE				
	C.M.P		H*	STEEL				
	DIAM	GAGE		ANGLE SIZE	NO. OF PIECES	LENGTH, FT.		
INCHES	INCHES		FEET			ℓ	P	L
36	24	14	4	2" x 2" x 1/4"	4	1'-5"	2"	2'-7"
42	24	14	4	2" x 2" x 1/4"		1'-5"	2"	2'-7"
48	30	14	4	2½" x 2½" x 1/4"		1'-10"	3"	3'-1"
54	36	12	4			2'-1"	3"	3'-4"
60	42	12	6			2'-6"	3"	3'-9"
66	42	12	6			2'-6"	3"	3'-9"
72	48	12	6			2'-10"	3"	4'-1"
78	48	12	6			2'-10"	3"	4'-1"
84	54	12	6	3" x 3" x 1/4"		3'-3"	3"	4'-6"
90	60	10	8			3'-6"	3"	4'-9"
96	60	10	8			3'-6"	3"	4'-9"
102	66	10	8			3'-11"	3"	5'-2"
108	72	10	8	3½" x 3½" x 5/16"		4'-3"	4"	5'-7"
114	72	10	8			4'-3"	4"	5'-7"
120	78	8	8			4'-8"	4"	6'-0"
126	84	8	10			5'-0"	4"	6'-4"
132	84	8	10			5'-0"	4"	6'-4"
138	90	8	10		6	5'-4"	7"	6'-11"
144	96	8	10			5'-6"	7"	7'-3"
150	96	8	10			5'-6"	7"	7'-3"
156	102†	12	12	4" x 4" x 3/8"	8	6'-0"	11"	7'-11"
162	108†	12	12			6'-5"	11"	8'-4"
168	108†	12	12			6'-5"	11"	8'-4"
174	114†	10	12			6'-9"	11"	8'-8"
180	120†	10	12			7'-1"	11"	9'-0"

* H May Be Varied. See Culvert Detail Sheets For Variable H.
† Structural Plate Pipe.

STANDARD RELIEF RISER

CALIFORNIA DIVISION OF HIGHWAYS

PLATE X



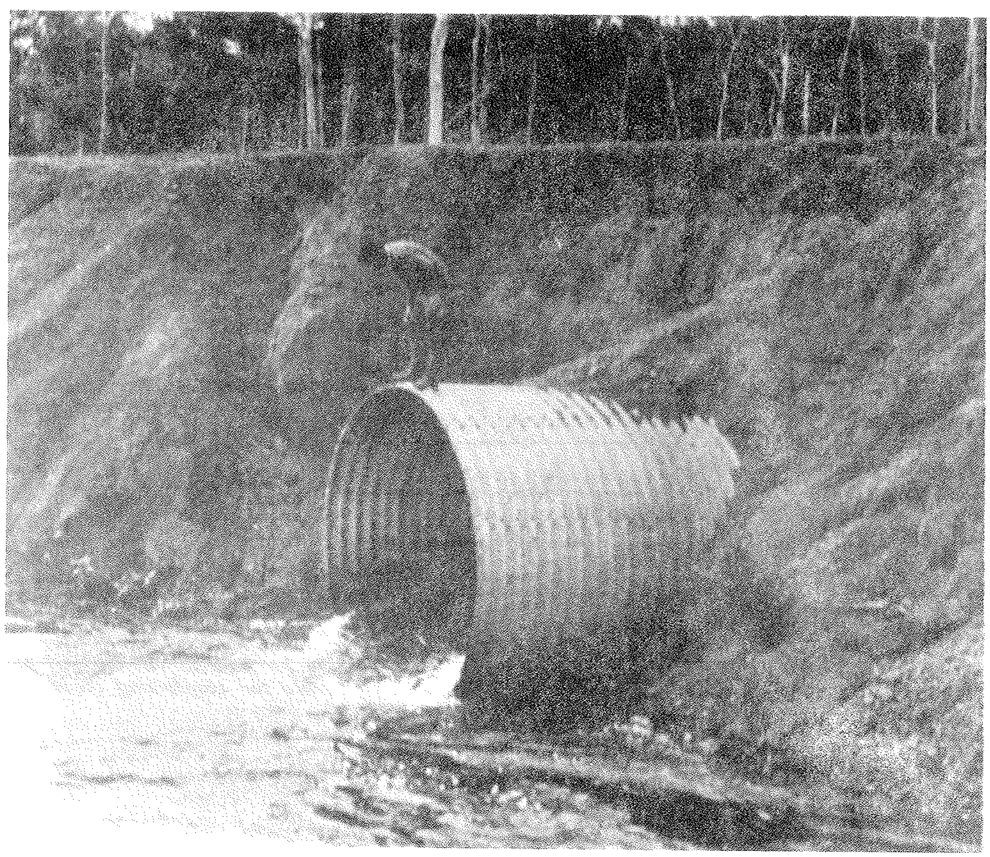
DEBRIS CRIB

CALIFORNIA DIVISION OF HIGHWAYS

DISTRICT 8

PLATE XI

182



Erosion is evident at and around culvert—Brazil.

TECHNICAL REPORT H-74-9

PRACTICAL GUIDANCE FOR DESIGN OF LINED CHANNEL EXPANSIONS AT CULVERT OUTLETS

Hydraulic Model Investigation

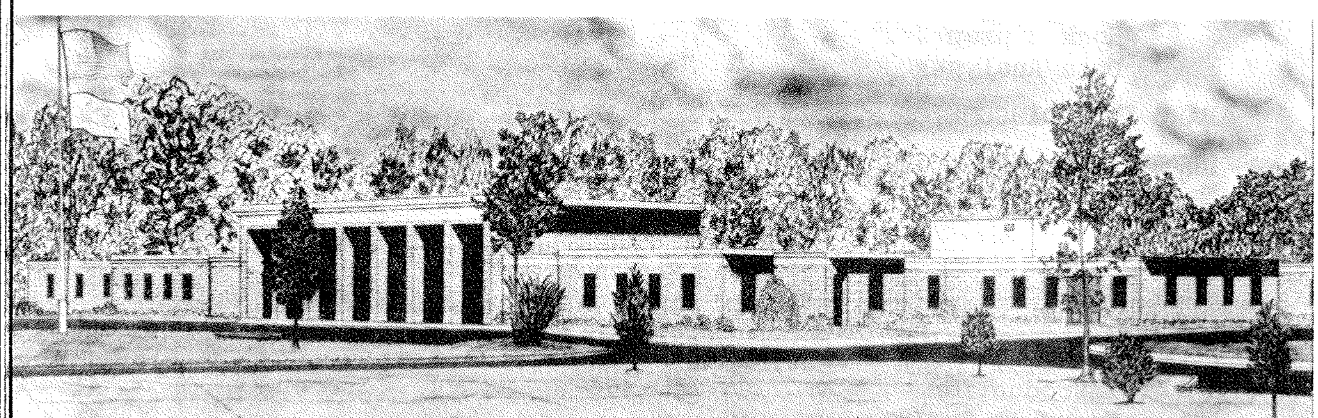
by

Bobby P. Fletcher, John L. Grace, Jr.

Hydraulics Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

October 1974
Final Report

Approved For Public Release; Distribution Unlimited



Prepared for Louisiana Department of Highways and
U. S. Department of Transportation, Federal Highway Administration

Under State Project No. 736-01-54
FAP No. HPR-1 (70-1H)

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CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	0.0254	meters
feet	0.3048	meters
square feet	0.092903	square meters
cubic feet	0.02831685	cubic meters
pounds (mass)	0.45359237	kilograms
feet per second	0.3048	meters per second
cubic feet per second	0.02831685	cubic meters per second
feet per second per second	0.3048	meters per second per second

APPENDIX A: PRACTICAL GUIDANCE FOR ESTIMATING AND CONTROLLING EROSION AT STORM SEWER AND CULVERT OUTLETS

Introduction

1. This appendix summarizes and demonstrates application of the results of research conducted at the U. S. Army Engineer Waterways Experiment Station (WES) during the past decade to develop practical guidance for estimating and controlling erosion downstream of storm sewer and culvert outlets. Initial efforts were concerned with investigation and development of means of estimating the extent of scour to be anticipated downstream of outlets. Subsequent efforts have involved investigation and evaluation of various schemes of protection for controlling erosion such as a cutoff wall, horizontal blankets of rock riprap, preformed scour holes lined with rock riprap, and channel expansions lined with natural and artificial revetments. In addition, efforts have been made to determine the limiting discharges for various energy dissipators including simple flared outlet transitions, stilling wells, U. S. Bureau of Reclamation type VI basins, and St. Anthony Falls stilling basins. Empirical equations and charts are presented for estimating the extent of localized scour to be anticipated downstream of outlets, the size and extent of various natural and artificial type revetments, and the appropriate dimensions of each type of energy dissipator investigated. With these results, designers can estimate the extent of scour to be expected and select appropriate and alternative schemes of protection for controlling erosion downstream of storm sewer and culvert outlets.

Scour at Outlets

2. In general, two types of channel instability can develop downstream from storm sewer and culvert outlets, i.e. either gully scour or localized erosion termed a scour hole. Distinction between the two conditions can be made by comparing the original or existing slope of the channel or drainage basin downstream of the outlet relative to that required for stability as illustrated in Figure A1.

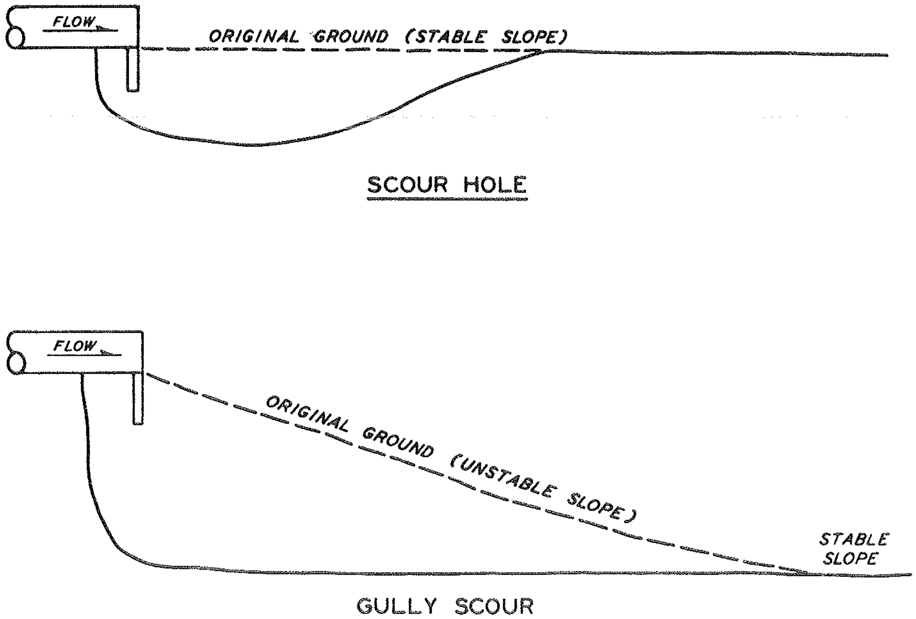


Figure A1. Types of scour at culvert outlets

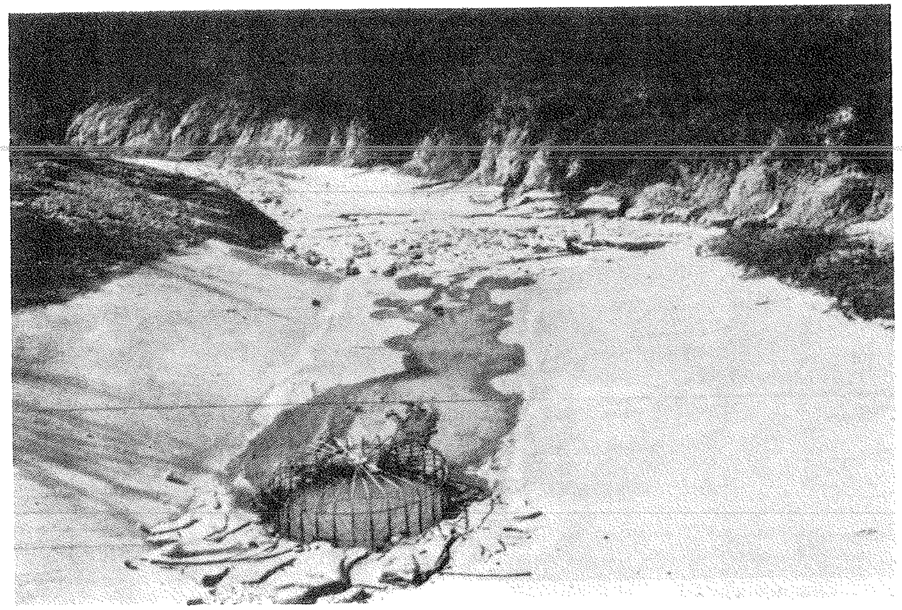
Figure A2. Failure of outlet structure due to gully scour

3. Gully scour is to be expected when the Froude number of flow in the channel exceeds that required for stability. It begins at a control point downstream where the channel is stable and progresses upstream. If sufficient differential in elevation exists between the outlet and the section of stable channel, the outlet structure will be completely undermined as shown in Figure A2. The primary cause of gully scour is the practice of siting outlets high, with or without energy dissipators, relative to a stable downstream grade in order to reduce quantities of pipe and excavation. Erosion of this type may be of considerable extent depending upon the location of the stable channel section relative to that of the outlet in both the vertical and downstream directions. To prevent gully erosion, outlets and energy dissipators should be located at sites where the slope of the downstream channel or drainage basin is naturally mild enough to remain stable under the anticipated conditions or else it should be controlled by ditch checks, drop structures, and/or other means to a point where a naturally stable slope and cross section exist. Outlets and energy dissipators should not be located within channels or drainage basins experiencing deposition but adjacent to the perimeter and provided with an outlet channel that is skewed rather than perpendicular to the main channel or basin (Figure A3).

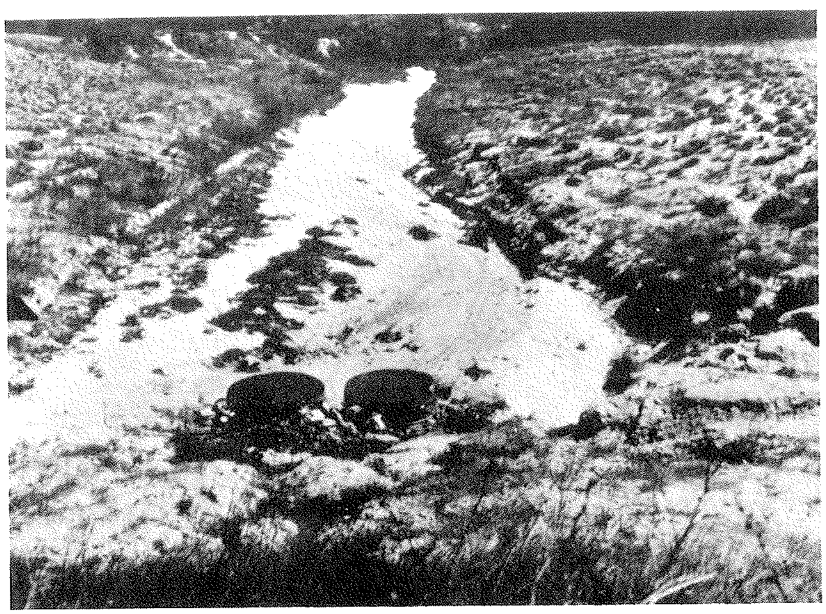
4. A scour hole or localized erosion is to be expected downstream of an outlet (Figure A4) even if the downstream channel is stable. The severity of damage to be anticipated depends upon the conditions existing or created at the outlet. In some instances, the extent of the scour hole may be insufficient to produce either instability of the embankment or structural damage to the outlet. However, in many situations flow conditions produce scour of the extent that embankment erosion (Figure 4a) as well as structural damage of the apron, end wall, and culvert (Figure 4b) is evident. Noteworthy surveys of conditions at culvert outlets have been accomplished by Keeley[1*] in Oklahoma and Scheer[2] in Montana.

5. The observations and empirical methods developed by Keeley,[1,3,4]

* Raised numbers refer to similarly numbered items in the References at the end of the main text.

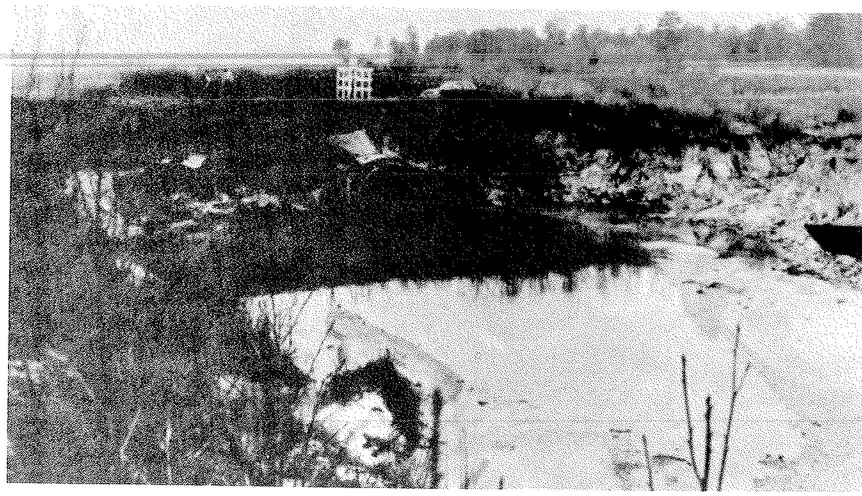


a. Single stilling well with paved perimeter

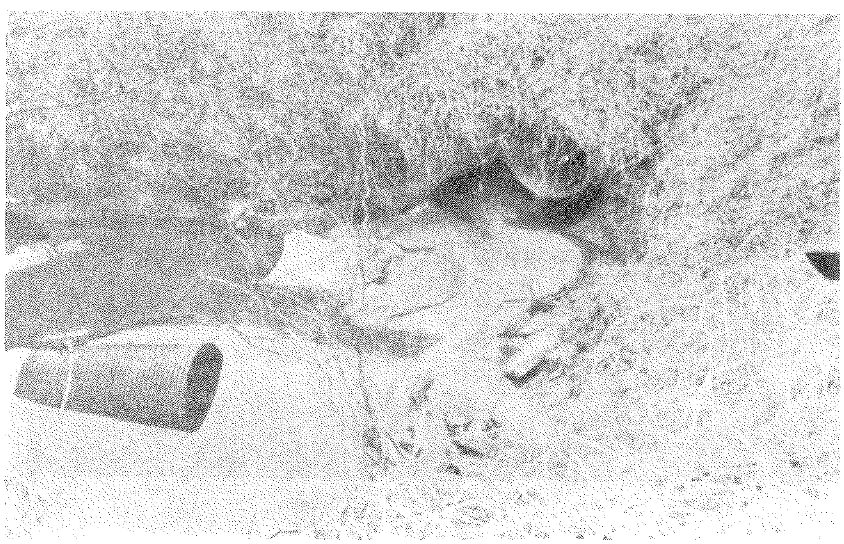


b. Multiple stilling wells without perimeter protection

Figure A3. Single and multiple stilling wells with and without perimeter protection



a. Embankment erosion



b. Structural damage of apron, end wall, and culvert

Figure A4. Damage resulting from localized erosion which provide specific guidance relative to the conditions that produce gully scour or only a localized scour hole as well as those required for stable channels in several Oklahoma soils, merit consideration and application in general. An example of a chart developed by Bohan[5] for design of trapezoidal channels with 1V-on-2H side slopes in a soil that would deposit and erode with Froude numbers of flow less than 0.15 and greater than 0.35, respectively, is shown in Figure A5.

6. Bohan also reported the results of research conducted at WES to determine the extent of localized scour that may be anticipated downstream of circular storm sewer and culvert outlets. These tests indicated that all of the tailwater conditions investigated could be grouped into two categories. Tailwater conditions of less than 0.5 D_o ft above the culvert invert produced approximately the same flow pattern and scour hole geometry and are termed minimum tailwater conditions; all tailwater conditions of 0.5 D_o ft and greater above the culvert invert produced approximately the same flow pattern and scour hole geometry and are termed maximum tailwater conditions. These results agreed very well with those presented by Seaburn and Laushey[6] which indicate that for a constant discharge the velocity just downstream of a circular culvert outlet remains constant for tailwater conditions from 0 to 0.5 D_o ft above the culvert invert. The velocity increases with increasing tailwater and

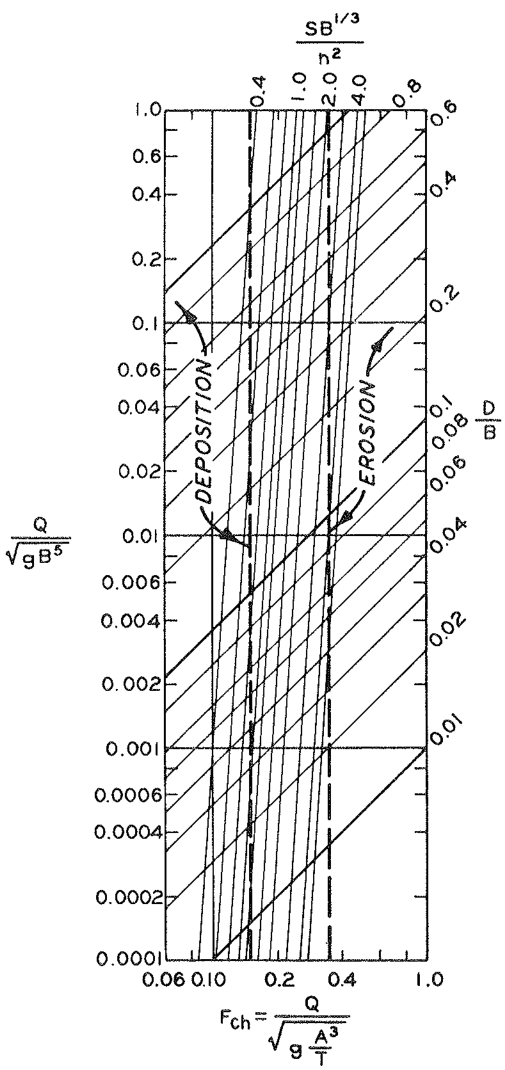


Figure A5. Characteristics of a trapezoidal channel with 1V-on-2H side slopes as a function of Froude number reaches a constant maximum velocity again at a tailwater approximately 1.0 D_o ft above the culvert invert.

7. Empirical equations were developed for estimating the extent of the anticipated scour hole based on knowledge of the design discharge, the culvert diameter, and the duration and Froude number of the design flow at the culvert outlet. However, the relationship between the Froude number of flow at the outlet and a discharge parameter, $Q/D_o^{5/2}$,* for circular and square outlets or $q/D_o^{3/2}$ for rectangular and other shaped outlets can be calculated; and the discharge parameter is just as representative of flow conditions as is the Froude number. The relations between the two parameters for both partial and full pipe uniform flow in square culverts are shown in Figure A6. Since the discharge parameter is easier to calculate and is suitable for application purposes, the original data reported by Bohan were reanalyzed to determine the relations shown in Figures A7-A10 for estimating the extent of localized

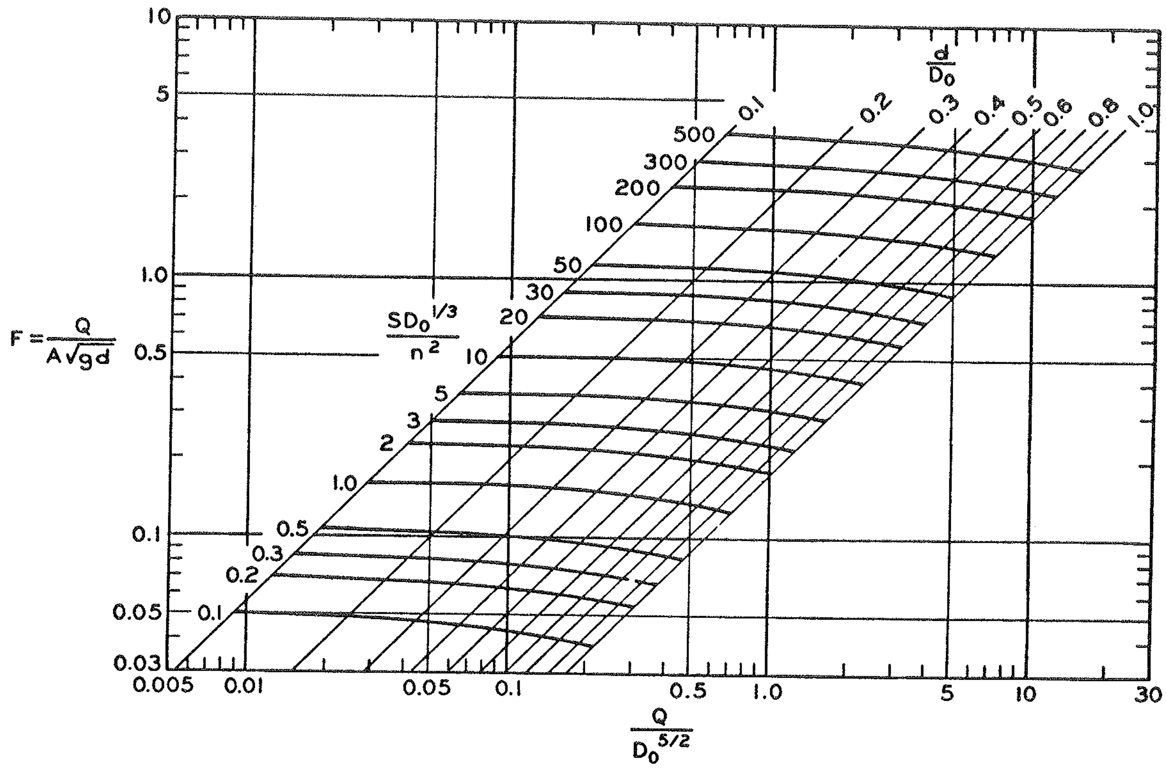


Figure A6. Square culvert - Froude number versus discharge

* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix B).

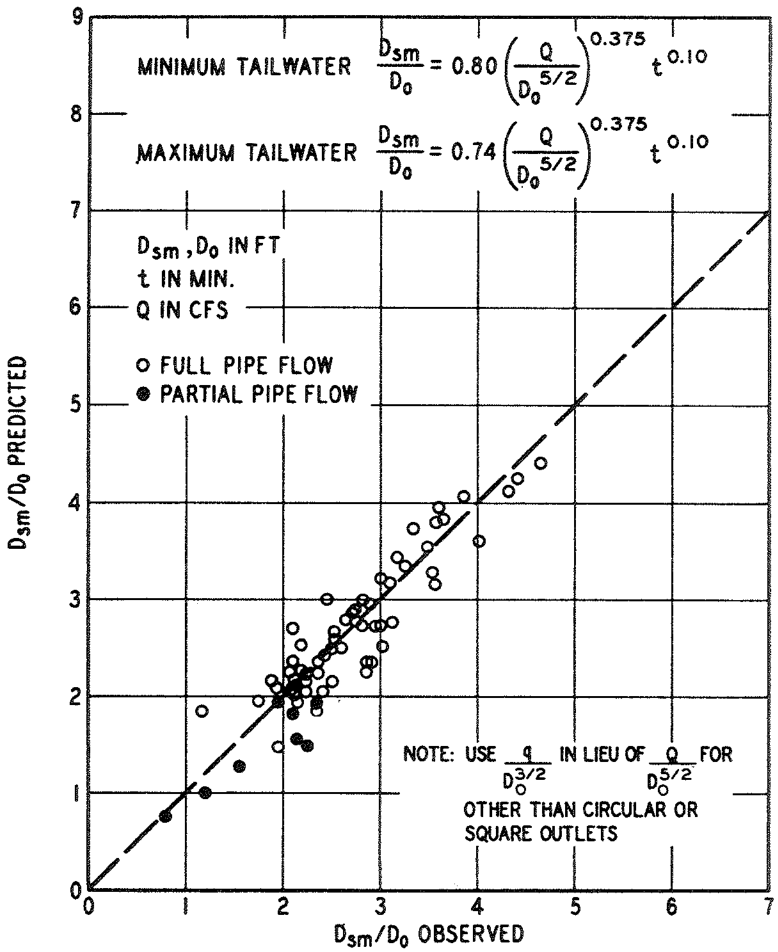


Figure A7. Predicted scour depth versus observed scour depth

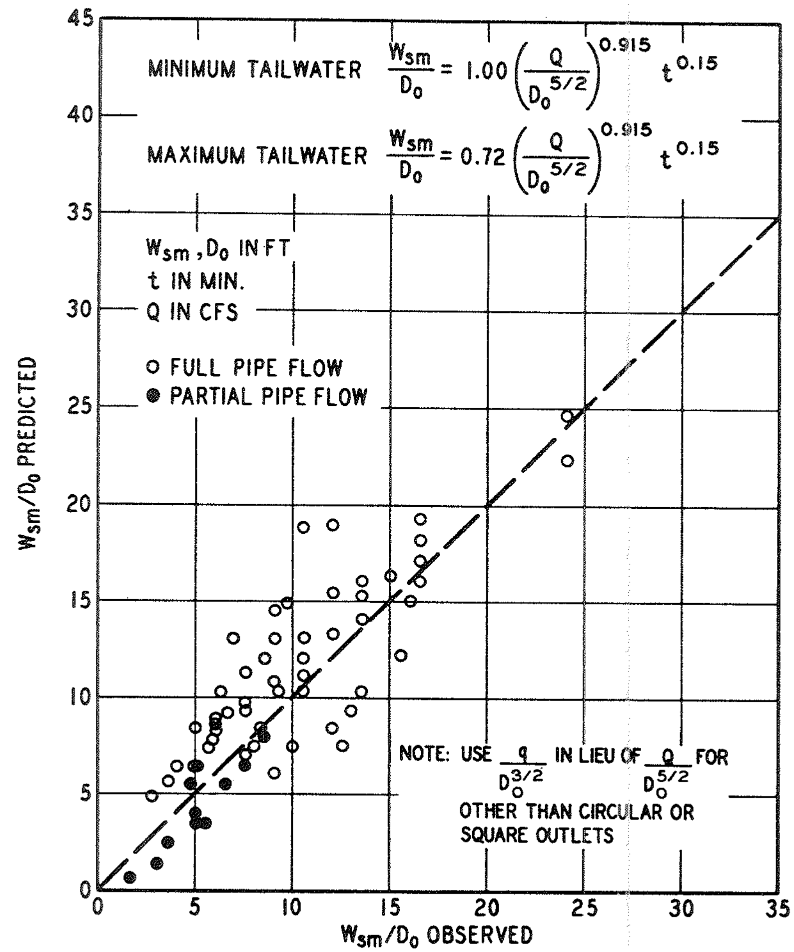


Figure A8. Predicted scour width versus observed scour width

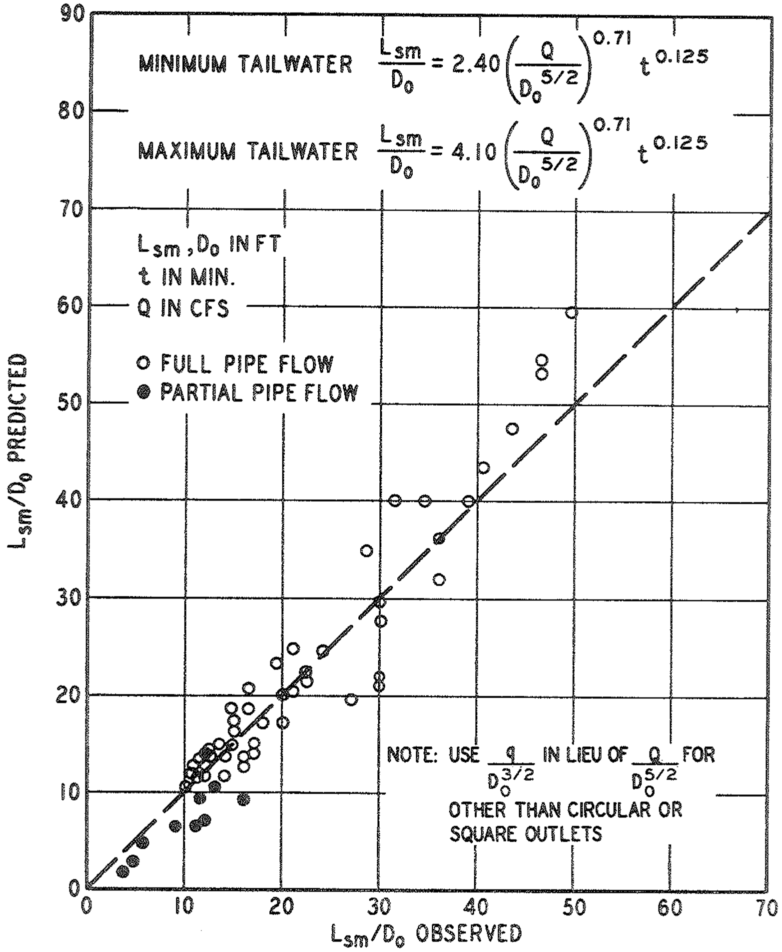


Figure A9. Predicted scour length versus observed scour length

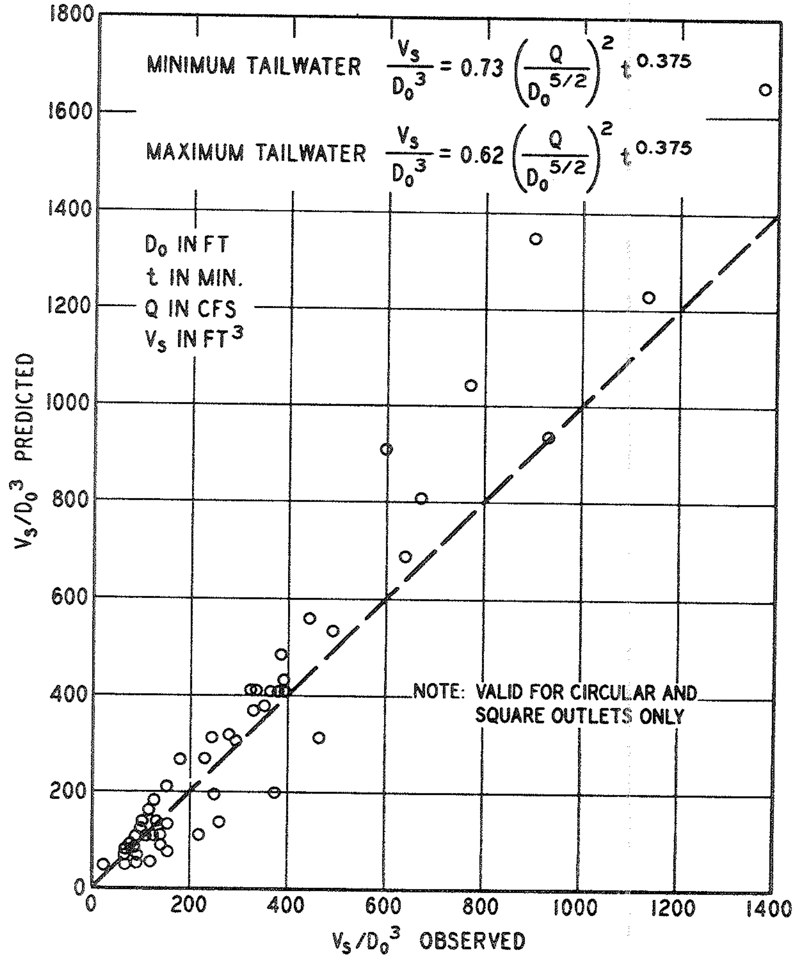


Figure A10. Predicted scour volume versus observed scour volume scour to be anticipated downstream of circular culvert and storm sewer outlets. The variables are defined in Appendix B, and comparisons of predicted and observed values are shown in Figures A7-A10.

8. Dimensionless scour hole geometries determined from model tests with 0.224-ft-, 0.33-ft-, and 1.00-ft-diam circular culverts, a sand with an average grain size of 0.25 mm, and tailwaters less than 0.5 D_o ft as well as equal to or greater than 0.5 D_o ft are presented in Figures A11 and A12, respectively. The maximum depth of scour occurred at a distance 0.4 of the maximum length of scour downstream of the culvert outlet for all tailwater conditions.

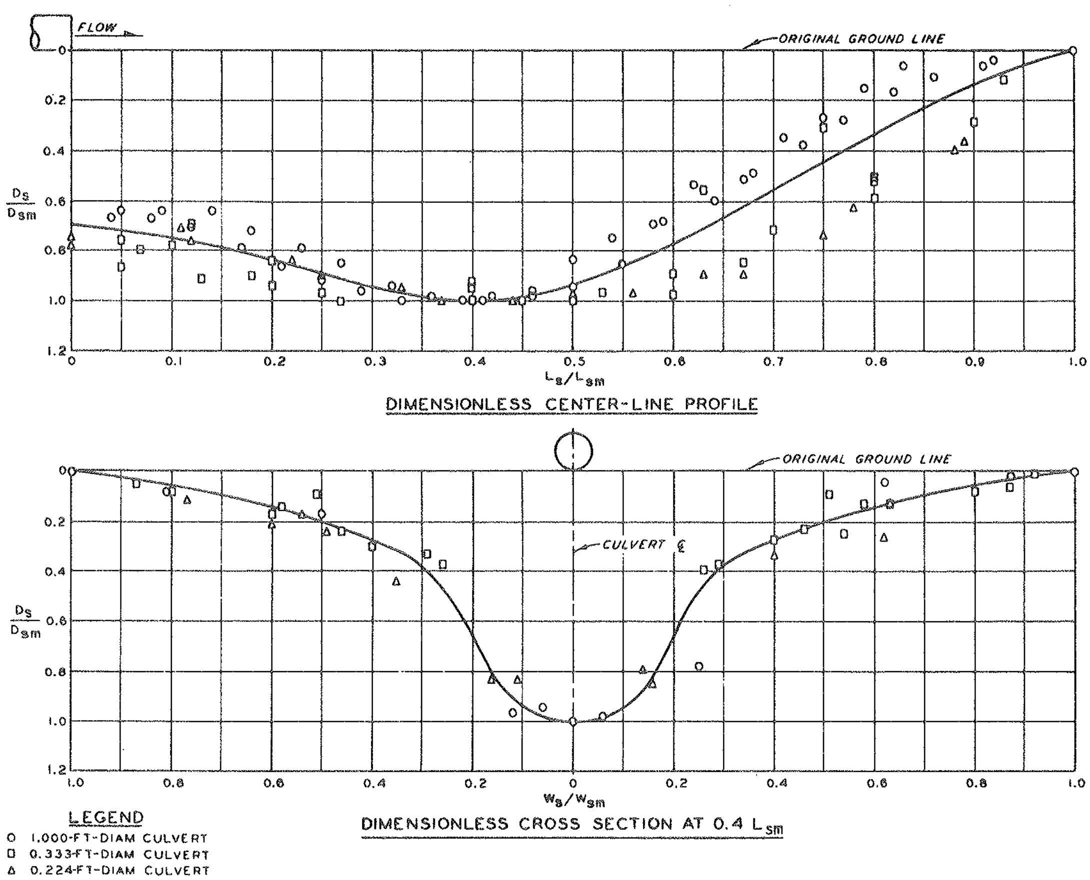


Figure A11. Dimensionless scour hole geometry for minimum tailwater

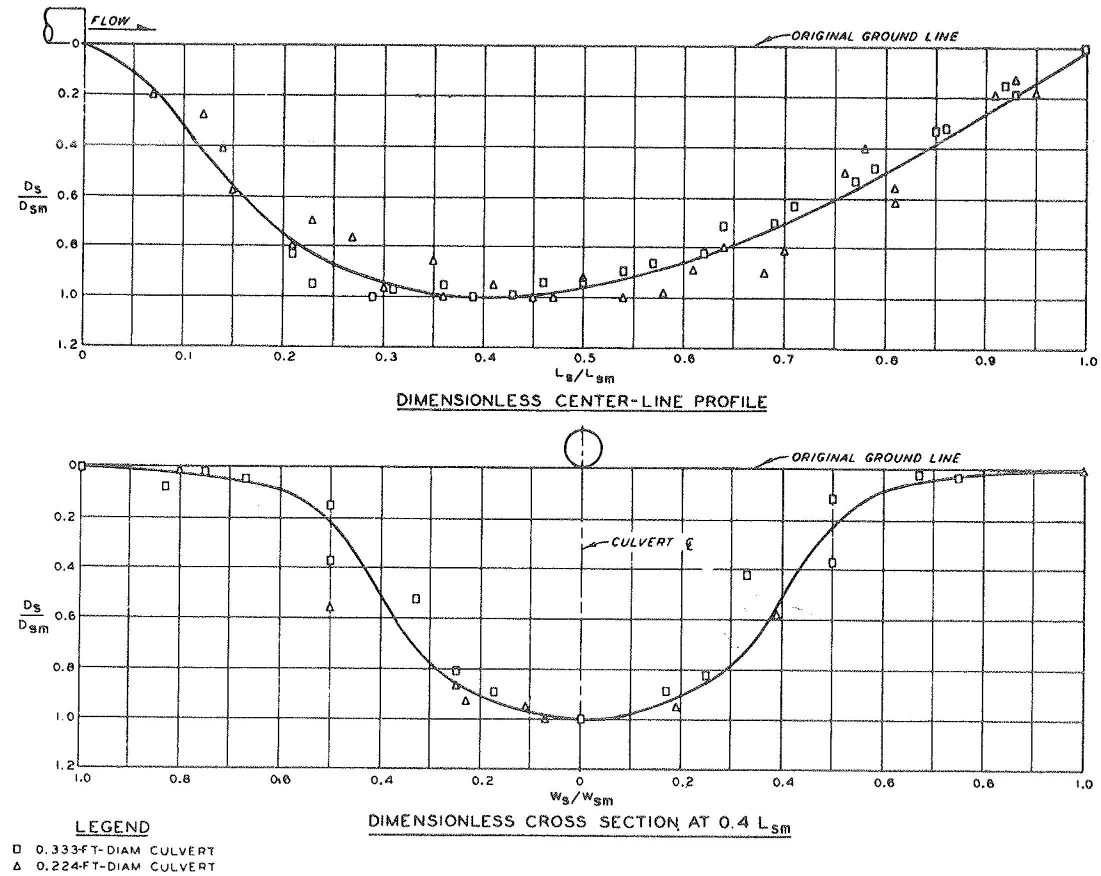


Figure A12. Dimensionless scour hole geometry for maximum tailwater

Cutoff Wall

9. If the location of the outlet is such that a scour hole is not objectionable, it may be practical to allow localized erosion since the scour hole acts as an excellent energy dissipator; however, a cutoff wall which extends to a depth of at least 0.7 of the maximum depth of scour expected (Figure A11) and of appropriate width should be provided to prevent undermining.

Horizontal Blanket of Riprap

10. The average size of stone and configuration of a horizontal blanket of riprap at outlet invert elevation required to control or prevent localized scour downstream of an outlet can be estimated based on the results reported by Bohan and subsequent unreported tests. For a given design discharge, culvert dimensions, and tailwater depth relative to the outlet invert, the minimum average size of stone for a stable horizontal blanket of protection can be estimated by the following relations:

$$\frac{d_{50}}{D_o} = 0.020 \frac{D_o}{TW} \left(\frac{Q}{D_o^{5/2}} \right)^{4/3} \quad \text{Circular and square outlets} \qquad (A1)$$

$$\frac{d_{50}}{D_o} = 0.020 \frac{D_o}{TW} \left(\frac{q}{D_o^{3/2}} \right)^{4/3} \quad \text{Rectangular and other shaped outlets} \qquad (A2)$$

The length of stone protection required downstream of an outlet can be estimated by the relations shown in Figure A13. The variables are defined in Table A1 and the recommended configuration of a horizontal blanket of riprap for control of erosion at an outlet is presented in Figure A14.

Preformed Scour Hole Lined with Riprap

11. The relative advantage of providing both vertical and lateral expansion downstream of an outlet to permit dissipation of excess kinetic energy in turbulence rather than direct attack of the boundaries is shown in Figure A15 which indicates that the required size of stone may be reduced considerably if a riprap-lined, preformed scour hole is provided in lieu of a horizontal blanket at an elevation essentially the same as the outlet invert. Details of a scheme of riprap protection termed "preformed scour hole lined with riprap" are shown in Figure A16.

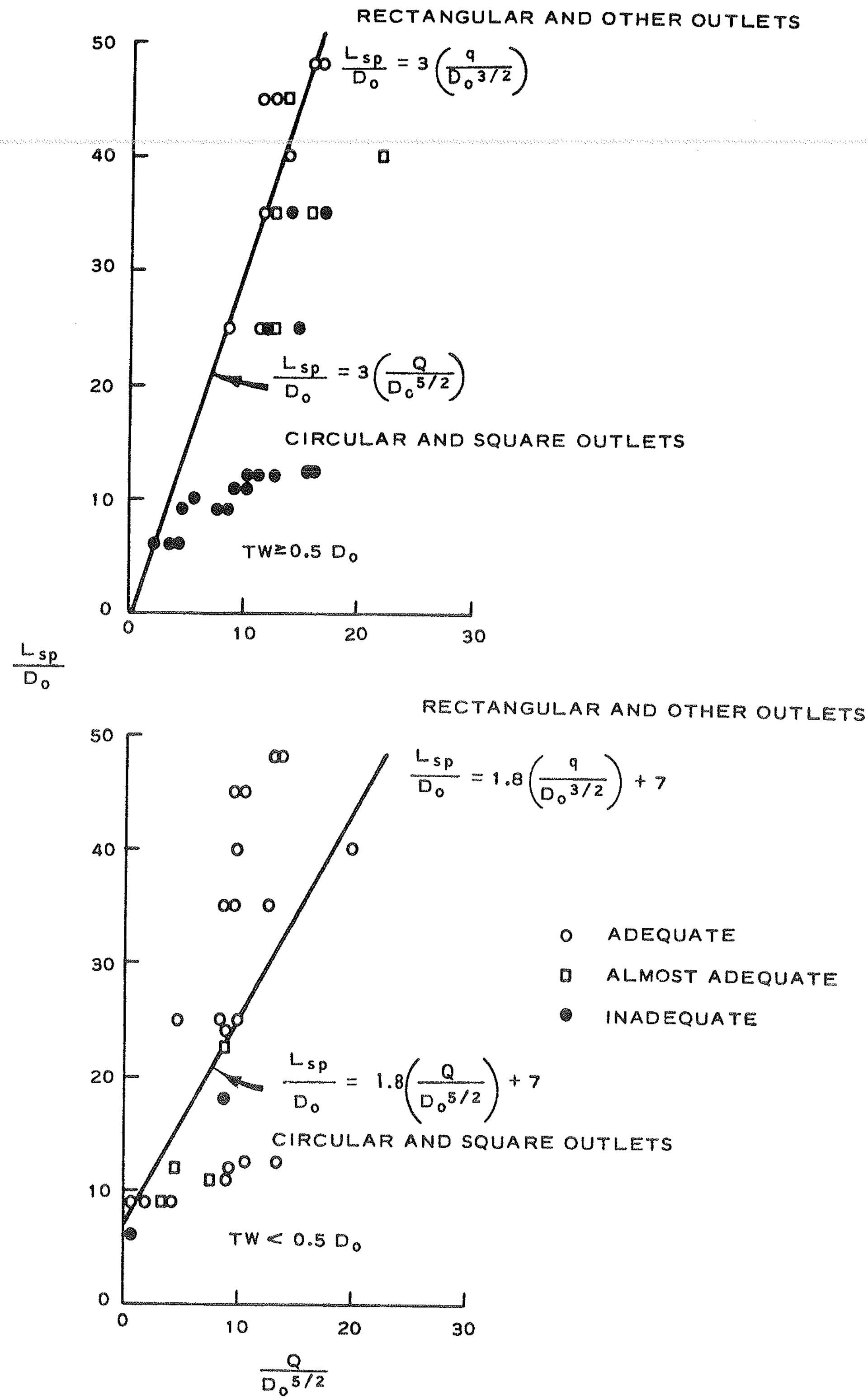


Figure A13. Length of stone protection, horizontal blanket

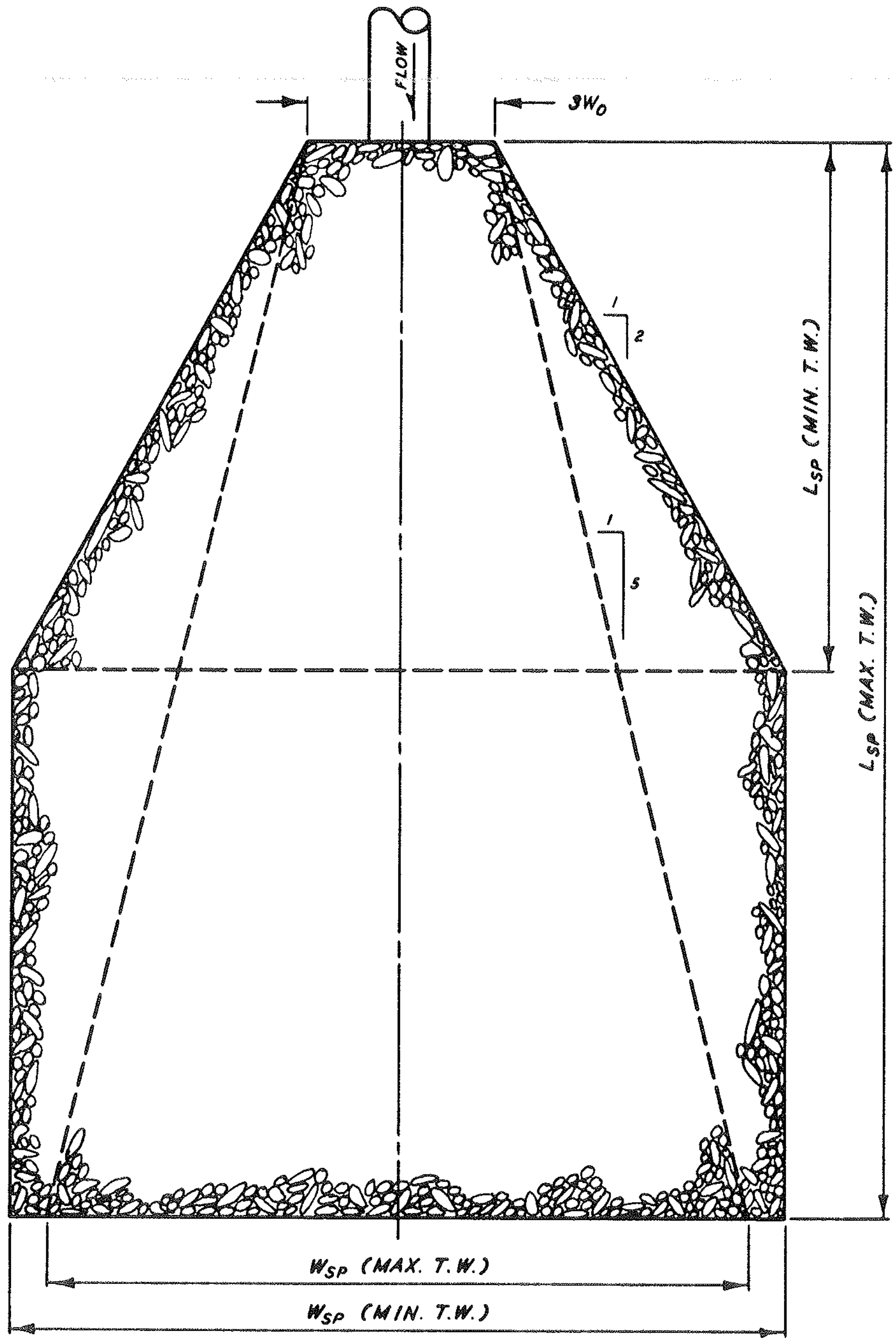


Figure A14. Recommended configuration of riprap blanket subject to minimum and maximum tailwaters

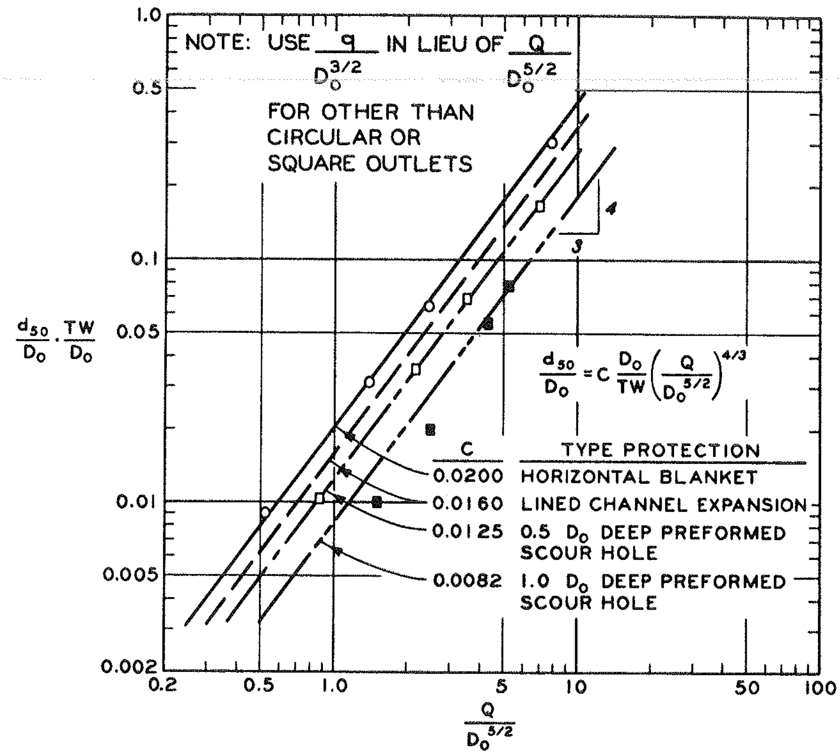


Figure A15. Recommended size of protective stone

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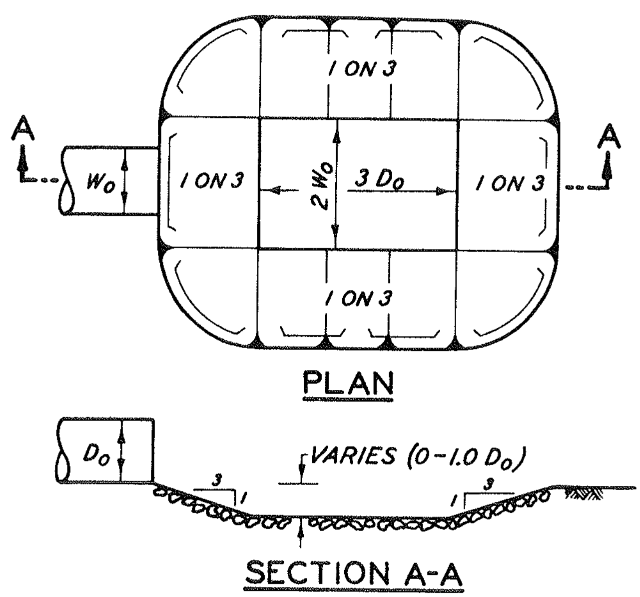


Figure A16. Preformed scour hole

Lined Channel Expansions

12. A research project sponsored by the Louisiana Department of Highways was recently completed at WES to investigate the feasibility of lining channel expansions downstream of square culvert outlets with either sack revetment, cellular blocks, or rock riprap. After observing flow conditions with various sizes of model culverts and geometries of channel expansions, the channel expansion geometry shown in Figure A17 was selected as a practical configuration. The dimensions of the lined channel expansion are related in terms of that of square box culverts.

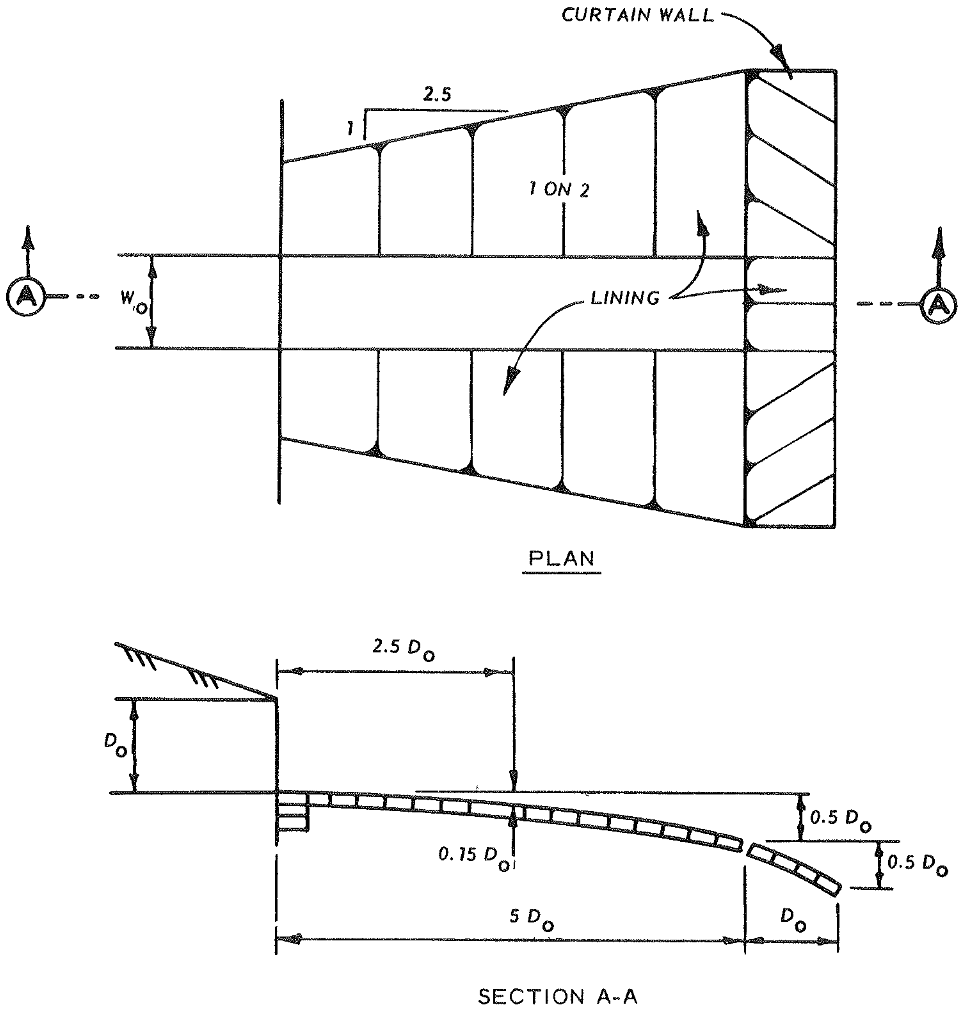


Figure A17. Culvert outlet erosion protection, lined channel expansion

For rectangular outlets, it is recommended that similarity be preserved in both the plan and elevation planes in terms of the respective width and height of the outlet.

13. Sack revetment with length, width, and thickness of 2, 1.5, and 0.33 ft, respectively, and weighing 120 lb was simulated at a scale of 1:8 as shown in Figure A18. Cellular blocks roughly 0.66 by 0.66 ft and 0.33 ft thick weighing 14 lb were simulated at a scale of 1:4 as shown in Figure A19. Rock of 6- to 8-in. diameter weighing 17 lb was simulated at a scale of 1:4 as shown in Figure A20. The results of tests to determine the conditions of discharge and tailwater required to displace or fail each of the revetments are shown in Figure A21 and indicate that the thickness of geometrically similar revetments can be calculated by the means of the following empirical equations:

$$\frac{d_{50}}{D_o} \text{ or } \frac{T_S}{D_o} \text{ or } \frac{T_B}{D_o} = 0.016 \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3} \quad \text{Square and circular outlets} \quad (A3)$$

$$\frac{d_{50}}{D_o} \text{ or } \frac{T_S}{D_o} \text{ or } \frac{T_B}{D_o} = 0.016 \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3} \quad \text{Rectangular and other shaped outlets} \quad (A4)$$

14. The variables are defined in Appendix B. The relative effectiveness of the lined channel expansion relative to the other schemes of riprap protection described previously is shown in Figure A15. The relations presented in Figure A15 are recommended for selection of either the size of revetment for a given scheme of protection, discharge, tailwater depth, and culvert dimension or for the selection of the size of culvert with which a given revetment and scheme of protection will remain stable under anticipated conditions of discharge and tailwater depth.

15. The maximum discharge parameters, $Q/D_o^{5/2}$ or $q/D_o^{3/2}$, of various schemes of protection can be calculated based on the results presented herein and comparisons relative to the cost of each type of

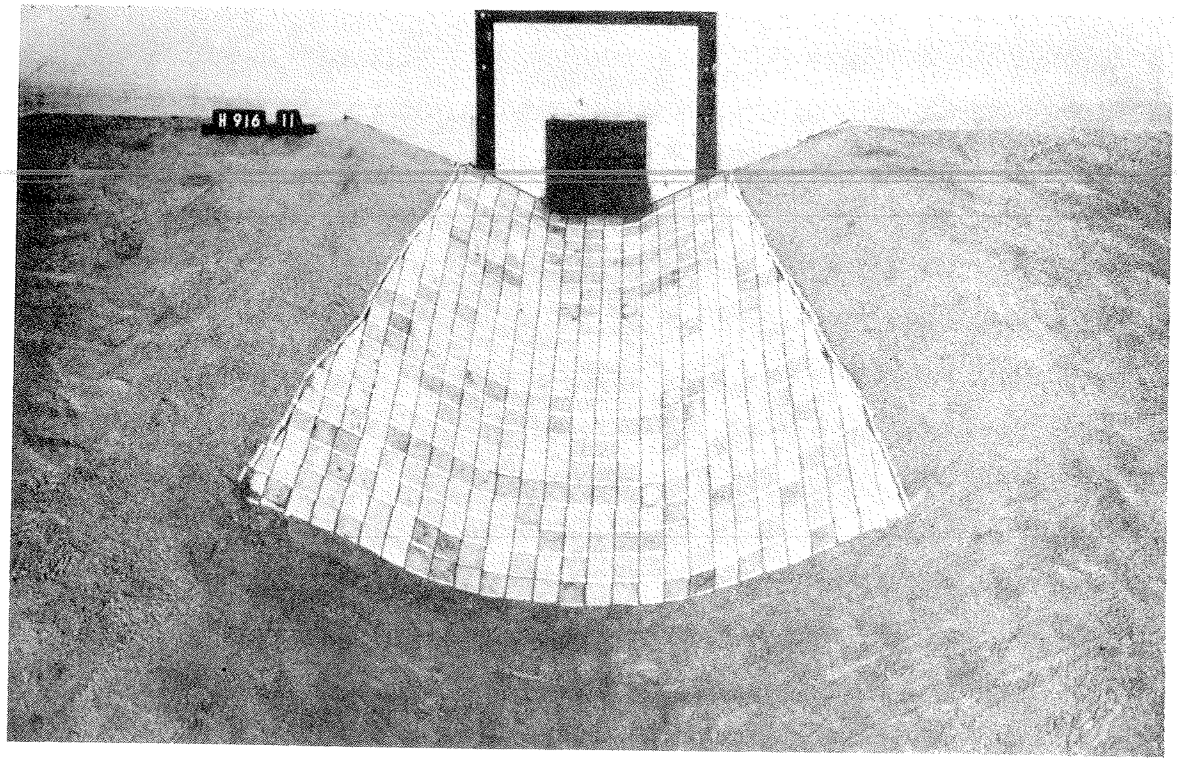


Figure A18. Channel expansion lined with sack revetment

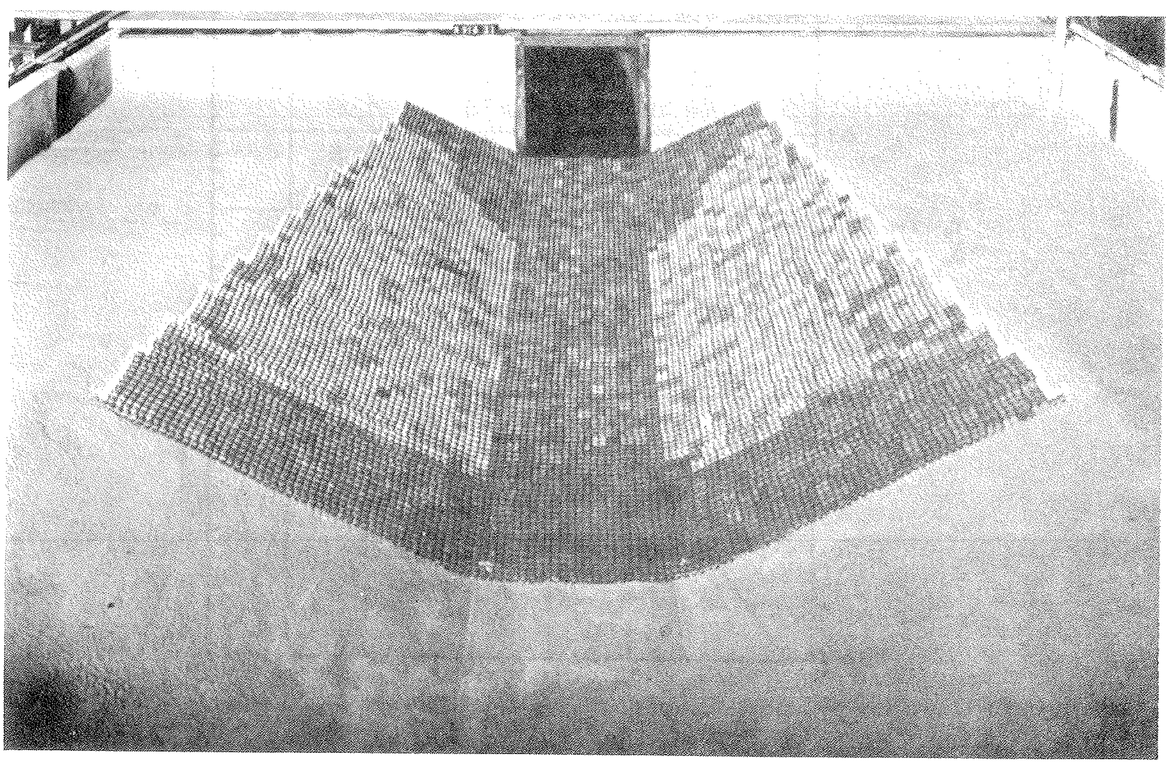


Figure A19. Channel expansion lined with cellular blocks

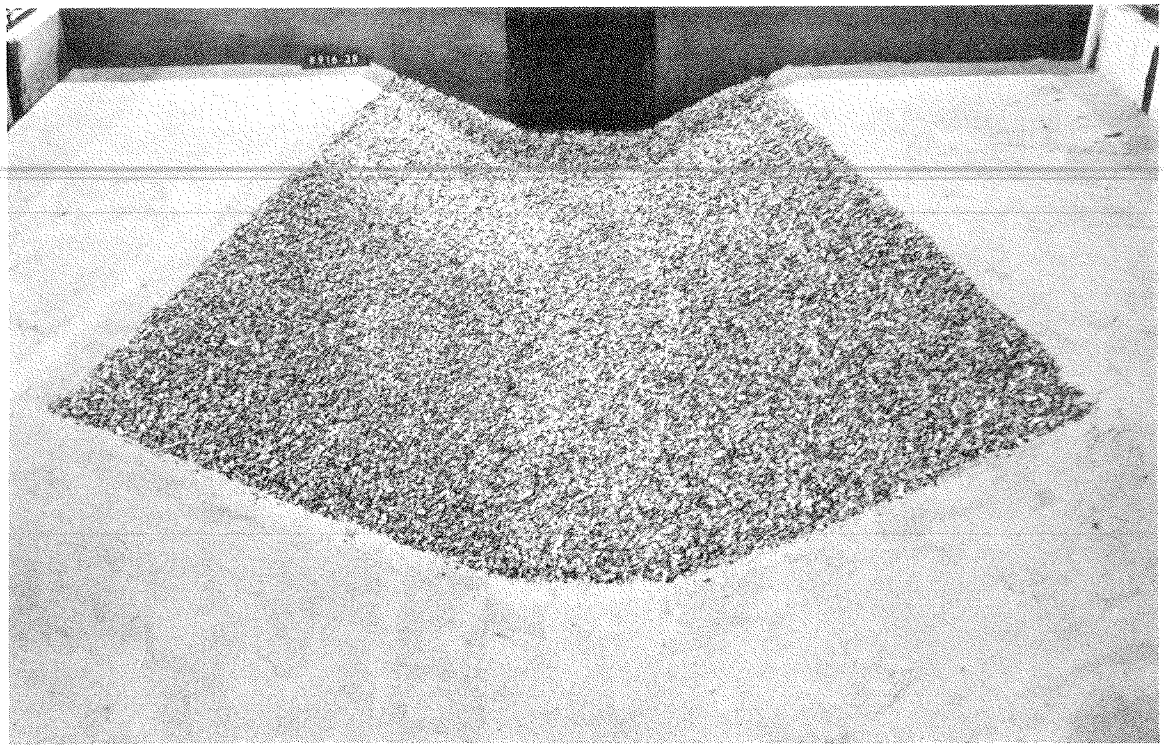


Figure A20. Channel expansion lined with riprap

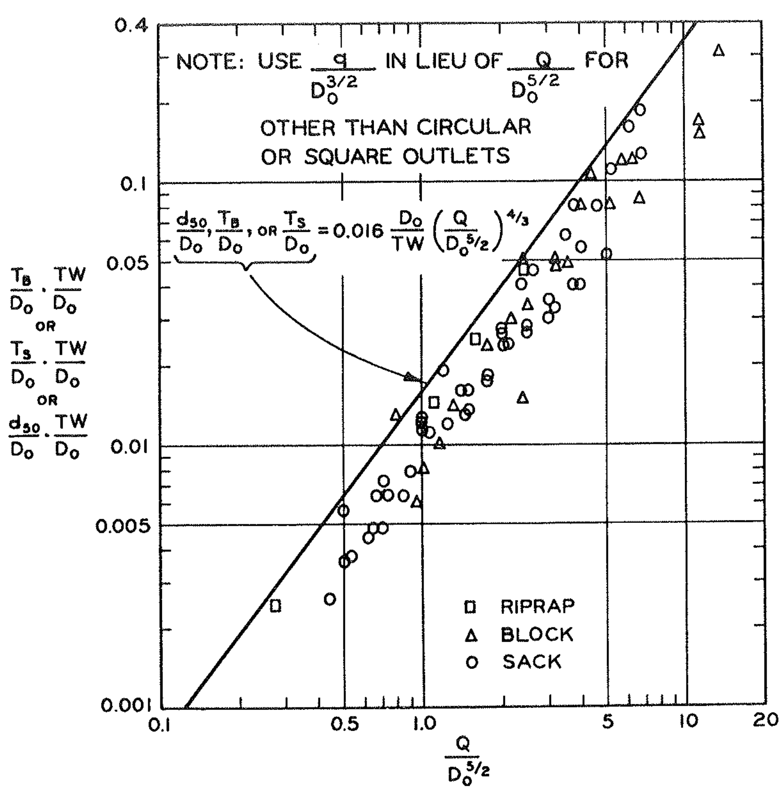


Figure A21. Maximum permissible discharge for lined channel expansions protection can be made to determine the most practical design of providing effective drainage and erosion control facilities for a given site. There will be conditions where the design discharge and economical size of culvert or storm sewer will result in a value of $Q/D_o^{5/2}$ or $q/D_o^{3/2}$, the discharge parameter, greater than the maximum value permissible with feasible schemes of protection discussed previously and some form of energy dissipator will be required. In other cases, the value of the discharge parameter may be less than that of the aforementioned feasible schemes of protection and a simpler more economical form of protection may be indicated.

Flared Outlet Transitions

16. Tests[7] were conducted to determine the maximum values of the discharge parameter (Table A1) that were considered satisfactory with various conditions of tailwater and 3-, 5-, and 8-D_o-long simple flared outlet transitions whose details are shown in Figure A22. Results of the tests of these simple outlet transitions with the apron at the same elevation as the circular culvert invert are shown in Figure A23 which indicate that the maximum discharge parameter for a given outlet, length of transition, and tailwater can be calculated by the equations

205

$$\frac{Q}{D_o^{5/2}} = 1.60 \frac{TW}{D_o} \left(\frac{L}{D_o}\right)^{0.4(D_o/TW)^{1/3}} \qquad \text{Circular and square outlets} \qquad (A5)$$

$$\frac{q}{D_o^{3/2}} = 1.60 \frac{TW}{D_o} \left(\frac{L}{D_o}\right)^{0.4(D_o/TW)^{1/3}} \qquad \text{Rectangular and other shaped outlets} \qquad (A6)$$

Similarly, the length of transition for a given situation can be calculated by the equations

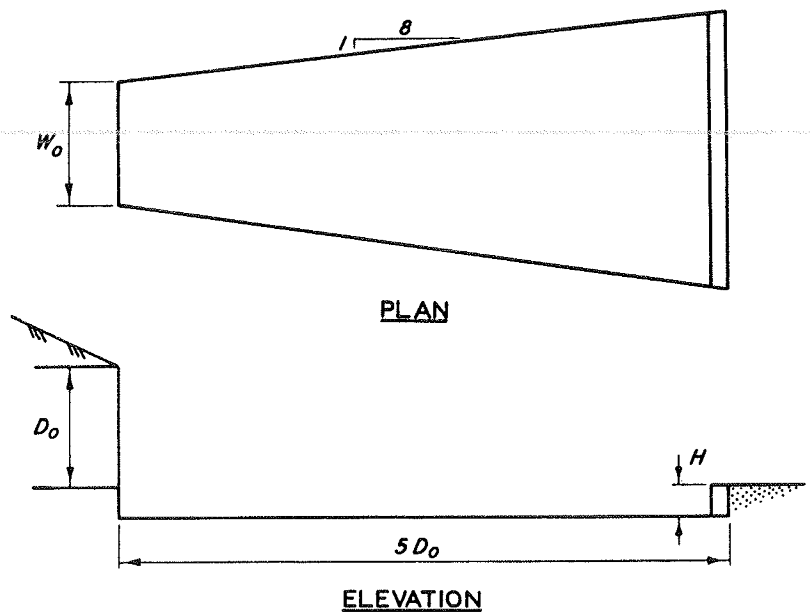


Figure A22. Flared outlet transition

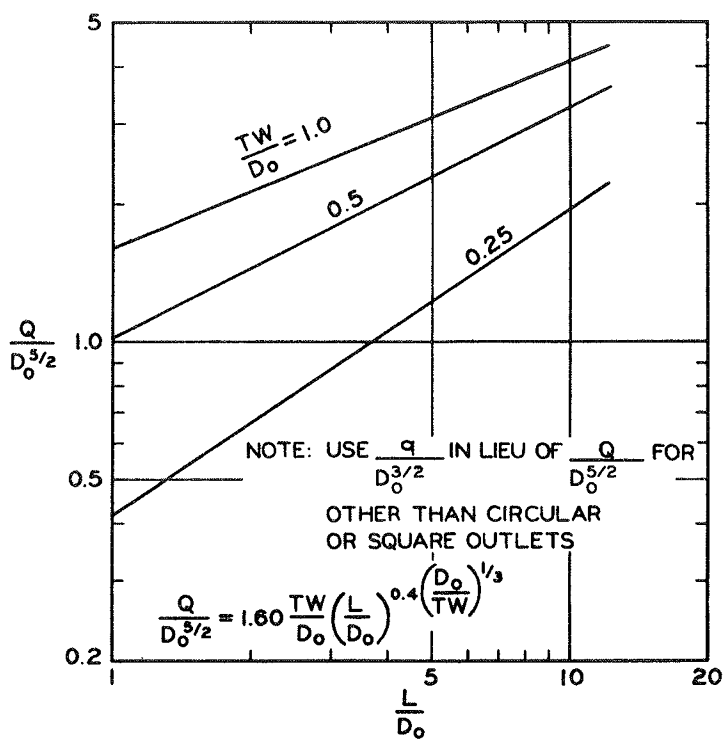


Figure A23. Maximum permissible discharge for various lengths of flared outlet transition and tailwaters

$$\frac{L}{D_o} = 0.30\left(\frac{D_o}{TW}\right)^2 \left(\frac{Q}{D_o^{5/2}}\right)^{2.5(D_o/TW)^{1/3}} \quad \text{Circular and square outlets} \qquad (A7)$$

$$\frac{L}{D_o} = 0.30\left(\frac{D_o}{TW}\right)^2 \left(\frac{q}{D_o^{3/2}}\right)^{2.5(D_o/TW)^{1/3}} \quad \text{Rectangular and other shaped outlets} \qquad (A8)$$

Variables are defined in Appendix B and Figure A23 shows that this type of protection is satisfactory only for low values of $Q/D_o^{5/2}$ or $q/D_o^{3/2}$ The arbitrary extent of scour depth equal to or less than 0.5 D_o was used to classify satisfactory conditions.

17. Attempts were made to investigate the effectiveness of recessing the apron of these flared outlet transitions and providing an end sill at the downstream end; however, Figure A24 indicates that this modification did not significantly improve energy dissipation or increase the applicable maximum values of the discharge parameter, $Q/D_o^{5/2}$ or $q/D_o^{3/2}$ 207

Commonly Used Energy Dissipators

18. Grace and Pickering[8] have reported the results of model tests to evaluate the maximum values of the discharge parameter, $Q/D_o^{5/2}$, applicable to circular culverts discharging into various sizes of three commonly used energy dissipators: stilling wells,[9] U. S. Bureau of Reclamation type VI basins,[10] and St. Anthony Falls stilling basins.[11]

19. The stilling well consists of a vertical section of circular pipe affixed to the outlet end of a storm sewer as shown in Figure A25. The recommended depth of the well below the invert of the incoming pipe is dependent on the slope and diameter of the incoming pipe and can be determined from the plot shown in Figure A25. The recommended height of stilling well above the invert of the incoming pipe is two times the diameter of the incoming pipe. The top of the well should be located at the elevation of the invert of a stable channel or drainage basin.

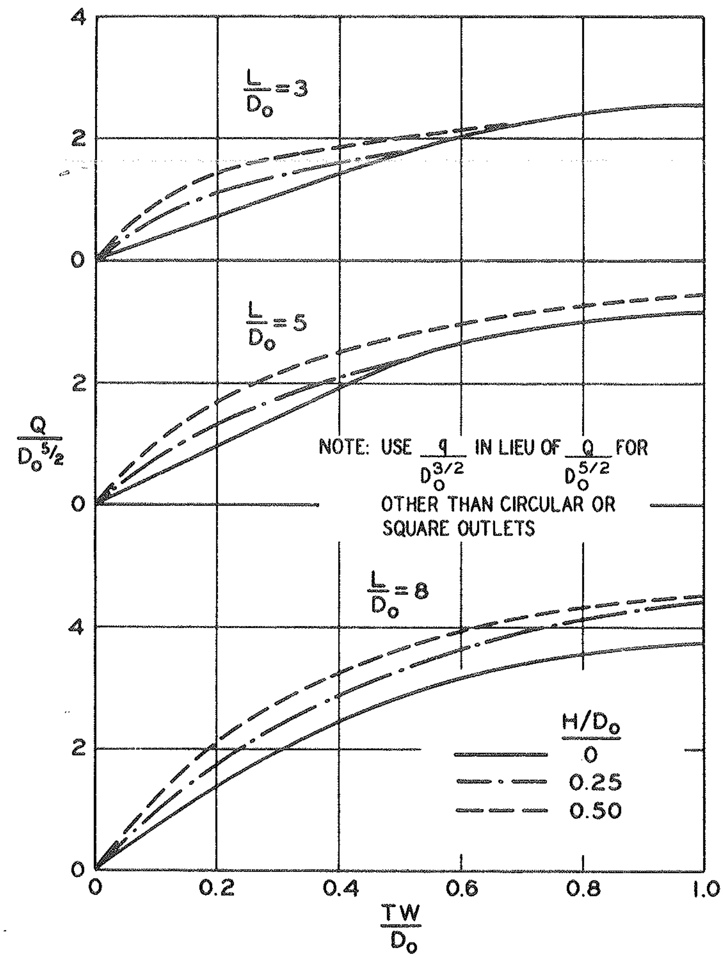


Figure A24. Relative effects of recessed apron and end sill on permissible discharge

The area adjacent to the well may be protected by riprap or paving; however, if there is no adjacent erodible embankment within two well diameters of the periphery of the stilling well, protection is not needed. Energy dissipation is accomplished without the necessity of maintaining a specified tailwater depth in the vicinity of the outlet.

20. Details of the U. S. Bureau of Reclamation type VI basin and the St. Anthony Falls stilling basin are presented in Figures A26 and A27. Maximum values of the discharge parameter, $Q/D_o^{5/2}$, considered satisfactory for various sizes of each of the energy dissipators are presented in Table A2. These data are satisfied by the following equations which can be used to compute the diameter or width of each type

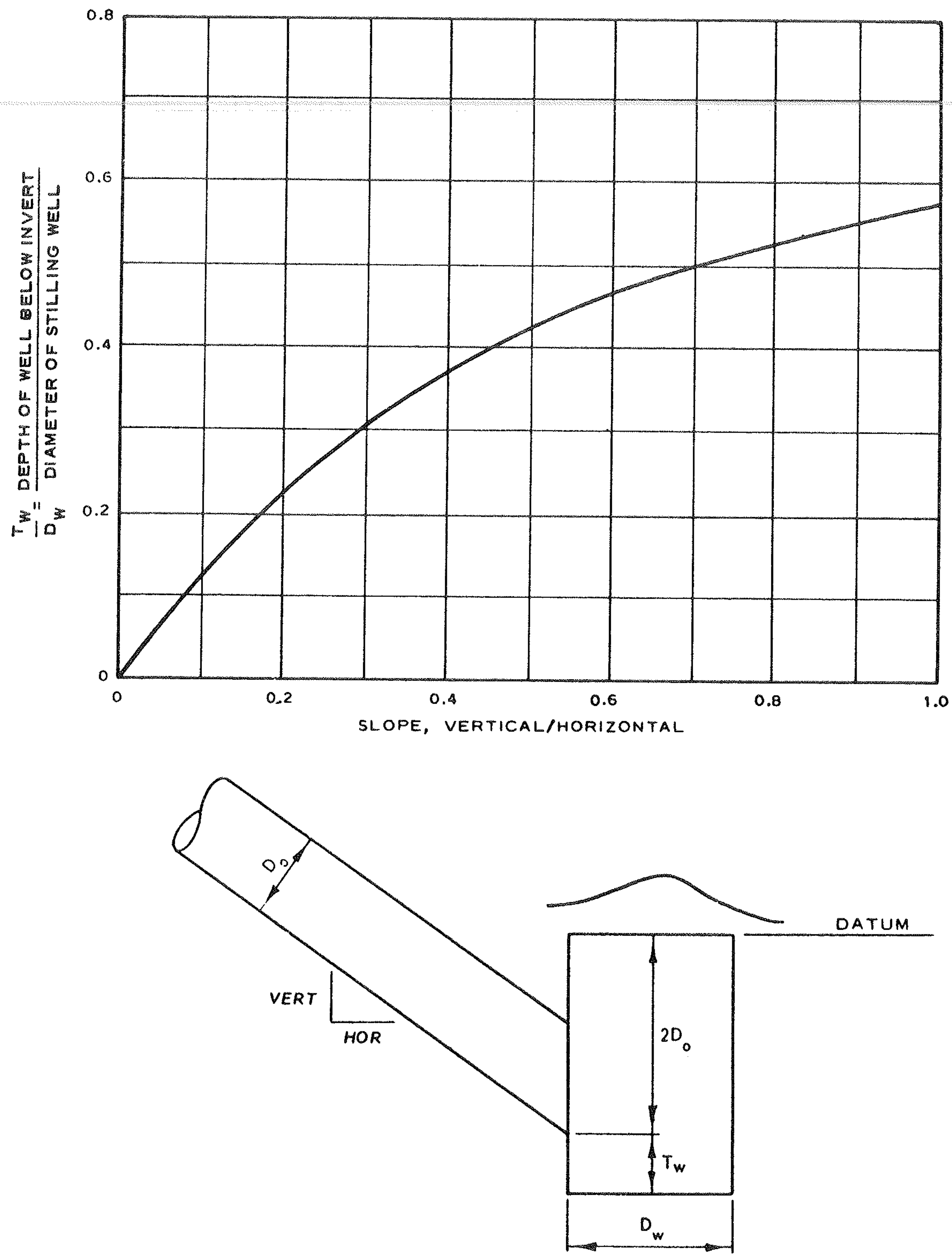
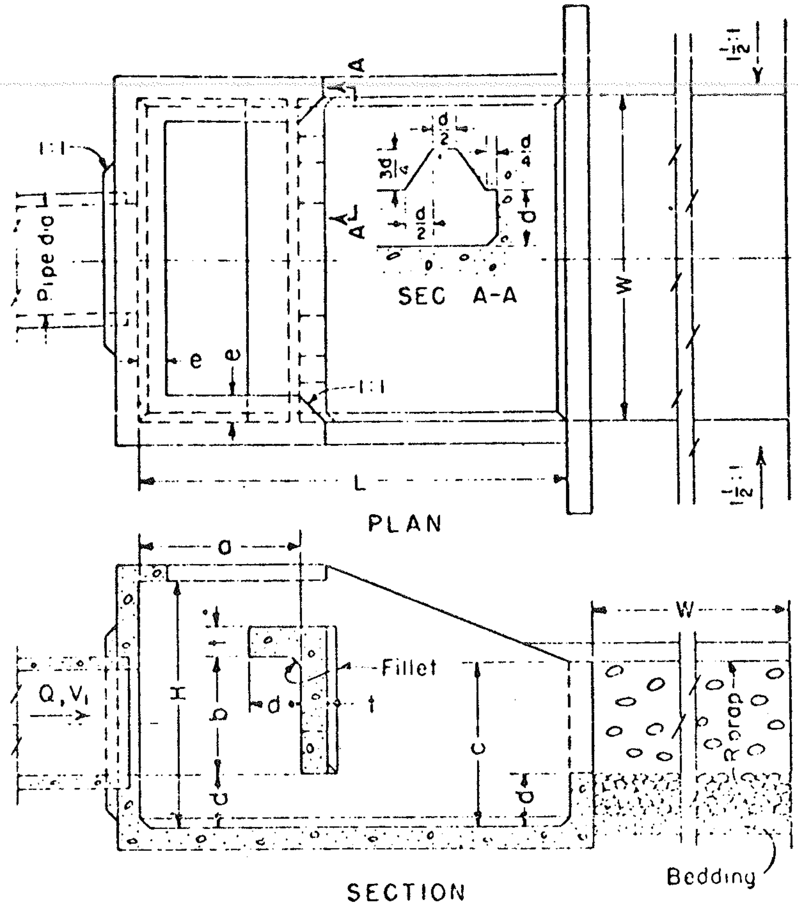


Figure A25. Stilling well

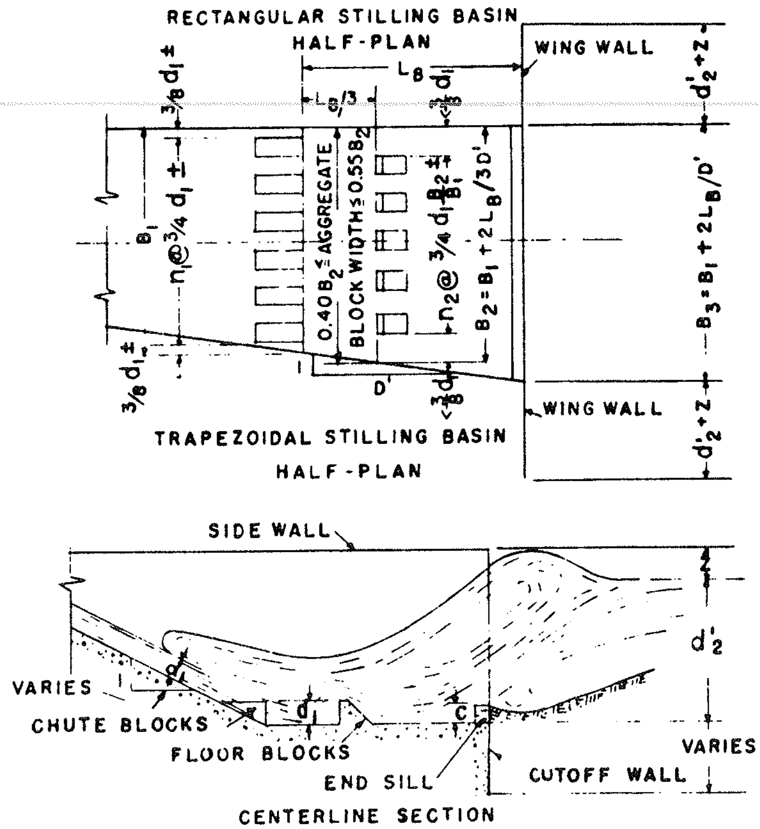


STILLING BASIN DESIGN

H = 3 4 (W)
L = 4/3 (W)
a = 1/2 (W)
b = 3 8 (W)
c = 1/2 (W)

d = 1/6 (W)
e = 1/12 (W)
t = 1/12 (W), SUGGESTED MINIMUM
RIPRAP STONE SIZE DIAMETER = 1/20 (W)

Figure A26. USBR type VI basin



DESIGN EQUATIONS

(1) $F = \frac{V_1^2}{gd_1}$ (2) $d_2 = \frac{d_1}{2}(-1 + \sqrt{8F + 1})$

(3a) F = 3 TO 30 $d'_2 = (1.10 - F/120)\,d_2$

(3b) F = 30 TO 120 $d'_2 = 0.85\,d_2$

(3c) F = 120 TO 300 $d'_2 = (1.00 - F/800)\,d_2$

(4) $L_B = \frac{4.5d_2}{F^{0.38}}$ (5) $Z = \frac{d_2}{3}$ (6) $c = 0.07d_2$

Figure A27. Proportions of SAF stilling basin of energy dissipator relative to that of the incoming circular or square pipe:

$$\frac{D_W}{D_o} = 0.53 \left(\frac{Q}{D_o^{5/2}} \right)^{1.0} \quad \text{Stilling well} \qquad \text{(A9)}$$

$$\frac{W_{SAF}}{D_o} = 0.30 \left(\frac{Q}{D_o^{5/2}} \right)^{1.0} \quad \text{St. Anthony Falls stilling basin} \qquad \text{(A10)}$$

$$\frac{W_{VI}}{D_o} = 1.30 \left(\frac{Q}{D_o^{5/2}} \right)^{0.55} \quad \text{U. S. Bureau of Reclamation type VI basin} \qquad \text{(A11)}$$

The above relations should be used only for design of each of the respective energy dissipators downstream of circular or square outlets. The SAF stilling basin is the only one of the above energy dissipators recommended for use with other shaped outlets, and in such cases, the
212
design should be conducted in accordance with the usual procedures for ensuring the formation of a hydraulic jump within the stilling basin rather than based on the above relation. It is recommended that the size of stone protection to be provided downstream of these energy dissipators be estimated by the following relation:

$$\frac{d_{50}}{D_e} = 1.0 \left(\frac{V_e}{\sqrt{gD_e}} \right)^3 \qquad \text{(A12)}$$

where D_e and V_e are the depth and velocity of flow exiting the energy dissipator. Guidance other than engineering judgment for estimating the length of stone protection required downstream of an energy dissipator is not available due to the lack of systematic investigations of this aspect of the problem. However, model studies of protection required downstream of spillway stilling basins indicate that a length of approximately 10 times the theoretical depth of flow required for a hydraulic jump is reasonably adequate.

Discussion

21. Contrary to the usual assumption, increased tailwater or excessive tailwater at outlets tends to concentrate rather than diffuse the efflux; and although the depth of scour may not be as severe, the length of scour relative to that observed with tailwaters less than one-half the height of the outlet is considerably greater. This is attributed to the fact that with tailwaters greater than or equal to one-half the outlet height, the efflux is confined by the relatively stagnant adjacent waters which are entrained with the efflux to effectively increase the unit discharge issue from the outlet.

22. Although the effect of outlet shape on the scour hole geometry was not investigated in detail, a comparison of the scour holes developed in 0.25-mm sand by a discharge of 0.87 cfs through each of four differently shaped outlets (circular, square, rectangular, and arch) with the same cross-sectional area (0.087 sq ft) and both minimum and maximum tailwater conditions indicated that outlet shape had no significant effect on the scour hole geometry. The tendency of the jet issued from an outlet to oscillate from side to side under conditions of maximum tailwater was observed with flows through each of the aforementioned conduit shapes. This oscillation was random and quite slow for all conditions except when flow from the arch-shaped outlet was discharged into maximum tailwaters after a scour hole had been developed with minimum tailwaters. For this condition, the oscillation was periodic and changed position about every 15 sec. Thus, it appears that a jet discharge from an arch-shaped outlet is less stable than those from the other outlet shapes investigated. This indicates that a greater extent of scour, particularly width of scour, may be expected downstream of arch outlets subject to both minimum and maximum tailwaters (see Figure A4).

23. Various degrees of success have been experienced with riprap and/or rubble or other forms of protection downstream of outlets and different opinions regarding the adequacy of protective stone have developed. One of the most common causes of failure of protective

material observed during field observations[12] was the lack of an adequate filter between the soil and the protective material. This permits progressive leaching of the soil and settlement of the blanket. The blanket can be grouted in areas subject to mild winters; however, an appropriate filter and weep holes should be provided for relief of hydrostatic pressure. Grouted riprap does not perform satisfactorily in areas where considerable freezing and thawing is experienced annually. Exit channel protection should be segregated from erodible soils by graded filters[13] and/or durable synthetic cloths.[14]

24. It is considered that the results presented herein, with the exception of the three commonly used energy dissipators which were developed for circular and square outlets, can be applied to other outlet shapes, provided geometric similarity is preserved in application of the recommended guidance. The discharge parameter should be calculated on the basis of the unit discharge per foot of width of the outlet, q, rather than the total discharge.

214

25. These results may also be applied to develop designs of protective measures downstream of multiple outlets, provided the spacing between outlets is relatively small (less than one-fourth the individual outlet widths). In such cases, it is recommended that analyses be conducted on the basis of a single outlet (one of the two outermost outlets) and that a total width of protection be provided which includes the total width of protection needed below a single outlet plus the width between the center lines of the two outermost outlets. If the spacing between outlets is appreciable, i.e. one-fourth or greater than the individual outlet widths, the individual jets and unit discharges of flow may be concentrated due to confinement by excessive tailwater or expansion and subsequent intersection downstream with minimum tailwater; and considerable turbulence may be generated which will increase the severity of attack on local boundaries. In such cases, it is recommended that the extent of the protective works be enlarged by a factor of judgment, i.e. 25 to 33 percent.

26. These generalized results offer considerable guidance since one can estimate the extent of scour to be anticipated in stable

channels of cohesionless soils and then decide what degree of protection is required. For example, is the anticipated scour hole with an appropriate cutoff wall that protects the outlet adequate for energy dissipation? Are the size and extent of riprap required for a stable horizontal blanket practicable? Is it practicable to compromise depth of scour and size of riprap by providing a preformed and riprap-lined scour hole? Is an energy dissipator required? Is it practicable to size the storm sewer or culvert on the basis of anticipated erosion and appropriate protective measures in lieu of hydraulic efficiency? Examples of the recommended application of the results are presented in Table A3.

Table A1

Maximum Discharge Recommended for Various Flared Outlet Transitions

Limiting Values of $Q/D_o^{5/2}$			
L/D_o	H/D_o	TW/D_o	$Q/D_o^{5/2}$
3	0	0	0.88
3	0	0.50	1.78
3	0	1.00	2.56
3	0.25	0.25	1.28
3	0.25	0.50	1.78
3	0.25	1.00	2.56
3	0.50	0.25	1.58
3	0.50	0.50	2.00
3	0.50	1.00	2.56
5	0	0.25	1.20
5	0	0.50	2.40
5	0	1.00	3.20
5	0.25	0.25	1.58
5	0.25	0.50	2.78
5	0.25	1.00	3.47
5	0.50	0.25	1.47
5	0.50	0.50	2.77
5	0.50	1.00	3.46
8	0	0.25	1.68
8	0	0.50	2.40
8	0	1.00	3.75
8	0.25	0.25	2.17
8	0.25	0.50	3.36
8	0.25	1.00	4.44
8	0.50	0.25	2.46
8	0.50	0.50	3.65
8	0.50	1.00	4.55

Table A2

Maximum Discharge Recommended for Various Types and Sizes of Energy Dissipators

Relative Width and Type of Energy Dissipator	Maximum $Q/D_o^{5/2}$
Stilling Well	
1 D_o diameter	2.0
2 D_o diameter	3.5
3 D_o diameter	5.0
5 D_o diameter	10.0
USBR Type VI Basin	
1 D_o wide	0.6
2 D_o wide	2.2
3 D_o wide	4.5
4 D_o wide	7.6
5 D_o wide	11.5
7 D_o wide	21.0
SAF Stilling Basin	
1 D_o wide	3.5
2 D_o wide	7.0
3 D_o wide	9.5

Table A3

Examples of recommended application to estimate extent of scour in a cohesionless soil and alternative schemes of protection required to prevent local scour downstream of a circular and rectangular outlet with equivalent cross-sectional areas that will be subjected to a range of discharges for a duration of one hour.

Given:

Dimensions of rectangular outlet = W_o = 10 ft, D_o = 5 ft

Diameter of circular outlet, D_o = 8 ft

Range of discharge, Q = 362 to 1086 cfs

Discharge parameter for rectangular culvert, $q/D_o^{3/2}$ = 3.2 to 9.7

Discharge parameter for circular culvert, $Q/D_o^{5/2}$ = 2 to 6

Duration of runoff event, t = 60 min

Maximum tailwater el = 6.4 ft above outlet invert (>0.5 D_o)

Minimum tailwater el = 2.0 ft above outlet invert (<0.5 D_o)

Example 1 - Determine maximum depth of scour for minimum and maximum flow conditions:

RECTANGULAR CULVERT (see Figure A7)

MINIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.80\left(\frac{q}{D_o^{3/2}}\right)^{0.375} t^{0.10}$$

$$D_{sm} = 0.80\ (3.2 - 9.7)^{0.375}\ (60)^{0.1}\ (5) = \underline{\underline{9.3\text{ ft}}} - \underline{\underline{14.0\text{ ft}}}$$

(Continued)

(Sheet 1 of 11)

Table A3 (Continued)

MAXIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.74 \left(\frac{q}{D_o^{3/2}}\right)^{0.375} t^{0.10}$$

$$D_{sm} = 0.74\ (3.2 - 9.7)^{0.375}\ (60)^{0.1}\ (5) = \underline{\underline{8.6\ \text{ft}}} - \underline{\underline{13.0\ \text{ft}}}$$

CIRCULAR CULVERT (see Figure A7)

MINIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.80 \left(\frac{Q}{D_o^{5/2}}\right)^{0.375} t^{0.10}$$

$$D_{sm} = 0.80\ (2 - 6)^{0.375}\ (60)^{0.1}\ (8) = \underline{\underline{12.5\ \text{ft}}} - \underline{\underline{18.9\ \text{ft}}}$$

MAXIMUM TAILWATER

$$\frac{D_{sm}}{D_o} = 0.74 \left(\frac{Q}{D_o^{5/2}}\right)^{0.375} t^{0.1}$$

$$D_{sm} = 0.74\ (2 - 6)^{0.375}\ (60)^{0.1}\ (8) = \underline{\underline{11.6\ \text{ft}}} - \underline{\underline{17.5\ \text{ft}}}$$

<u>Example 2 - Determine maximum width of scour for minimum and maximum flow conditions:</u>

RECTANGULAR CULVERT (see Figure A8)

MINIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 1.00 \left(\frac{q}{D_o^{3/2}}\right)^{0.915} t^{0.15}$$

$$W_{sm} = 1.00\ (3.2 - 9.7)^{0.915}\ (60)^{0.15}\ (5) = 27\ \text{ft} - 74\ \text{ft}$$

(Continued)

(Sheet 2 of 11)

Table A3 (Continued)

$$W_{smr} = W_{sm} + \frac{W_o}{2} - \frac{D_o}{2} = (27 - 74) + \frac{10}{2} - \frac{5}{2} = \underline{\underline{29.5\ ft}} - \underline{\underline{76.5\ ft}}$$

MAXIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 0.72 \left(\frac{q}{D_o^{3/2}}\right)^{0.915} t^{0.15}$$

$$W_{sm} = 0.72\ (3.2 - 9.7)^{0.915}\ (60)^{0.015} = 19\ ft - 53\ ft$$

$$W_{smr} = W_{sm} + \frac{W_o}{2} - \frac{D_o}{2} = (19 - 53) + \frac{10}{2} - \frac{5}{2} = \underline{\underline{21.5\ ft}} - \underline{\underline{55.5\ ft}}$$

CIRCULAR CULVERT (see Figure A8)

MINIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 1.00 \left(\frac{Q}{D_o^{5/2}}\right)^{0.915} t^{0.15}$$

$$W_{sm} = 1.00\ (2 - 6)^{0.915}\ (60)^{0.15}\ (8) = \underline{\underline{28\ ft}} - \underline{\underline{76\ ft}}$$

MAXIMUM TAILWATER

$$\frac{W_{sm}}{D_o} = 0.72 \left(\frac{Q}{D_o^{5/2}}\right)^{0.915} t^{0.15}$$

$$W_{sm} = 0.72\ (2 - 6)^{0.915}\ (60)^{0.15}\ (8) = \underline{\underline{20\ ft}} - \underline{\underline{55\ ft}}$$

(Continued)

(Sheet 3 of 11)

Table A3 (Continued)

Example 3 - Determine maximum length of scour for minimum and maximum flow conditions:

RECTANGULAR CULVERT (see Figure A9)

MINIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 2.40 \left(\frac{q}{D_o^{3/2}}\right)^{0.71} t^{0.125}$$

$$L_{sm} = 2.4\ (3.2 - 9.7)^{0.71}\ (60)^{0.125}\ (5) = \underline{\underline{46\ ft}} - \underline{\underline{101\ ft}}$$

MAXIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 4.10 \left(\frac{q}{D_o^{3/2}}\right)^{0.71} t^{0.125}$$

$$L_{sm} = 4.10\ (3.2 - 9.7)^{0.71}\ (60)^{0.125}\ (5) = \underline{\underline{78\ ft}} - \underline{\underline{171\ ft}}$$

CIRCULAR CULVERT (see Figure A9)

MINIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 2.40 \left(\frac{Q}{D_o^{5/2}}\right)^{0.71} t^{0.125}$$

$$L_{sm} = 2.4\ (2 - 6)^{0.71}\ (60)^{0.125}\ (8) = \underline{\underline{52\ ft}} - \underline{\underline{114\ ft}}$$

MAXIMUM TAILWATER

$$\frac{L_{sm}}{D_o} = 4.10 \left(\frac{Q}{D_o^{5/2}}\right)^{0.71} t^{0.125}$$

$$L_{sm} = 4.10\ (2 - 6)^{0.71}\ (60)^{0.125}\ (8) = \underline{\underline{90\ ft}} - \underline{\underline{195\ ft}}$$

(Continued)

(Sheet 4 of 11)

Table A3 (Continued)

Example 4 - Determine profile and cross section of scour for maximum discharge and minimum tailwater conditions (see Figure A11):

CIRCULAR CULVERT

For L_{sm} = 114 ft and D_{sm} = 18.9 ft

L_s/L_{sm}	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
L	0.0	11.4	22.8	34.2	45.6	57.0	68.4	79.8	91.2	102.6	114.0
D_s/D_{sm}	0.7	0.75	0.85	0.95	1.0	0.95	0.75	0.55	0.33	0.15	0.0
D_s	13.2	14.2	16.1	18.0	18.9	18.0	14.2	10.4	6.3	2.9	0.0

For W_{sm} = 76 ft and D_{sm} = 18.9 ft

W_s/W_{sm}	0.0	0.2	0.4	0.6	0.8	1.0
W_s	0.0	15.2	30.4	45.6	60.8	76.0
D_s/D_{sm}	1.0	0.67	0.27	0.15	0.05	0.0
D_s	18.9	12.6	5.1	2.8	0.95	0.0

RECTANGULAR CULVERT

For L_{sm} = 101 ft and D_{sm} = 14.0 ft

L_s/L_{sm}	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
L	0.0	10.1	20.2	30.3	40.4	50.5	60.6	70.7	80.8	90.9	101.0
D_s/D_{sm}	0.7	0.75	0.85	0.95	1.0	0.95	0.75	0.55	0.33	0.15	0.0
D_s	9.8	10.5	11.9	13.3	14.0	13.3	10.5	7.7	4.6	2.1	0.0

For W_{sm} = 74 ft and D_{sm} = 14.0 ft

W_s/W_{sm}	0.0	0.2	0.4	0.6	0.8	1.0
W_s	0.0	14.8	29.6	44.4	59.2	74.0
D_s/D_{sm}	1.0	0.67	0.27	0.15	0.05	0.0
D_s	14.0	9.38	3.78	2.10	0.70	0.0
$W_{sr} = W_s + \frac{W_o}{2} - \frac{D_o}{2}$	0-2.5	17.3	32.1	46.9	61.7	76.5

(Continued)

(Sheet 5 of 11)

Table A3 (Continued)

Example 5 - Determine depth and width of cutoff wall:

RECTANGULAR CULVERT, Maximum depth and width of scour = 14 ft and 76.5 ft

From Figure A11, depth of cutoff wall = $0.7\ (D_{sm}) = 0.7\ (14) = \underline{\underline{9.8\ \text{ft}}}$

From Figure A11, width of cutoff wall = $2\ (W_{smr}) = 2\ (76.5) = \underline{\underline{153\ \text{ft}}}$

CIRCULAR CULVERT, Maximum depth and width of scour = 18.9 ft and 76.0 ft

From Figure A11, depth of cutoff wall = $0.7\ (D_{sm}) = 0.7\ (18.9) = \underline{\underline{13.2\ \text{ft}}}$

From Figure A11, width of cutoff wall = $2\ (W_{sm}) = 2\ (76) = \underline{\underline{152\ \text{ft}}}$

Note: The depth of cutoff wall may be varied with width in accordance with the cross section of the scour hole at the location of the maximum depth of scour, see Figures A11 and A12.

Example 6 - Determine size and extent of horizontal blanket of riprap:

RECTANGULAR CULVERT

MINIMUM TAILWATER

From Figure A15, $$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3}$$

$$d_{50} = 0.020\ (5/2)(3.2 - 9.7)^{4/3}\ (5) = \underline{\underline{1.2\ \text{ft}}} - \underline{\underline{5.2\ \text{ft}}}$$

From Figure A13, $$\frac{L_{sp}}{D_o} = 1.8\left(\frac{q}{D_o^{3/2}}\right) + 7$$

$$L_{sp} = [1.8\ (3.2 - 9.7) + 7]\ 5 = \underline{\underline{64\ \text{ft}}} - \underline{\underline{122\ \text{ft}}}$$

MAXIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3}$$

(Continued)

(Sheet 6 of 11)

Table A3 (Continued)

$$d_{50} = 0.020\ (5/6.4)\ (3.2 - 9.7)^{4/3}\ (5) = \underline{\underline{0.37\ ft}} - \underline{\underline{0.76\ ft}}$$

$$\frac{L_{sp}}{D_o} = 3\ \left(\frac{q}{D_o^{3/2}}\right)$$

$$L_{sp} = 3\ (3.2 - 9.7)\ 5 = \underline{\underline{48\ ft}} - \underline{\underline{145\ ft}}$$

CIRCULAR CULVERT

MINIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3}$$

$$d_{50} = 0.020\ (8/2)\ (2 - 6)^{4/3}\ (8) = \underline{\underline{1.6\ ft}} - \underline{\underline{7.0\ ft}}$$

$$\frac{L_{sp}}{D_o} = 1.8\left(\frac{Q}{D_o^{5/2}}\right) + 7$$

$$L_{sp} = [1.8\ (2 - 6) + 7]\ 8 = \underline{\underline{85\ ft}} - \underline{\underline{142\ ft}}$$

MAXIMUM TAILWATER

$$\frac{d_{50}}{D_o} = 0.020\ \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3}$$

$$d_{50} = 0.020\ (8/6.4)\ (2 - 6)^{4/3}\ (8) = 0.50\ ft - 2.18\ ft$$

$$\frac{L_{sp}}{D_o} = 3\ \left(\frac{Q}{D_o^{5/2}}\right)$$

(Continued)

(Sheet 7 of 11)

Table A3 (Continued)

$$L_{sp} = 3\ (2 - 6)\ 8 = \underline{\underline{48\ \text{ft}}} - \underline{\underline{144\ \text{ft}}}$$

Use Figure A14 to determine recommended configuration of horizontal blanket of riprap subject to minimum and maximum tailwaters.

Example 7 - Determine size and geometry of riprap-lined preformed scour holes 0.5- and 1.0-D_o deep for minimum tailwater conditions:

RECTANGULAR CULVERT (see Figure A15)

0.5-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0125\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3}$$

$$d_{50} = 0.0125\ (5/2)\ (3.2 - 9.7)^{4/3}\ (5) = \underline{\underline{0.73\ \text{ft}}} - \underline{\underline{3.2\ \text{ft}}}$$

1.0-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0082\ \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3}$$

$$d_{50} = 0.0082\ (8/2)\ (2 - 6)^{4/3}\ (8) = \underline{\underline{0.66\ \text{ft}}} - \underline{\underline{2.9\ \text{ft}}}$$

CIRCULAR CULVERT

0.5-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0125\ \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3}$$

$$d_{50} = 0.0125\ (8/2)\ (2 - 6)^{4/3}\ (8) = \underline{\underline{1.0\ \text{ft}}} - \underline{\underline{4.4\ \text{ft}}}$$

(Continued)

(Sheet 8 of 11)

Table A3 (Continued)

1.0-D_o-DEEP RIPRAP-LINED PREFORMED SCOUR HOLE

$$\frac{d_{50}}{D_o} = 0.0082 \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3}$$

$$d_{50} = 0.0082\ (8/2)(2 - 6)^{4/3}\ (8) = \underline{\underline{0.66\ ft}} - \underline{\underline{2.9\ ft}}$$

See Figure A16 for geometry.

<u>Example 8 - Determine size and geometry of riprap-lined-channel expansion for minimum tailwaters (see Figure A21):</u>

RECTANGULAR CULVERT

$$\frac{d_{50}}{D_o} = 0.016 \frac{D_o}{TW}\left(\frac{q}{D_o^{3/2}}\right)^{4/3}$$

$$d_{50} = 0.016\ (5/2)\ (3.2 - 9.7)^{4/3}\ (5) = \underline{\underline{0.94\ ft}} - \underline{\underline{4.1\ ft}}$$

CIRCULAR CULVERT

$$\frac{d_{50}}{D} = 0.016 \frac{D_o}{TW}\left(\frac{Q}{D_o^{5/2}}\right)^{4/3}$$

$$d_{50} = 0.016\ (5/2)\ (2 - 6)^{4/3}\ (8) = \underline{\underline{0.81\ ft}} - \underline{\underline{3.5\ ft}}$$

See Figure A17 for geometry.

<u>Example 9 - Determine length and geometry of a flared outlet transition for minimum tailwaters:</u>

RECTANGULAR CULVERT

$$\frac{L}{D_o} = 0.30 \left(\frac{D_o}{TW}\right)^2 \left(\frac{q}{D_o^{3/2}}\right)^{2.5(TW/D_o)^{1/3}}$$

$$L = \left[0.3\ (5/2)^2\ (3.2 - 9.7)^{2.5(2/5)^{1/3}}\right] 5 = \underline{\underline{80\ ft}} - \underline{\underline{616\ ft}}$$

(Continued)

(Sheet 9 of 11)

Table A3 (Continued)

CIRCULAR CULVERT

$$\frac{L}{D_o} = 0.30\left(\frac{D_o}{TW}\right)^2 \left(\frac{Q}{D_o^{5/2}}\right)^{2.5(TW/D_o)^{1/3}}$$

$$L = \left[0.3\ (8/2)^2\ (2 - 6)^{2.5(2/8)^{1/3}}\right] 8 = \underline{\underline{114\text{ ft}}} - \underline{\underline{645\text{ ft}}}$$

See Figure A22 for geometric details; above equations developed for H = 0 or horizontal apron at outlet invert elevation without an end sill.

<u>Example 10 - Determine diameter of stilling well required downstream of the 8-ft-diam outlet:</u>

From page A27
$$\frac{D_W}{D_o} = 0.53\left(\frac{Q}{D_o^{5/2}}\right)^{1.0}$$

$$D_W = 0.53\ (2 - 6)\ 8 = \underline{\underline{8.5\text{ ft}}} - \underline{\underline{25.4\text{ ft}}}$$

See Figure A25 for additional dimensions.

<u>Example 11 - Determine width of USBR type VI basin required downstream of the 8-ft-diam outlet:</u>

From page A27
$$\frac{W_{VI}}{D_o} = 1.30\left(\frac{Q}{D_o^{5/2}}\right)^{0.55}$$

$$W_{VI} = \left[1.3\ (2 - 6)^{0.55}\right] 8 = \underline{\underline{15.2\text{ ft}}} - \underline{\underline{27.9\text{ ft}}}$$

See Figure A26 for additional dimensions.

<u>Example 12 - Determine width of SAF basin required downstream of the 8-ft-diam outlet</u>

From page A27
$$\frac{W_{SAF}}{D_o} = 0.30\left(\frac{Q}{D_o^{5/2}}\right)^{1.0}$$

(Continued)

(Sheet 10 of 11)

Table A3 (Concluded)

$$W_{SAF} = 0.30\ (2 - 6)\ 8 = \underline{\underline{4.8\ ft}} - \underline{\underline{14.4\ ft}}$$

See Figure A27 for additional dimensions.

<u>Example 13 - Determine size of riprap required downstream of 8-ft-diam culvert and 14.4-ft-wide SAF basin with discharge of 1086 cfs:</u>

$$q = \frac{Q}{W_{SAF}} = \frac{1086}{14.4} = 75\ cfs/ft$$

$$V_1 = \frac{Q}{A} = \frac{1086}{0.785(8)^2} = 21.6\ fps$$

$$d_1 = \frac{q}{V_1} = \frac{75}{21.6} = 3.5\ ft$$

$d_2 = 8.4$ ft (from conjugate depth relations)

MINIMUM TAILWATER REQUIRED FOR A HYDRAULIC JUMP = 0.90 (8.4) = 7.6 ft

From page A27

$$\frac{d_{50}}{D_e} = 1.0 \left(\frac{V_e}{\sqrt{gD_e}} \right)^3$$

$$V_e = \frac{q}{D_e} = \frac{75}{7.6} = 9.9\ fps$$

$$d_{50} = 1.0 \left[\frac{9.9}{\sqrt{32.2(7.6)}} \right]^3 7.6$$

$$d_{50} = \underline{\underline{1.9\ ft}}$$

(Sheet 11 of 11)

APPENDIX B: NOTATION

A	Cross-sectional area of flow, ft^2
A_c	Rectangular culvert aspect ratio, W_o/D_o
A_d	Ratio of depth of flow to height of rectangular or square culvert or diameter of circular culvert d/D_o
A_r	Ratio of area of flow to the square of the culvert height, $A_c A_d$
B	Base width of channel, ft
C	Coefficient
d	Depth of uniform flow in culvert, ft
d_1	Depth of flow upstream of hydraulic jump, ft
d_2	Theoretical depth of flow required for hydraulic jump, ft
d_{50}	Diameter of average size stone, ft
D	Depth of flow in channel, ft
D_e	Depth of flow exiting energy dissipator, ft
D_o	Height of rectangular, width and height of square, and diameter of circular culverts, ft
D_s	Depth of scour, ft
D_{sm}	Maximum depth of scour, ft
D_W	Diameter of stilling well, ft
F	Froude number of flow at culvert outlet, $F = Q/A\sqrt{gd}$
F_{ch}	Froude number of flow in channel, $F_{ch} = Q/\sqrt{gA^3/T}$
g	Acceleration due to gravity, ft/sec^2
H	Depth of recessed apron and height of end sill, ft
K, K_2	Coefficients
L	Length of flared outlet transition, ft
L_s	Length of scour, ft

L_{sm} Maximum length of scour, ft

L_{sp} Length of stone protection, ft

n Manning's roughness coefficient

q Discharge per foot of outlet width, cfs/ft

Q Discharge, cfs

S Slope of channel bottom for partial pipe flow and slope of energy gradient for full pipe flow

t Duration of flow, minutes

T Top width of flow in channel, ft

T_B Thickness of geometrically similar cellular block, ft

T_S Thickness of geometrically similar sack revetment, ft

T_W Depth of stilling well below invert of incoming pipe, ft

TW Tailwater depth above invert of culvert outlet, ft

230

V Average velocity of flow in channel, fps

V_e Average velocity of flow exiting energy dissipator, fps

V_s Volume of scour, ft^3

V_1 Average velocity of flow upstream of hydraulic jump, fps

W_o Width of rectangular, square, or circular culvert, ft

W_s Width of scour from center line of single circular or square outlet

W_{sm} One-half maximum width of scour from center line of single circular or square outlet, ft

W_{smr} One-half maximum width of scour from center line of single rectangular outlet or a multiple outlet installation, ft

$$W_{smr} = W_{sm} + \frac{W_o}{2} - \frac{D_o}{2}$$

W_{sp} Width of stone protection, ft

W_{sr}	Width of scour from center line of single rectangular outlet or a multiple outlet installation, ft $W_{sr} = W_s + \frac{W_o}{2} - \frac{D_o}{2}$
W_{VI}	Width of U. S. Bureau of Reclamation type VI basin, ft
W_{SAF}	Width of St. Anthony Falls stilling basin, ft

Backfill is compacted over corrugated metal pipe—Bolivia.

CORRUGATED METAL PIPE CULVERTS

Structural Design Criteria
and
Recommended Installation Practices

By the Bridge Division
Office of Engineering and Operations

Reported by Merrill Townsend
Structural Engineer

U.S. DEPARTMENT OF COMMERCE
John T. Connor, Secretary

BUREAU OF PUBLIC ROADS
Rex M. Whitton, Administrator

June 1966

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C., 20402 - Price 25 cents

CONTENTS

Figures, Charts, and Tables

INTRODUCTION

A design method is presented herein which takes into consideration the major factors that influence design and performance of corrugated metal pipe culverts. The factors take into account the many years of field experience with the performance of flexible culverts and the vast amount of research studies on buried flexible structures.

Based on these factors, design criteria are presented in section 2, Design, and a design chart has been prepared for each type of corrugation showing maximum permissible fill heights for each of the design criteria.

Inasmuch as the adequacy of any pipe design can be nullified by poor installation practices such as lack of uniformity in pipebed bearing, poor quality of sidefill material, or lack of adequate compaction thereof, a section on installation practices is included which sets up basic installation requirements necessary to obtain satisfactory performance of pipe culverts.

Section 1: GENERAL COVERAGE

1.1 These criteria cover the design of corrugated steel and corrugated aluminum pipe culverts of riveted, resistance spot-welded, helical, and bolted fabrication. The design charts included provide for a rapid determination of the maximum allowable fill height for a specific gage, or the gage required for a specific fill height, for given pipe diameters.

1.2 The recommended installation practices cover the requirements for adequate installations which are necessary to obtain satisfactory performance of flexible pipe culverts designed in accordance with the criteria.

1.3 Design charts prepared from the design criteria are shown in section 4. Recommended fill height tables obtained from these charts are also shown in section 4 for use by those preferring tables.

Section 2: DESIGN

2.0 The following criteria embody the factors which must be investigated in the design of corrugated metal pipe culvert:

I—Deflection or flattening of pipe
II—Critical buckling of pipe wall
III—Longitudinal seam strength

The first two criteria, I and II, consider the mutual function of the metal pipe barrel and the soil surrounding it as a composition structure.

2.0.1 Loads on culvert pipe.

In applying the above criteria, the weight of embankment per linear foot of pipe which the culvert pipe must carry is assumed to be the weight of a column of earthfill equal in width to the nominal pipe diameter, "D," in height to "H," and weighing 120 pounds per cubic foot. A more accurate estimate of this load on the pipe could be made by using load coefficients from Marston's charts for positive and negative projecting embankment installations if the range of the settlement ratio coefficient, r_{sd}, for flexible pipe was established. The little research made thereon indicates that the values of r_{sd} range from -0.5 (negative) to $+0.1$ (positive) and since negative values of r_{sd} will result in lower weights than that of the column of embankment over the pipe the latter is generally on the conservative side. The settlement ratio coefficient r_{sd} is a value determined by an equation involving deflection of pipe, settlement of pipe flow line, settlement of sidefill adjacent to pipe, and deformation of embankment fill adjacent to and above top of pipe. Further research is needed to more definitely establish the range of r_{sd} so that a more accurate estimate of the pipe load can be made. The value of 120 pounds per cubic foot is considered a more realistic weight of compacted embankment than 100 pounds. However fill heights determined from the design charts (and fill height tables) can easily be adjusted for the actual unit weight by multiplying by the factor $\frac{120}{w}$ where "w" is the actual weight per cubic foot.

2.0.2 Definition of symbols used.

A_s=area in square inches per linear inch of pipe (table A)
D_L=deflection lag factor 1.5 in deflection formula
D=nominal pipe diameter in feet
E=modulus of elasticity of metal, p.s.i.
E'=modulus of passive earth resistance (soil reaction) p.s.i.
f_b=ultimate buckling stress, p.s.i.
h=height of fill above top of pipe in feet
I=moment of inertia of pipe wall in inches^4 per inch (table A)
k=bedding constant 0.1 in deflection formula
K=soil stiffness coefficient
R=radius of pipe in inches
r=radius of gyration of pipe wall, inches
w=unit weight of embankment in pounds per cubic foot (120)
W=total fill weight on pipe in pounds per linear foot of pipe
W_c=W/12, pounds per linear inch of pipe
x=vertical deflection of pipe=total horizontal deflection

2.1 Criterion I—Deflection of pipe.

A deflection of 5 percent of nominal pipe diameter below circular shape has generally been accepted as the limiting deflection and the gage-fill height curves for deflection shown in design charts I to V, section 4, are based on this 5 percent. Higher values of fill height "h" may be realized by vertical elongation of pipe. For a 5-percent elongation a total deflection of 10 percent from the elongated shape may be allowed and values of "h" are taken to be two times the values of "h" obtained from the deflection curves. The above assumption is not theoretically correct because of changes in pipe radii but inasmuch as the load on the pipe is based on the circular pipe diameter, doubled values of "h" are believed to be on the conservative side.

Deflection is computed by formula 1, $x=\frac{D_L k W_c R^3}{EI+0.061R^3E'}$ from "Soils Engineering" by M. G. Spangler. (*1*)*

Since the deflection x is limited to 5 percent of D the value of x may be replaced by $0.05\times 12D$ (all values in inches). Substituting values for coefficients and rearranging to solve for h results in the formula $0.05D\times 12=\frac{0.15R^3\times 10Dh}{EI+0.061R^3E'}$ where $W_c=\frac{120Dh}{12}$ and $h=\frac{0.6DEI}{15DR^3}+\frac{0.6D\times 0.061R^3E'}{1.5DR^3}$ $=\frac{EI}{2.5R^3}+0.0244E'$. For $E'=700$ p.s.i., $h=\frac{EI}{2.5R^3}$ $+17.08$ feet in which 17.08 feet (which is $0.0244E'$) represents the fill load the soil structure will carry while $\frac{EI}{2.5R^3}$ represents that which the metal pipe will carry.

Values of E' (passive soil pressure), and K (*2, 3, 5*), soil stiffness coefficient used in criterion II, are interdependent and are influenced by the quality of the sidefill material and the degree of compaction (density) thereof. The design charts have been prepared on the basis of normal installation conditions, which require a value of 700 p.s.i. for E' with good sidefill material compacted to 85-percent Proctor Density which is estimated to have a soil stiffness coefficient of K=0.44 The use of better quality sidefill material with a greater degree of compaction will increase the value of E'. Correspondingly the value of K will decrease in numerical value which means conversely a higher value of ultimate buckling stress f_b. With excellent sidefill material (graded gravel or crushed stone) compacted to 95-percent Proctor Density it is estimated that a value of $E'=1{,}400$ p.s.i. (and value of K=0.22) may be used for special designs. Special designs shall be used only when the engineer is reasonably certain that the requirements for excellent sidefill material with 95-percent compaction can be met.

All values of E' and K are estimated values based on results of research studies but further research is needed to correlate their values with various kinds of sidefill material compacted to varying degrees of density.

Failure of flexible pipe by deflection (decrease in vertical diameter) will not usually occur until deflection exceeds 18 to 20 percent below circular shape consequently designs based on 5 percent will provide a factor of safety of at least 3.33.

2.1.1 Preparation of criterion I curves.

Maximum values of h for this criteria are obtained from the formula $h=\frac{EI}{2.5R^3}+17.08$, using $E'=700$ p.s.i. Values of EI for all gages and depths of corrugations are obtained from table A, and values of h are then computed and plotted for steel and aluminum pipe for all gages and pipe diameters conventionally used.

Where special designs are required to meet unusual conditions the higher value of $E'=1{,}400$ p.s.i. previously cited may be used but formula 1 should be modified as follows: $E'=1{,}400$ and D_l (lag factor) decreased from 1.5 to 1.25 and then

$$h=\frac{EI}{2.0833R^3}+0.0298E'=\frac{EI}{2.0833R^3}+41.0\text{ feet.}$$

A curve has been computed only for 1# (gage) six inch by two-inch corrugations to show the increase in fill heights, "h", that may be realized by very good installation practices.

2.2 Criterion II—Critical buckling of pipe wall.

This criterion provides for the design of pipe based on the wall area required for a limiting buckling stress which takes into account the restraining effect of the soil structure around the

*All italic figures shown in parentheses are reference numbers.

pipe. The restraining effect of the soil structure (sidefill material) depends on the charateristics of the sidefill material and its density (degree of compaction) and is reflected in the value of the soil stiffness coefficient K which ranges from 1.0, representing no restraint, to 0.00 which represents an ideal condition of full restraint (*2*, *3*, *5*).

2.2.1 Wall buckling stresses are determined from diagrams for steel and aluminum shown in figures 1 and 2. A curve is drawn for each of three soil conditions: $K=1$ for hydrostatic soil, $K=0.44$ ($E'=700$) for good sidefill compacted to 85-percent Proctor Density, and $K=0.22$ ($E'=1{,}400$) for excellent sidefill compacted to 95-percent Proctor Density. The formulas for plotting the curves are shown on the diagrams. The value of $\left(\frac{D}{r}\right)^2$ for each diameter of pipe is then plotted at the bottom of the diagram for each depth of corrugation used in fabrication so that the value of f_b for a specific diameter and depth of corrugation can be obtained.

The diagram for aluminum buckling stresses shown on figure 2 is based on ultimate and yield strength values shown in table A for aluminum corrugated metal pipe, AASHO M-196. The ultimate and yield strength for structural plate material are slightly higher but will effect the buckling stress values "f_b" very little for some diameters, consequently the wall buckling curves for structural plate pipe are based on values of f_b obtained from figure 2.

2.2.2 Preparation of criterion II curves.

Maximum fill heights are calculated from the formula $h=\frac{0.12\, f_b A_s}{D}$ which provides a factor of safety of 2. The values of f_b are taken from figure 1 for steel and figure 2 for aluminum and fill heights "h" are calculated and curves plotted for all gages and pipe diameters conventionally used.

2.3 Criterion III—Longitudinal seam strength.

A pipe culvert can be only as strong as the riveted, welded, or bolted longitudinal seams. Maximum-fill heights for this condition are calculated and plotted for all gages and pipe diameters conventionally used.

2.3.1 Preparation of criterion III curves.

The load used in calculations for h is the weight of one-half the column of earthfill over the pipe, $W/2$, which expressed in terms of h and D is $\frac{120\, Dh}{2}$ or $60\, Dh$ (*4*). By equating this to the longitudinal seam strength values h is calculated and plotted for all gages and pipe diameters conventionally used. Longitudinal seam strength values based on actual tests are shown in table B and a factor of safety of 3.33 is applied thereto.

2.4 Design charts.

Selection of the governing fill height h for a given diameter and gage requires a comparison of h values by the three criteria. In order that this may be readily accomplished fill-height curves for all three criteria are plotted on a separate design chart for each depth of corrugation and metal. Design charts I to V covering these corrugations are located at the back of the brochure. It must be kept in mind in using them that maximum fill heights from deflection curves may be doubled if pipe is vertically elongated 5 percent. This is applicable to pipe of 30 inches diameter and larger.

Values of h determined from these charts may be easily adjusted for other values of unit fill weight w by applying the factor

$$\frac{120}{\text{actual weight per cubic foot}}.$$

Longitudinal seam strength curves in charts I to V are based on values shown in table B for riveted and bolted fabrication. Tests on resistance spot-welded pipe indicate somewhat higher values, and for helical pipe the seam becomes the helical folded lock seam and higher values are also indicated by tests. However, until higher values for these seams become well established by a sufficient number of tests, designs for welded and helical pipe will be based on the curves shown on the design charts. When higher values are established for seam strengths adjustment in value of fill heights is easily made by applying the factor $\frac{\text{higher seam strength}}{\text{riveted seam strength}}$ to the values of h as determined from the seam-strength curves.

2.5 Use of design charts.

Example 1.—Given: 54-inch diameter steel pipe; fill height—37 feet; required—gage and corrugation type.

Charts I and II both cover this diameter of pipe. Using chart I, spot the 37 feet on the ordinate axis and follow horizontally to the right to its intersection with the vertical 54-inch diameter line. The location of this point in relation to the deflection and buckling curves will determine the gage required to satisfy these criteria. It should be noted that the deflection curves are based on original circular shape and elongation of 5 percent vertically will permit doubling the fill height. In this example the 37-foot fill height exceeds the fill height shown for even an 8-gage metal; consequently elongation will be required. To obtain the gage required for seam strength spot the 37-foot point on the ordinate axis at the right side of the chart and follow horizontally to the left to its intersection with the vertical 54-inch diameter line and read the gage curve at or above the intersection. The gage selection from the deflection and buckling curves should also be based on the gage curve at or above the intersection point previously described for those curves. In all cases the heaviest gage required by any one of the criteria is the governing gage. The above procedure applies in the use of all five design charts.

Design Chart I, 2⅔ inch by ½ inch Corrugations

h=37 feet	Diameter 54 in.	Seam Strength
Deflection	Wall Buckling	14 gage—33 feet
14-gage	14 gage—33.5 feet	12 gage—67 feet
h=18.5 feet	12 gage—48.0 feet	
elongated h—		
18.5×2=37 feet		

Wall buckling and seam-strength criteria require 12-gage metal of 2⅔ inch by ½ inch corrugations.

Design Chart II, 3 inch by 1 inch Corrugations

Deflection	Wall Buckling	Seam Strength
16-gage h=22 feet	16 gage—54.5 feet	14 gage—29.5 feet
elongated—		12 gage—46.5 feet
22×2=44 feet		

Seam-strength curves require 12-gage metal, 3 inch by 1 inch corrugations.

Result: 12-gage metal of 2⅔ inch by ½ inch or 3 inch by 1 inch corrugations is required.

Example 2.—Given: Steel pipe, 120-inch diameter; fill height—21 feet; required—gage and corrugation.

Design charts II and III both cover this size.

Design Chart II

Deflection	Wall Buckling	Seam Strength
16 gage—17.5 feet	12 gage—19.5 feet	12 gage—21 feet
elongated—	10 gage—25 feet	
7.5×2=35 feet		

Design Chart III

12 gage—20.5 feet	12 gage—43 feet	12 gage—21 feet

Result: 10-gage, 3 inch by 1 inch (elongated) or 12-gage, 6 inch by 2 inch metal is required.

Example 3.—Given: Aluminum pipe, 72-inch diameter; fill height—11 feet; required—gage and corrugation.

Design Chart IV, 2⅔ inch by ½ inch

Deflection	Wall Buckling	Seam Strength
16 gage—17 feet	8 gage—11 feet	16 gage—12 feet

Design Chart V, 9 inch by 2½ inch

Deflection	Wall Buckling	Seam Strength
0.09 inch—24.5 feet	0.09 inch—42 feet	0.09 inch—18.5 feet

Result: 8-gage, 2⅔ inch by ½ inch or 0.09-inch, 9 inch by 2½ inch corrugations is required (0.09 inch approximately equal to 13 gage).

Example 4.—Given: Steel pipe, 180-inch (15 feet) diameter; fill height—47 feet; weight of fill—140 pounds per cubic foot; required—gage.

Design Chart III, 6 inch by 2 inch

First, 47 feet fill height at 140 pounds is equivalent to 55 feet at 120 pounds.

Then, 55 feet fill height must be used with the chart.

Deflection	Wall Buckling	Seam Strength
1-gage elongated	1 gage—59 feet	1 gage at 4 bolts—
2×20=40 feet		47.5 feet
Use 1-gage curve		1 gage at 8 bolts—
at E'=1,400		73 feet
1-gage elongated—		1 gage at 6 bolts—
2×44=88 feet		61 feet (computed)

This design requires 1-gage metal with excellent sidefill compacted to 95 percent Proctor Density. If the design engineer is not reasonably sure of attainment of the above requirements then redesign must be considered based on normal installation requirements. To accomplish this, twin pipes of smaller diameter giving the same flow capacity will be considered. In this problem two 112-inch-diameter (9.5 feet) pipes will provide approximately the same flow capacity (with inlet control) as the 180-inch pipe. Using the chart curves for normal installation the following results are derived:

Deflection	Wall Buckling	Seam Strength
1-gage elongated—2×27.5= 55 feet	1 gage—100 feet plus	1 gage—75 feet

Result: Two 112-inch pipes elongated, 1 gage at 4 bolts per foot with normal installation requirements.

This problem illustrates a choice for the design engineer of one 180-inch, elongated pipe of 1 gage at 6 bolts per foot with excellent sidefill (95 percent Proctor Density) or a more costly one of two 112-inch, elongated pipes, 1 gage metal, at 4 bolts per foot with normal installation if the excellent installation requirements cannot be met.

2.6 Pipe arch design.

Pipe arches are intended for use where cover over pipe "h" is limited. Consequently fill heights are only shown to 15 feet or less if the corner bearing pressure, 2 tons per square feet, governs. Higher values of bearing may be used if justified by foundation investigation. Corner bearing pressure is determined by dividing one-half the total load on the pipe $W/2$ by the corner radius (in feet).

The gage of metal required for fills up to 15 feet is obtained from the appropriate design chart using a diameter of circular pipe approximately equal to the span of pipe arch.

2.7 Effect of live load on pipe.

Corrugated metal pipe, steel and aluminum, designed in accordance with the design charts or fill height tables are stronger than would be required to carry H20 trucks with the minimum cover specified in the fill height tables which is 12 inches from top of pipe to top of subgrade, for diameters up to 96 inches.

This cover will actually provide at least 18 inches to top of pavement. As an example, a 24-inch-diameter pipe, 16-gage metal, will carry 30-foot fill, if steel, and 22 feet, if aluminum. An H20 wheel load will, at 18-inch cover, produce an equivalent fill height of 14 feet (including the weight of cover), which is considerably less than the design capacity. The effect of live load decreases rapidly as pipe diameters increase.

2.8 Handling and installation strength.

Inasmuch as stiffnss of pipe required to withstand the handling and the compaction of sidefill around the pipe is a matter of experience, no attempt will be made to propose a criterion for this item. However, it is suggested that the deflection curves provide an approximate guide for increasing gages to maintain satisfactory stiffness. To use this guide, a heavier gage should be selected when the height of fill indicated by the deflection curves (5-percent deflection, no elongation) is 18.1 feet or less. This means that since the sidefill will carry 17.1 feet of fill, the gage of pipe metal should be made heavy enough to carry better than 1 foot, making the total fill height 18.1+ feet. Effect of elongation should not enter into this determination.

2.9 Durability of corrugated metal pipe.

The service life of corrugated metal pipe may be seriously affected by corrosion and/or abrasion. Corrosion may be caused by excessive acid or alkaline condition (pH) of the fluid carried by the pipe or of the sidefill material placed around the pipe. A method of estimating service life of steel pipe culverts is described in Highway Research Board Proceeding, 1962, "Field Test for Estimating the Service Life of Corrugated Metal Pipe Culverts" by J. L. Beaton and R. F. Stratfull, California Division of Highways (*8*). Results of corrosion and abrasion are also described in "Culvert Performance Evaluation" by Washington State Highway Commission, Department of Highways (*9*). Information in regard to corrugated aluminum pipe service life may be obtained from a paper in Highway Research Record No. 95, "A Preliminary Study of Aluminum as a Culvert Material" by Eric F. Nordlin and R. F. Stratfull, California Division of Highways (*10*).

Abrasion is caused by the materials carried with the flow through the pipe. Its severity will depend on the nature of the materials carried and velocity of flow.

Additional protection against corrosion may be obtained by the use of bituminous coatings (AASHO M–190), and paved inverts (AASHO M–190, type B or C) may be used as additional protection against abrasion. For structural plate pipe, heavier gages may be specified for the plates in the invert. Experience has shown that 16-gage metal is the lightest material that should be used to provide a reasonable service life.

The need for and type of protection against corrosion and abrasion should be determined on the basis of actual site investigations.

For important installations where interruption of traffic would be undesirable or where the cost of replacement would be excessive, a minimum of 10-gage metal shall be used for steel structural plate and a minimum of 0.15-inch thickness for aluminum structural plate.

241

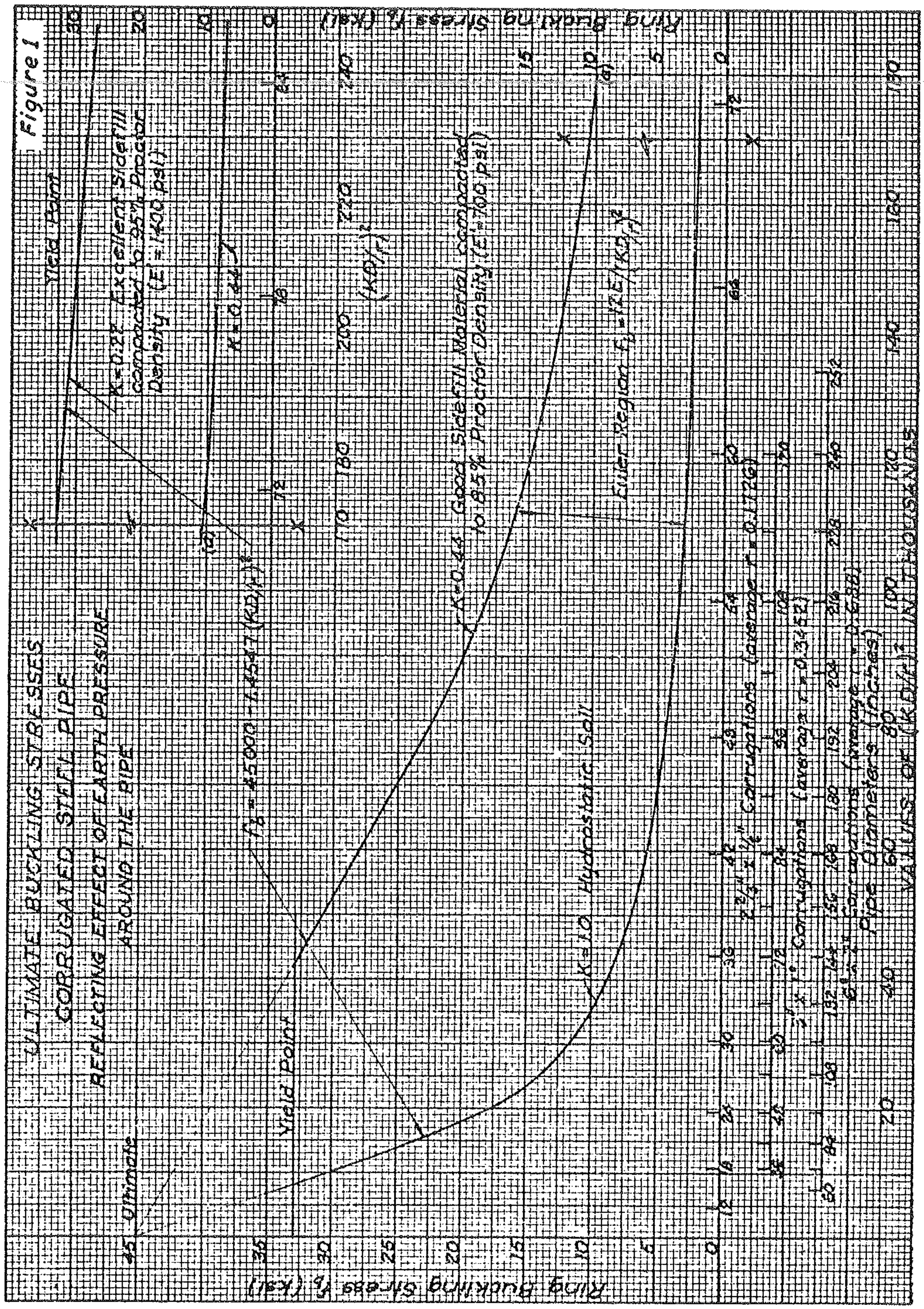


Figure 1. Ring buckling stress diagram, steel

Figure 2. Ring buckling stress diagram, aluminum

TABLE A.—Geometrical and physical properties upon which design charts are based

Corrugated steel (6)

Gage	2⅔″ x ½″ Corr.		3″ x 1″ Corr.		6″ x 2″ Corr.	
	A_s (sq. in./in.)	I (in.^{4}/in.)	A_s (sq. in./in.)	I (in.^{4}/in.)	A_s (sq. in./in.)	I (in.^{4}/in.)
16	.0646	.001892	.0742	.008658		
14	.0808	.002392	.0927	.010833		
12	.1130	.003425	.130	.015458	.1297	.060416
10	.1454	.004533	.1674	.020175	.1669	.078166
8	.17775	.005725	.2048	.025083	.2041	.096166
7					.2283	.1078
5					.2666	.126916
3					.3048	.146166
1					.3432	.165833

Chemical Requirements: Corrugated Steel Pipe—AASHO M-36-60; Structural Plate Pipe—AASHO M-167.
Physical Properties: E—29,000,000 p.s.i.

Minimum		
Tensile Strength, p.s.i.	Yield Strength, p.s.i.	Elongation, 2 inches
45,000	33,000	20 percent

Corrugated aluminum

Gage	2⅔″ x ½″ Corr. (6)		9″ x 2½″ Corr. (7)		
	A_s (sq. in./in.)	I (in.^{4}/in.)	Thickness (inches)	A_s (sq. in./in.)	I (in.^{4}/in.)
16	.0646	.001892	.09	.105	.082
14	.0808	.002392	.10	.117	.091
12	.1130	.003425	.125	.146	.114
10	.1454	.004533	.15	.175	.136
8	.17775	.005725	.175	.204	.159
			.20	.234	.182
			.225	.263	.205
			.25	.292	.227

Chemical Requirements: Corrugated Aluminum Pipe—AASHO M-196; Structural Plate Pipe—ASTM B-209 Alloy 5052.
Physical Properties: Corrugated Aluminum Pipe—E=10,200,000 p.s.i.

Thickness (inches)	Minimum		
	Tensile Strength, p.s.i.	Yield Strength, p.s.i.	Elongation, 2 inches
0.051 to 0.113	31,000	24,000	4 percent
0.114 to 0.249	31,000	24,000	5 percent
Structural Plate Pipe—E—10,200,000 p.s.i.			
0.090 to 0.175	35,500	----------	6 percent
0.175 to 0.250	34,400	----------	8 percent

TABLE B.—Ultimate longitudinal seam strength values

A factor of safety of 3.33 shall be applied to those ultimate values for computation of fill heights.

(1) Riveted pipe

Gage of Metal	Steel-kips/foot (6)						Aluminum-kips/ft. (5)			
	5/16" rivets			3/8" rivets			5/16" rivets		3/8" rivets	
	2 2/3" x 1/2"		3" x 1"	2 2/3" x 1/2"		3" x 1"	2 2/3" x 1/2"			
	Single	Double	Double	Single	Double	Double	Single	Double	Single	Double
16	16. 75	21. 5	19. 2				9. 0	14. 0		
14	18. 2	29. 8	26. 5				9. 0	18. 0		
12				23. 4	46. 8	41. 6			15. 6	31. 5
10				24. 5	49. 0	43. 5			16. 2	33. 0
8				25. 6	51. 3	45. 6			16. 8	34. 0

(2) 3" x 1" riveted, welded, or bolted steel pipe

When specifications require 3/8" rivets for 16- and 14-gage metal and 7/16" rivets for 12-, 10-, and 8-gage metal or double spot welds, or 1/2" diameter ASTM A-325 bolts for 16- to 8-gage metal, the fill heights determined from seam strength curves in design chart II can be adjusted to reflect the increased seam strengths shown in the following table by applying the appropriate adjustment factor shown therein.

Gage of metal	3/8" double riveted	7/16" double riveted	Adjustment factor
	Kips/foot (11)		
16	25. 8		1. 34
14	34. 3		1. 29
12		53. 0	1. 28
10		61. 0	1. 40
8		64. 0	1. 40

(3) Structural plate pipe (3/4" bolts)

Gage	Steel plate-kips/ft. (6)			Aluminum plate-kips/ft. (7)		
	ASTM A 325 bolts			Metal thickness	Aluminum bolts (2 2/3/ft.)	Steel (2 2/3/ft.)
	4/ft.	6/ft.	8/ft.			
12	42			. 09"	22. 2	
10	62			. 10"	26. 4	
8	81			. 125"	34. 8	
7	93			. 15"	44. 4	
5	112			. 175"	52. 8	
3	132			. 20"		60. 0
1	144	184	220	. 225"		66. 0
				. 25"		72. 0

Section 3: INSTALLATION

3.0 General.

Pipe culverts and underdrains must serve two functions—hydraulic and structural. They must not only provide adequate passage of the fluids to be carried, but must be structurally adequate to support the weight of fill over them and any live loads superimposed thereon. Poor installation of pipe may result in structural failure with partial or total loss of hydraulic capacity. Therefore, pipe should not be buried and forgotten, but should be properly installed following the principles outlined in this section.

Special attention should be given to the protection of metal culverts to prevent failure due to the forces of water. Ends of culvert pipe projecting from a roadway embankment, particularly at the entrance, are vulnerable to failure by buoyant forces. Mitered or beveled ends bend inward because of reduced strength due to cut corrugations. Masonry headwalls, properly constructed, are means of reducing the risk of these types of failures. The flow of water along a culvert barrel can remove supporting material and, in some cases, the hydraulic pressures have been sufficient to collapse the metal barrel. If pervious material is used for culvert bedding and backfill, cutoff walls, collars, or impervious material should be placed at the entrance and at intervals along the culvert barrel to prevent loss of bedding and sidefill material.

3.1 Assembly.

Corrugated metal pipe, underdrains, and structural plate pipe shall be assembled in accordance with the manufacturer's instructions. All pipe shall be unloaded and handled with reasonable care. They should not be dragged over gravel or rock and should be prevented from striking rock or other hard objects during placement in trench or on bedding.

Corrugated metal pipe, riveted or welded, shall be placed on the bed starting at downstream end with the inside circumferential laps pointing downstream and with the longitudinal laps at the side or quarter points. The pipe sections shall be joined by coupling bands of like material. Standard one-piece bands are used for most installations. Two-piece bands may be used for larger pipe where installations are difficult. Bands are first slipped into position at the end of one pipe section with the band open to receive the next section. That section is then brought to within about three-fourths of an inch of the other section, corrugations of both sections matched, and bolts tightened. For smaller pipe, the bands should be tapped with a mallet to take up the slack, and for large pipe, a chain or cable-cinching device is required to draw the bands tight and insure tight joints. Coupling bands for annular and helical corrugated metal pipe shall provide circumferential and longitudinal strength to preserve the culvert alignment, prevent separation of the pipe sections, and prevent infiltration of sidefill material. Helically corrugated metal pipe shall be installed in a similar manner to the above procedure.

Bituminous coated pipe and paved invert pipe shall be installed in a similar manner to standard corrugated metal pipe with special care in handling to avoid damage to coatings. Paved invert pipe shall be installed with the invert pavement placed and centered on the bottom.

Where pipe is perforated for underdrains, the pipe shall be placed with perforations at the lower quarter points with bedding and backfill as specified on the plans for underdrains.

Structural plate pipe, pipe arches, and arches shall be installed in accordance with the plans and detailed erection instructions shipped with each structure that show the position of each plate and order of assembly. They should be assembled with as few bolts as possible until all the plates are in place. Three or four untightened bolts placed near the center of each plate along longitudinal and circumferential seams are sufficient. After several rings have been assembled, the remaining bolts can be inserted (loose), always working from center of seam to the corner of plate. Corner bolts shall be inserted only after all other bolts are in place and tightened. After all the plates have been assembled and bolted, the nuts shall be tightened progressively and uniformly, starting at one end of structure. This operation should be repeated to insure that bolts are tight. Bolts shall not be torqued above 300 foot-pounds in tightening.

3.2 Types of installations.

The most common type of installation used for highway culverts is the embankment installation where embankment fill is placed over the pipe and above existing ground surface. This type is further broken down into positive and negative projecting embankment installations as shown in figure 3. The positive projecting type covers those installations where the pipe is bedded with its top anywhere from 0.9 of the nominal pipe diameter above existing ground surface to level with it (the latter commonly referred to as zero projecting). The negative projecting embankment type covers those installations where the pipe is bedded in a trench with its top below the existing ground surface. Another type of installation is the trench installation which is seldom used for highway culverts. This type covers those installations where the pipe is placed in a trench with no fill above existing ground surface; for example, storm drains, sewers, etc.

3.3 Bedding.

The contact between a pipe and the foundation upon which it rests is the bedding. The load transmitted to a pipe from above must in turn be transmitted to the underlying soil. If a firm support of the pipe by its bed is established only over a narrow width or line, such as a round pipe on a flat bed, the intensity of the load (stress) at the bottom of pipe will be high and excessive deflection may occur. A wide band of support under the pipe will provide a better load distribution. The band of support should be uniform for the full length of the pipe. A modified class C bedding shown in figure 3A is recommended. The bedding blanket shown is to obtain better seating of the corrugations on the pipe bed.

3.4 Pipe foundation.

The foundation material under the pipe should be investigated for its ability to support the load which it must carry. If rock is closer than 12 inches under the pipe, it shall be removed and replaced with slightly yielding material as shown in figure 3B. Where in the opinion of the engineer the natural foundation soil is such as to require stabilization such material shall be replaced by a layer of good granular material as shown in figure 3C. Where an unsuitable material (peat, muck, etc.) is unexpectedly encountered at or below invert elevation during excavation, the necessary subsurface exploration and analysis shall be made and corrective treatment determined by the engineer.

3.5 Sidefill.

One of the most important phases of installation is the placing and compaction of sidefill material (commonly called backfill) around the pipe. Side support must be provided for flexible pipe so that they will carry the fill and live loads without excessive deflection. Side support can only be obtained by adequate compaction of good fill material around the pipe. Sidefill material within one pipe diameter of the sides of pipe and to 1 foot over the pipe shall be fine readily compactible soil or granular fill material. Sidefill beyond the one pipe diameter limit at sides of pipe may be regular embankment fill. Job-excavated soil shall not contain stones retained on a 2-inch ring, frozen lumps, chunks of highly plastic clay, or other objectionable material. Granular fill material shall be well-graded crushed stone or gravel with not less than 95 percent passing a one-half-inch sieve, and not less than 95 percent being retained on a No. 4 sieve. Care should be taken in selecting sidefill material to keep the mineral content low enough to avoid serious corrosion therefrom.

Sidefill material shall be placed as shown in figure 3D in layers not exceeding 6 inches in depth and compacted at near optimum moisture content by approved hand or pneumatic tampers to the density required for superimposed enbankment fill. Other approved compacting equipment may be used for sidefill more than 3 feet from sides of pipe. The sidefill shall be placed and compacted with care under the haunches of the pipe and shall be brought up evenly and simultaneously on both sides of the pipe to 1 foot above the top for the full length of the pipe. Fill above this elevation may be material for embankment fill. For negative projecting installations, the width of trench shall be kept to the minimum width required for placing pipe and adequate bedding and sidefill. Ponding or jetting of sidefill should not be permitted.

3.6 Elongated pipe.

When pipes are vertically elongated to carry higher fills, they shall be installed with the longer axis truly vertical.

3.7 Alinement.

Pipe shall be assembled to reasonably straight alinement and shall be so maintained until placing and compaction of sidefill has been completed.

3.8 Camber.

The invert grade of the pipe shall be cambered, when required, by an amount sufficient to prevent the development of a sag or back slope in the flow line as the foundation under the pipe settles under the weight of embankment. The amount of camber shall be based on consideration of the flow-line gradient, height of fill, compressive characteristics of the supporting soil, and depth of supporting soil stratum to rock.

3.9 Multiple installations.

Where multiple lines of pipe are to be installed they shall be spaced far enough apart to permit thorough tamping of sidefill around the pipe. To this end the sides of pipe shall be at least one-half the nominal pipe diameter or 3 feet apart, whichever is less.

3.10 Cover over pipe during construction.

All pipe shall be protected by a cover of at least 4 feet before permitting heavy construction equipment to pass over them during construction stage as the movement of heavy earthmoving equipment over the uneven fill surface during fill placement may generate high impact, subjecting the pipe to extremely heavy damaging loads.

3.11 Inspection.

Installation conditions have a very important effect on both the load on and the supporting strength of the pipe and a satisfactory installation requires attainment of design conditions in the field. Consequently, the engineer on the job should not only be familiar with good installation practices but should also keep a close check on the contractor's operations to insure fulfillment of that objective.

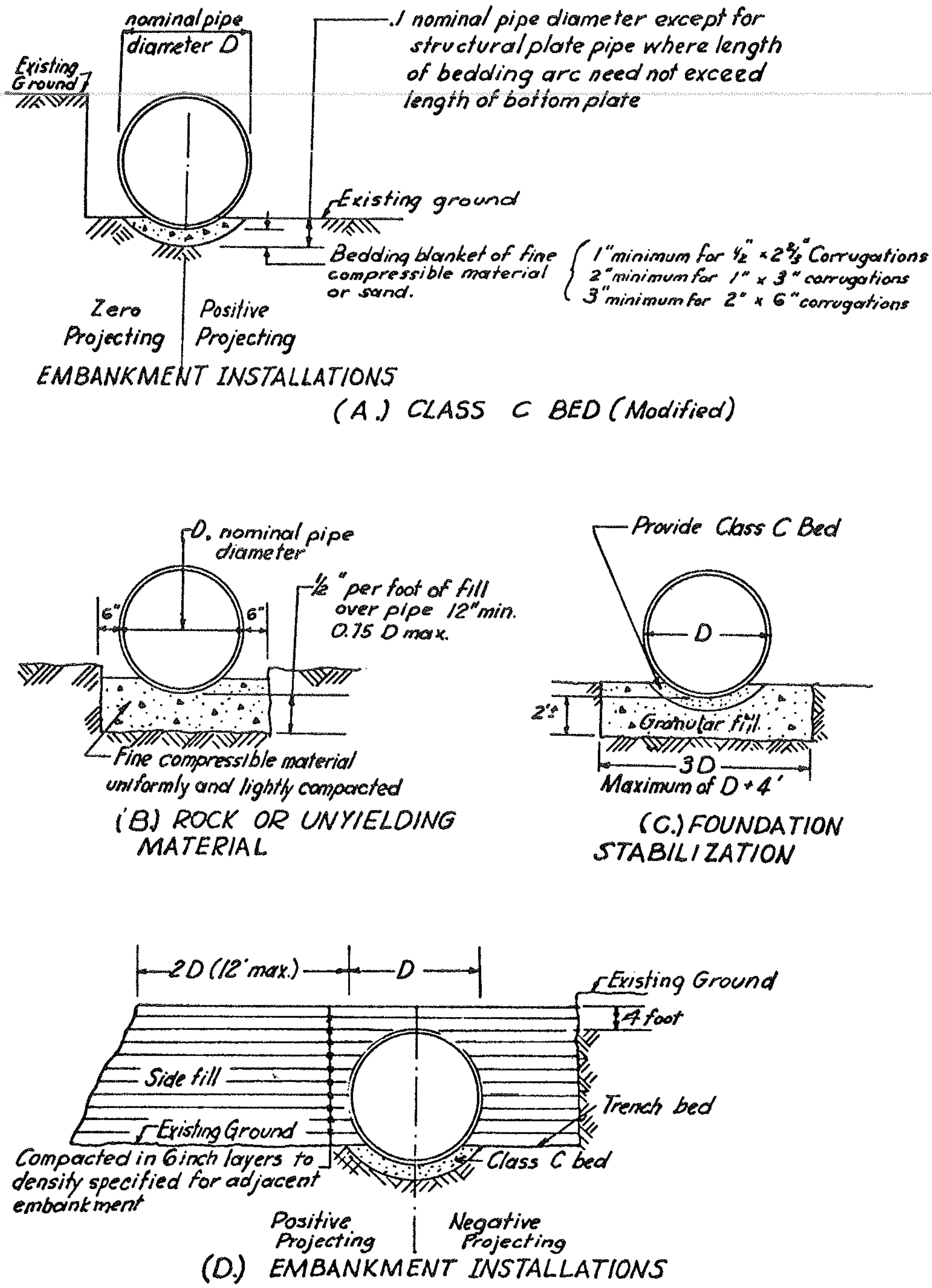


Figure 3. Installation types and beddings

Section 4: DESIGN CHARTS AND RECOMMENDED FILL HEIGHT TABLES

4.1 Design charts I to V have been developed from the design criteria given in section 2 and are based on the pertinent data shown in section 4.2. Fill-height (gage) tables 1 to 5 for pipe and tables 1a to 4a for pipe arches have been prepared from the design charts of like numerical designation and both reflect only the structural design requirements for pipe.

Corrosive and abrasive conditions may require heavier gages and/or protective coatings and determination of such requirements should be based on actual site investigations.

4.2 Basic data.

w = weight of embankment: 120 pounds per cubic foot

Maximum deflection below circular shape: 5 percent

Vertical elongation: 5 percent ± of nominal diameter

Safety factor—Longitudinal seam strength: 3.33

Safety factor—Pipe wall buckling: 2

E' = modulus passive soil resistance (sidefill) = 700 p.s.i. for soil coefficient K = 0.44

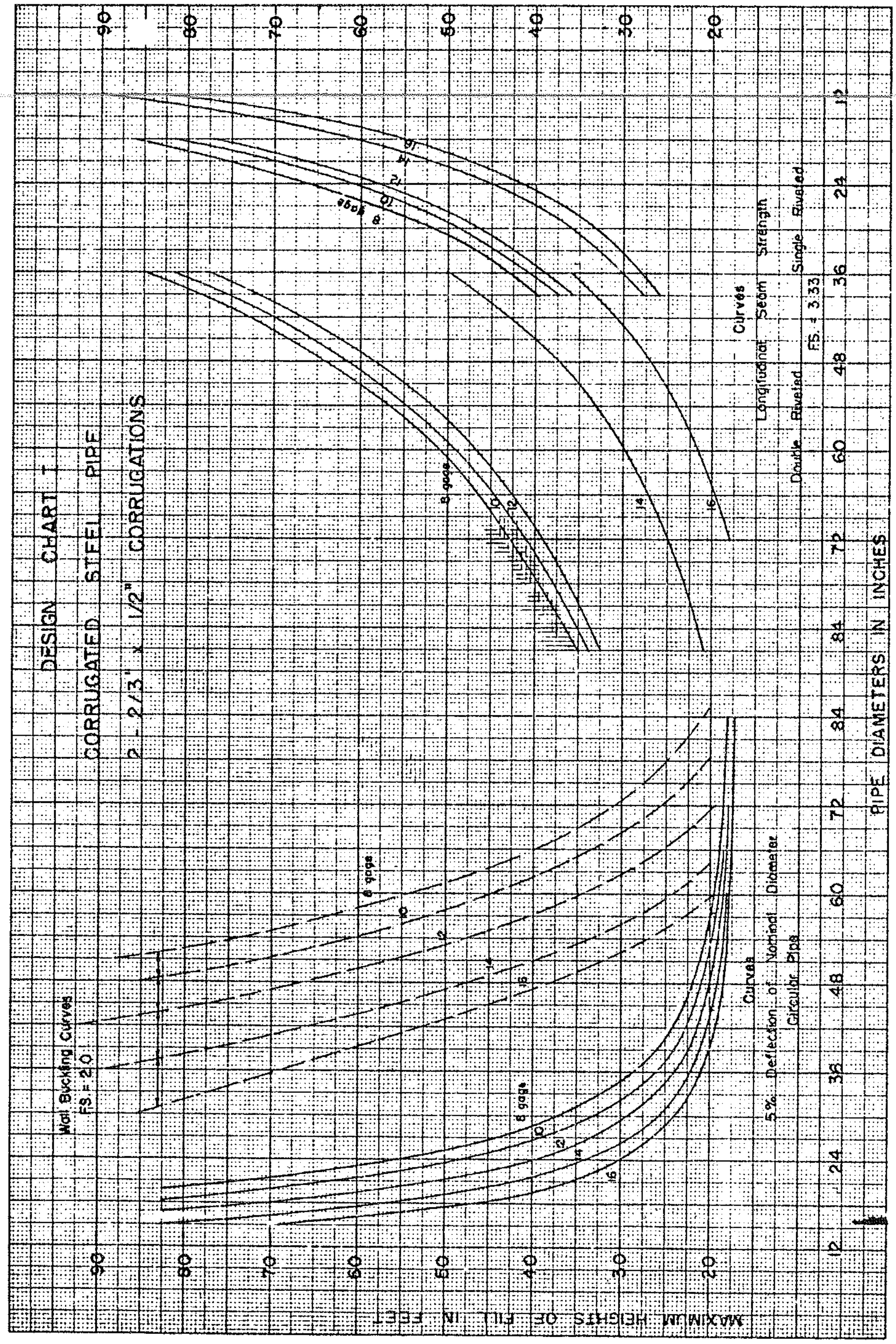
K = 0.44—Soil coefficient for good sidefill material compacted to 85-percent Proctor Density

4.3 Revisions to fill heights.

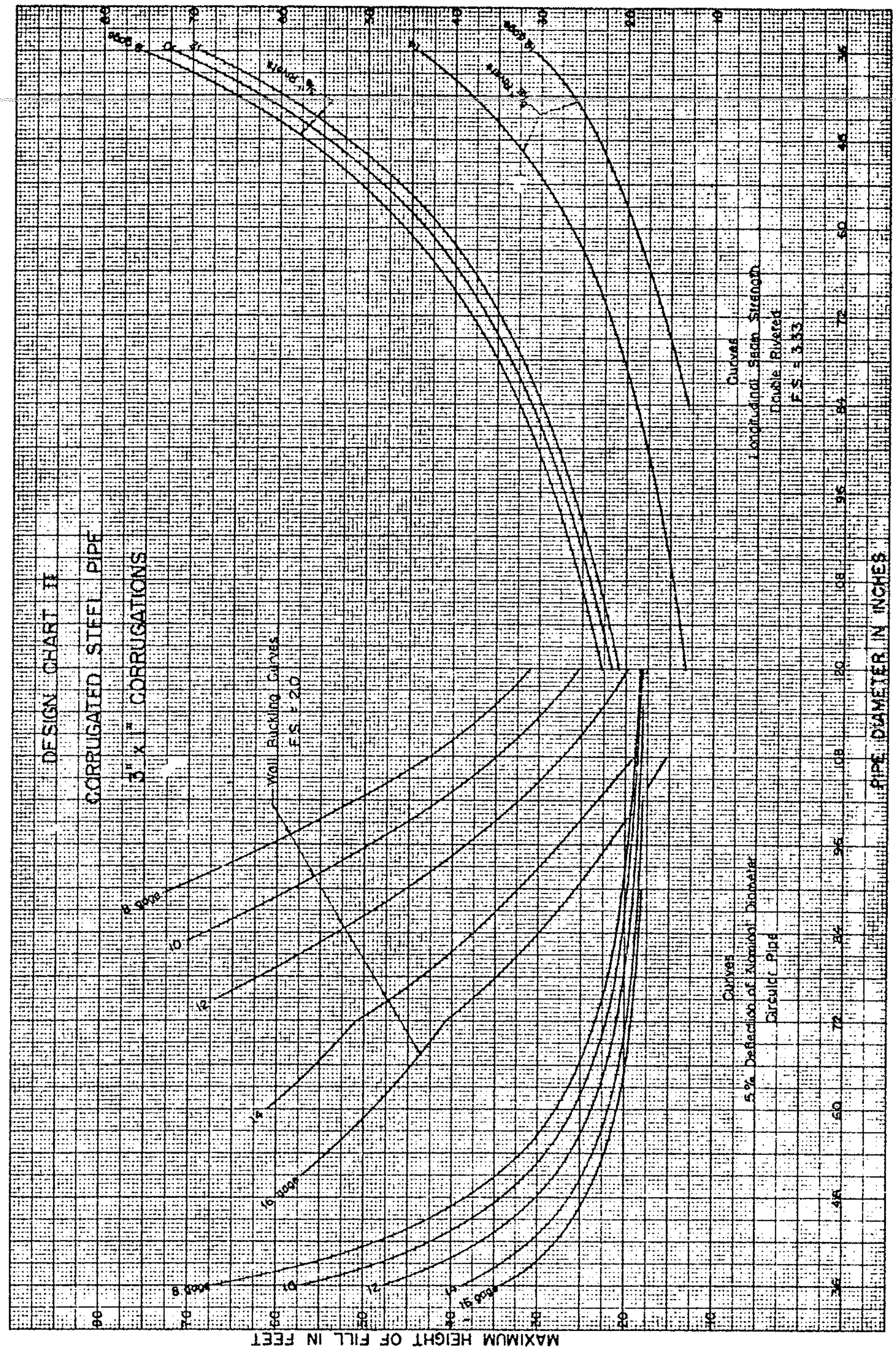
Fill-height values obtained from the design charts or fill-height tables may be modified for other values of "w" than 120 pounds per cubic foot by applying the factor $\frac{120}{w_a}$ where w_a is the actual weight per cubic foot.

3-Inch by 1-Inch Corrugated Steel Pipe

When specifications require the use of ⅜-inch rivets for 16- and 14-gage metal and 7/16-inch rivets for 12-, 10-, and 8-gage metal, or double spot welds, or ½-inch-diameter A.S.T.M. A-325 bolts for 16- to 8-gage metal, inclusive, increased seam strengths shown in table B(2) may be used and the fill heights determined from seam-strength curves, design chart II can be adjusted by the appropriate factor shown in table B(2). Comparison of these fill heights with criterion I and criterion II fill heights must still be made to determine the governing criterion.

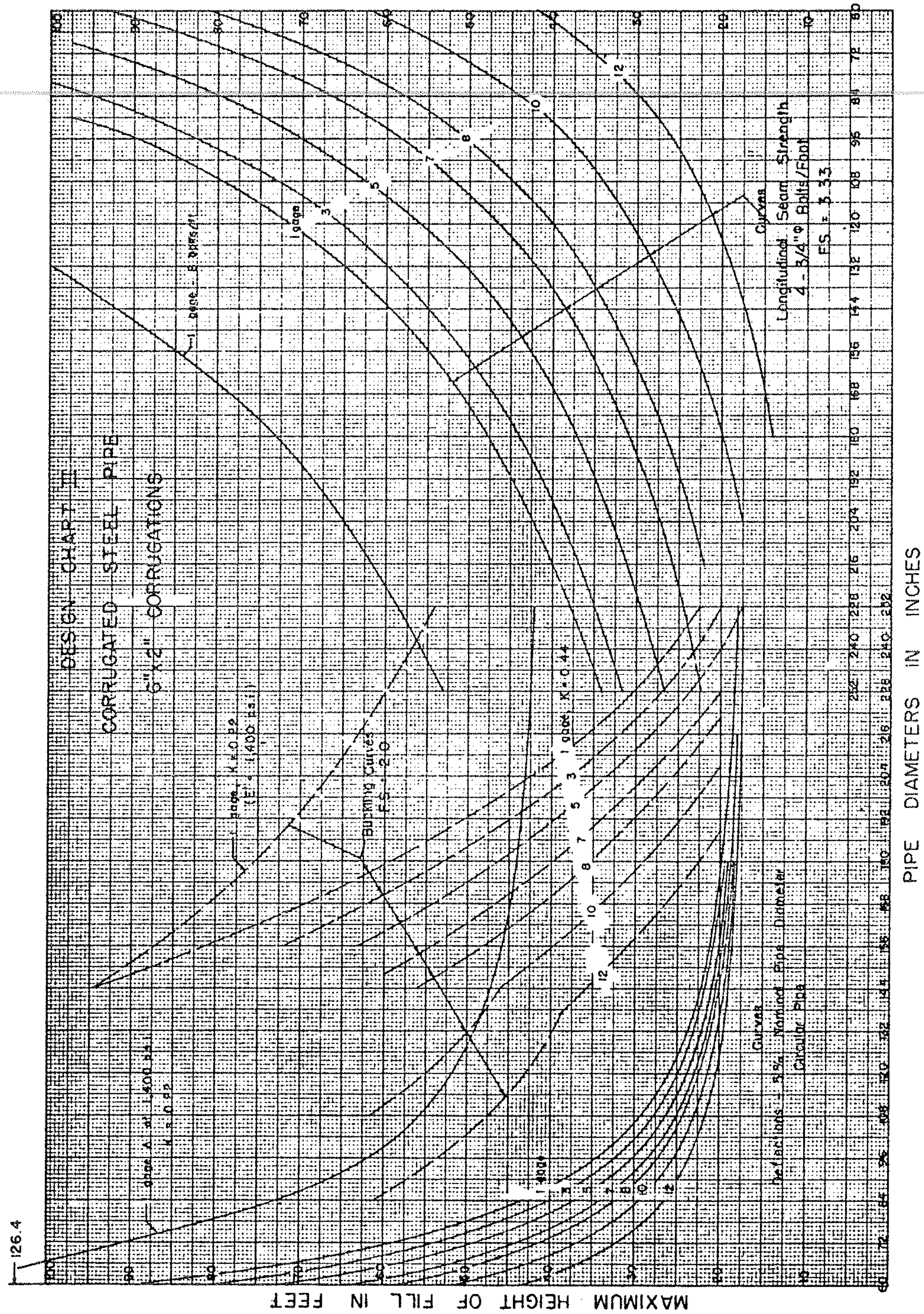


Design Chart I. Steel pipe, 2⅔" x ½" corrugations

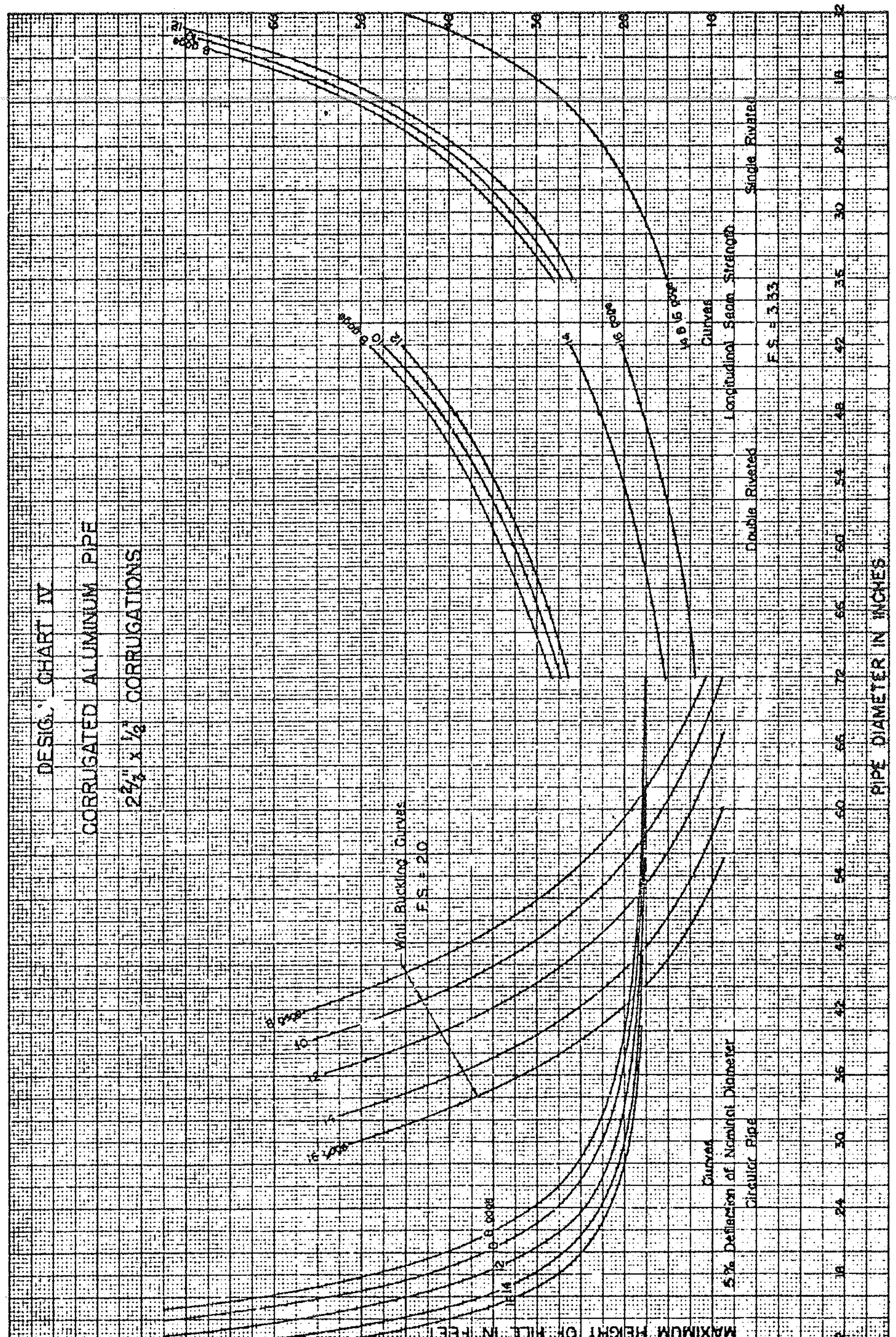


Design Chart II. Steel pipe, 3" x 1" corrugations

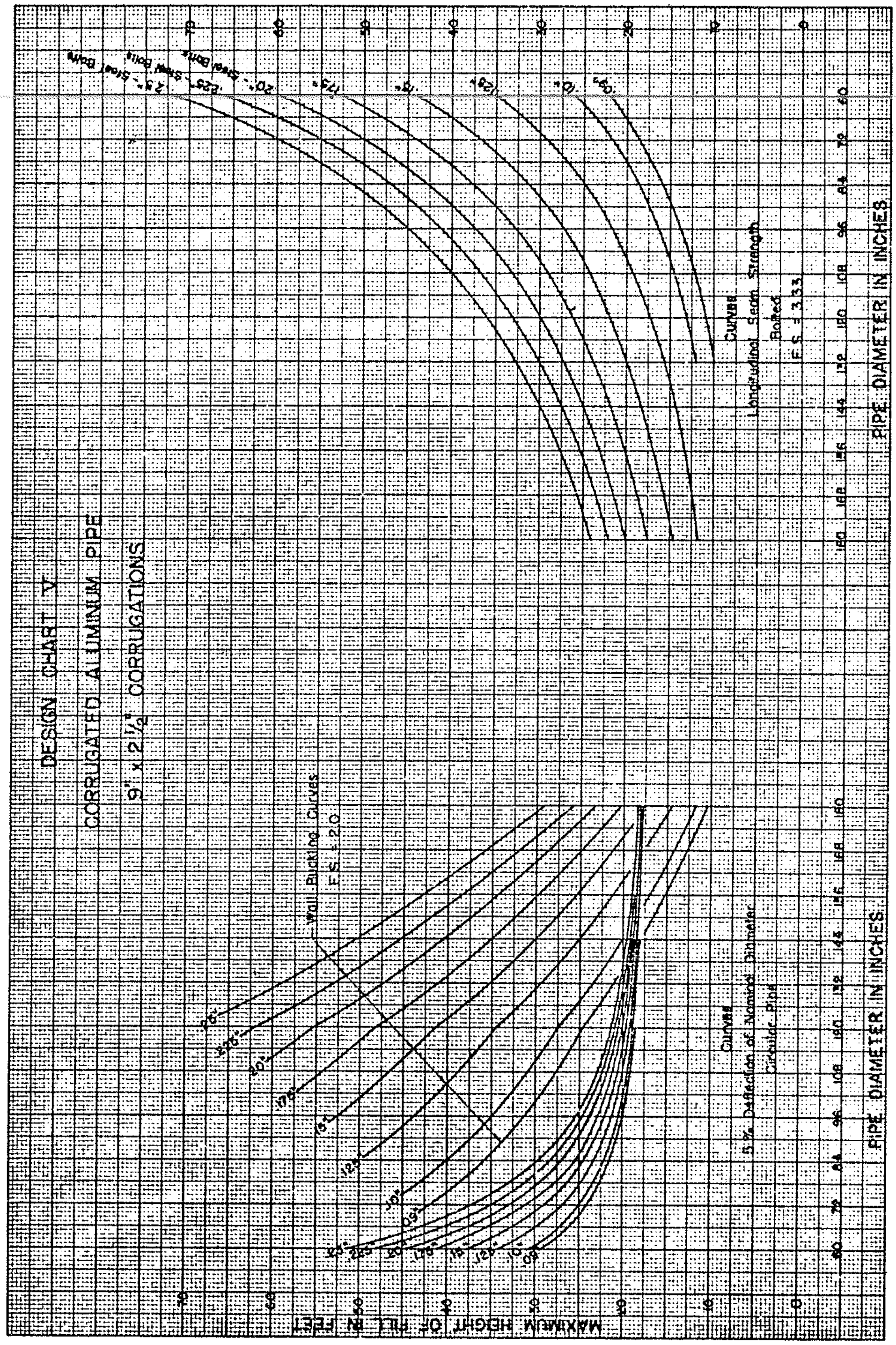
252



Design Chart III. Steel pipe, 6" x 2" corrugations



Design Chart IV. Aluminum pipe, 2⅔" x ½" corrugations



Design Chart V. Aluminum pipe, 9" x 2½" corrugations

RECOMMENDED FILL-HEIGHT TABLE 1.—Corrugated steel pipe, 2⅔-inch by ½-inch corrugations only, riveted, welded, or helical fabrication, H-20 loading

Pipe diameter	Minimum cover, top of pipe to top of subgrade	16 gage		14 gage		12 gage		10 gage		8 gage	
		Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated
Inches		Maximum fill heights above top of pipe in feet									
12	12	83	—	90	—	(115)	—	(122)	—	(127)	—
15	12	67	—	73	—	93	—	98	—	(102)	—
18	12	47	—	55	—	70	—	82	—	86	—
24	12	30	—	33	—	40	—	48	—	54	—
30	12	24	34	25	36	29	47	33	49	37	52
36	12	21	28	22	30	24	39	26	41	28	43
42	12	19	31	20	38	21	43	23	46	24	48
48	12	18	27	19	37	20	40	21	42	22	44
54	12	—	—	18	33	19	38	20	39	21	41
60	12	—	—	—	—	18	34	19	38	20	40
66	12	—	—	—	—	18	25	18	35	19	38
72	12	—	—	—	—	—	—	18	25	18	31
78	12	—	—	—	—	—	—	—	—	18	25
84	12	—	—	—	—	—	—	—	—	18	20

NOTES:

Fill heights exceeding 100 feet (enclosed in parentheses) shall be used only after thorough investigation of foundation material.

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

256

RECOMMENDED FILL HEIGHT TABLE 2.—Corrugated steel pipe, 3-inch by 1-inch corrugations, riveted, helical, welded, or bolted fabrication, H–20 loading

Pipe diameter	Minimum cover top of pipe to top of subgrade	5/16-inch rivets or helical				3/8-inch rivets or helical fabrication					
		16 gage		14 gage		12 gage		10 gage		8 gage	
		Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated
Inches		Maximum fill heights above top of pipe in feet									
36	12	30	—	38	44	48	69	56	72	63	75
42	12	26	—	30	38	37	60	42	63	48	65
48	12	23	—	27	34	30	52	34	54	38	57
54	12	20	—	24	29	26	47	29	48	32	50
60	12	19	—	22	26	24	42	26	43	28	45
66	12	17	—	20	24	22	38	23	39	25	41
72	12	15	—	20	22	21	35	22	36	23	38
78	12	14	—	19	21	20	32	21	33	22	35
84	12	—	—	19	—	19	30	20	31	21	32
90	12	—	—	18	—	19	28	19	29	20	30
96	12	—	—	—	—	18	26	19	27	20	28
102	24	—	—	—	—	18	25	19	25	19	26
108	24	—	—	—	—	18	23	19	24	19	25
114	24	—	—	—	—	—	—	18	22	19	24
120	24	—	—	—	—	—	—	18	21	19	22
		Spot welded or bolted (1/2-inch A325 bolts) fabrication									
		Or 3/8-inch rivets				Or 7/16-inch rivets					
36	12	34	43	38	58	48	92	56	106	63	118
42	12	28	38	30	50	37	74	42	84	48	91
48	12	24	32	27	44	30	60	34	68	38	76
54	12	22	29	24	39	26	52	29	58	32	64
60	12	21	25	22	35	24	48	26	52	28	56
66	12	20	23	20	32	22	44	23	46	25	50
72	12	19	22	20	29	21	42	22	44	23	46
78	12	18	20	19	27	20	40	21	42	22	44
84	12	—	—	19	25	19	38	20	40	21	42
90	12	—	—	18	23	19	37	19	38	20	40
96	12	—	—	—	—	18	35	19	38	20	39
102	24	—	—	—	—	18	33	19	36	19	37
108	24	—	—	—	—	18	31	19	34	19	35
114	24	—	—	—	—	—	—	18	32	19	33
120	24	—	—	—	—	—	—	18	30	19	32

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

RECOMMENDED FILL-HEIGHT TABLE 3.—Corrugated steel pipe, 6-inch by 2-inch corrugations, bolted fabrication, H–20 loading

Pipe diameter	Minimum cover top of pipe to top of subgrade	12 gage		10 gage		8 gage		7 gage		5 gage		3 gage		1 gage	
		Four ¾-inch A-325 bolts per foot of seam													
		Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated
Inches		Maximum fill heights above top of pipe in feet													
60	12	42	—	56	62	58	81	63	93	71	(112)	80	(132)	89	(144)
72	12	33	35	36	51	41	67	44	77	49	93	54	(108)	59	(118)
84	12	27	30	30	44	33	57	34	66	37	75	40	80	43	86
96	12	24	26	25	38	27	50	29	58	31	63	33	66	35	70
108	24	22	23	23	34	24	45	25	51	26	53	28	56	29	59
120	24	20	21	21	31	22	40	23	46	24	48	25	50	26	52
132	24	19	—	20	28	21	36	21	42	22	45	23	46	24	48
144	24	17	—	19	25	20	33	20	38	21	42	21	42	22	44
156	24	16	—	19	23	20	31	20	35	20	40	21	41	21	42
168	24	15	—	18	22	19	28	19	33	19	38	20	40	20	41
180	24	14	—	18	20	18	27	18	31	19	37	19	38	19	40
192	24	—	—	18	—	18	25	18	29	19	35	19	38	19	39
204	36	—	—	18	—	18	23	18	27	18	32	19	37	19	38
216	36	—	—	—	—	18	21	18	23	18	27	18	31	19	35
228	36	—	—	—	—	18	—	18	—	18	23	18	26	18	30
240	36	—	—	—	—	—	—	18	—	18	—	18	23	18	26
252	36	—	—	—	—	—	—	—	—	18	—	18	20	18	23

NOTES:

Fill heights exceeding 100 feet (enclosed in parentheses) shall be used only after thorough investigation of foundation material.

For important installations where interruption of traffic would be undesirable or where the cost of replacement would be excessive, a minimum of 10-gage metal shall be used. 257

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

RECOMMENDED FILL-HEIGHT TABLE 4.—Corrugated aluminum pipe, 2⅔-inch by ½-inch corrugations, riveted, welded, or helical fabrication, H–20 loading

Pipe diameter	Minimum cover, top of pipe to top of subgrade	16 gage		14 gage		12 gage		10 gage		8 gage	
		Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated
Inches		Maximum fill heights above top of pipe in feet									
12	12	45	—	45	—	77	—	—	—	—	—
18	12	28	—	31	—	36	—	42	—	49	—
24	12	22	—	23	—	25	—	28	—	31	—
30	12	19	—	20	—	21	31	22	33	24	34
36	12	16	—	16	—	19	25	20	27	21	28
42	12	—	—	18	25	19	35	19	38	20	40
48	12	—	—	—	—	18	24	18	31	19	38
54	12	—	—	—	—	17	—	18	21	18	27
60	12	—	—	—	—	—	—	16	—	18	20
66	12	—	—	—	—	—	—	12	—	15	—
72	12	—	—	—	—	—	—	—	—	12	—

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

RECOMMENDED FILL-HEIGHT TABLE 5.—Corrugated aluminum pipe, 9-inch by 2½-inch corrugations, bolted fabrication, H–20 loading

Pipe diameter	Minimum cover, top of pipe to top of subgrade	¾-inch aluminum bolts										¾-inch A325 steel bolts					
		Plate thickness (in inches)															
		0.09		0.10		0.125		0.15		0.175		0.20		0.225		0.25	
		Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated	Circular	Elongated
Inches		Maximum fill heights above top of pipe (in feet)															
72	12	19	—	22	—	27	29	29	37	31	44	33	50	35	54	37	59
84	12	16	—	19	—	23	25	25	32	26	38	27	42	28	47	30	51
96	12	14	—	16	—	22	—	22	27	23	33	24	37	25	41	26	44
108	24	12	—	14	—	19	—	20	24	21	30	22	33	22	36	23	40
120	24	—	—	13	—	17	—	19	22	20	26	20	30	21	33	21	36
132	24	—	—	12	—	16	—	19	20	19	24	20	27	20	30	20	33
144	24	—	—	—	—	14	—	18	—	18	22	19	25	20	27	20	30
156	24	—	—	—	—	12	—	16	—	18	21	19	23	19	25	19	28
168	24	—	—	—	—	—	—	15	—	17	20	18	21	19	23	19	26
180	24	—	—	—	—	—	—	—	—	17	—	18	20	18	21	18	23

NOTE.—For important installations where interruption of traffic would be undesirable or where the cost of replacement would be excessive, a minimum thickness of 0.15-inch thickness for structural aluminum plate shall be used.

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

RECOMMENDED FILL-HEIGHT TABLE 1a–2a.—Corrugated steel pipe arches, 2⅔-inch by ½-inch and 3-inch by 1-inch corrugations, riveted, welded, or helical fabrications, H–20 loadings

Pipe dimensions—span rise (inches)	Corner radius (inches)	Minimum cover, top of pipe to top of subgrade for 2 tons per square foot (inches)	Minimum gage required	Maximum fill heights above top of pipe (in feet) for the following corner bearing pressures as in tons per square foot	
				2 tons	3 tons*
2⅔-inch by ½-inch corrugations					
18 x 11	3½	18	16	13	15+
22 x 13	4	18	16	12	15+
25 x 16	4	18	16	10	15+
29 x 18	4½	18	16	9	15
36 x 22	5	18	16	9	14
43 x 27	5½	18	16	7	13
50 x 31	6	18	14	7	12
58 x 36	7	18	12	7	12
65 x 40	8	18	12	7	12
72 x 44	9	18	10	7	12
79 x 49	10	18	8	7	12
85 x 54	11	18	8	8	13
3-inch by 1-inch corrugations					
43 x 27	7¾	18	16	12	15+
50 x 31	9	18	16	12	15+
58 x 36	10½	18	16	12	15+
65 x 40	12	18	16	12	15+
72 x 44	13¼	18	16	12	15+
73 x 55	18	18	16	15	------------
81 x 59	18	18	14	15	------------
87 x 63	18	18	14	14	15+
95 x 67	18	18	12	12	15+
103 x 71	18	24	12	11	15+
112 x 75	18	24	12	10	15+
117 x 79	18	24	12	10	15
128 x 83	18	24	10	10	14

*Where bearing pressures exceeding 2 tons per square foot are required for given fill heights, the foundation material shall be investigated to determine its bearing capacity.

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

260

Recommended Fill-Height Table 3a.—Corrugated steel pipe arches, 6-inch by 2-inch corrugations, bolted fabrication, H-20 loading

Pipe dimensions—span rise (in feet)	Corner radius (inches)	Minimum cover, top of pipe to top of subgrade for 2 tons per square foot (inches)	Minimum gage required	Maximum fill heights above top of pipe (in feet) for the following corner bearing pressures in tons per square foot		
				2 tons	*3 tons	*4 tons
6'1" x 4'7"	18	18	12	15	--------	--------
7'0" x 5'1"	18	18	12	15	--------	--------
7'11" x 5'7"	18	18	12	12	15+	--------
8'10" x 6'1"	18	24	12	11	15+	--------
9'9" x 6'7"	18	24	12	10	15	--------
10'11" x 7'1"	18	24	12	9	13	--------
11'10" x 7'7"	18	24	12	7	12	15+
12'10" x 8'4"	18	24	12	6	11	15
14'1" x 8'9"	18	24	12	5	11	14
15'4" x 9'3"	18	24	10	(a)	10	13
15'10" x 9'10"	18	24	10	(a)	9	12
16'7" x 10'1"	18	36	10	(a)	8	12
13'3" x 9'4"	31	24	12	13	15+	--------
14'2" x 9'10"	31	24	12	12	15+	--------
15'4" x 10'4"	31	24	10	11	15+	--------
16'3" x 10'10"	31	36	10	11	15+	--------
17'2" x 11'4"	31	36	10	10	15	--------
18'1" x 11'10"	31	36	8	9	14	--------
19'3" x 12'4"	31	36	8	9	13	--------
19'11" x 12'10"	31	36	8	8	13	--------
20'7" x 13'2"	31	36	7	7	12	--------

*Where bearing pressures exceeding 2 tons per square foot are required for given fill heights, the foundation material shall be investigated to determine its bearing capacity.

(a) Use pipe arches with 31-inch corner radius.

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

Recommended Fill-Height Table 4a.—Corrugated aluminum pipe arches, 2⅔-inch by ½-inch corrugations, riveted, welded, or helical fabrication, H-20 loading

Pipe dimensions—span rise (inches)	Corner radius (inches)	Minimum cover, top of pipe to top of subgrade for 2 tons per square foot (inches)	Minimum gage required	Maximum fill heights above top of pipe (in feet) for the following corner bearing pressures in tons per square foot	
				2 tons	*3 tons
18 x 11	4¾	18	16	15	--------------
22 x 13	4¾	18	16	14	--------------
25 x 16	4½	18	16	13	15+
29 x 18	4½	18	16	11	15+
36 x 22	5	18	16	9	14
43 x 27	5½	18	14	7	13
50 x 31	6	18	12	7	12
58 x 36	7	18	10	7	12
65 x 40	8	18	10	7	12
72 x 44	9	18	8	7	12

*Where bearing pressures exceeding 2 tons per square foot are required for given fill heights, the foundation material shall be investigated to determine its bearing capacity.

This table shows minimum gages for structural requirements only and is intended for use only where corrosive and/or abrasive conditions are negligible. Heavier gages and/or protective coatings shall be used where site investigations recommended in par. 2.9 indicate corrosive and/or abrasive conditions or where anticipated velocities exceed 5 feet per second.

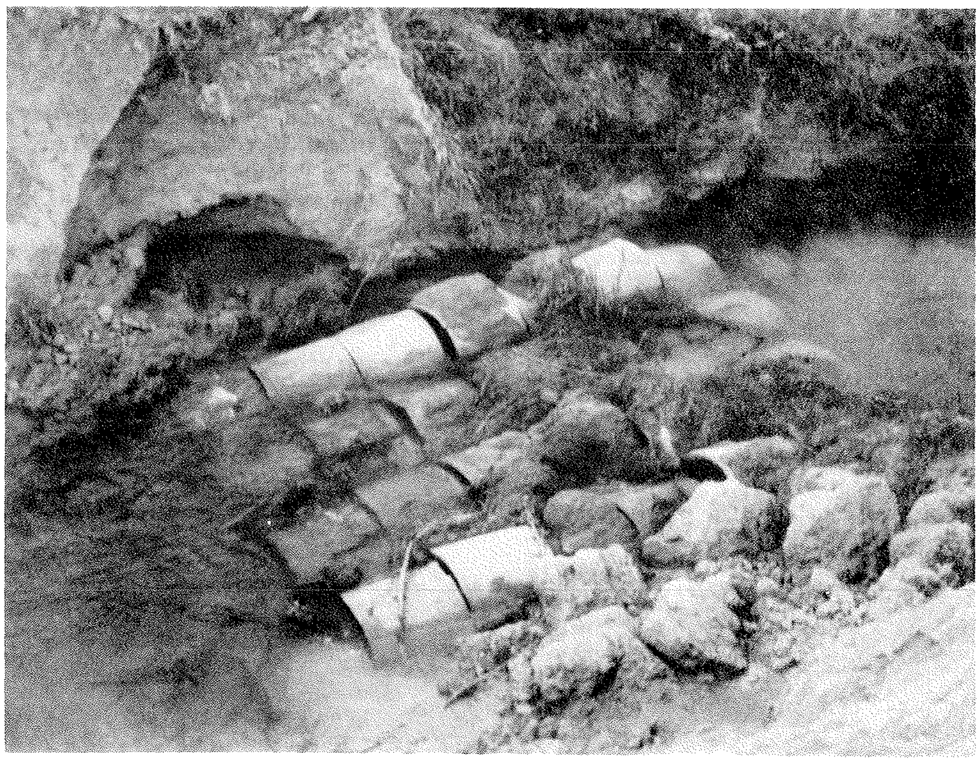
REFERENCES

1. *Soil Engineering*, M. G. Spangler, 1960, International Textbook Co.
2. *Strength of Steel Culvert Sheets Bearing Against Compacted Sand Backfill*, Dr. G. G. Meyerhalf and L. D. Baikse, Highway Research Board, January 1963, and Discussion by Dr. R. K. Watkins, *Some Observations on the Ring Buckling of Buried Flexible Conduits*, Highway Research Board, 1963.
3. *Proposed Structural Design Method for Buried Flexible Conduits*, Dr. R. K. Watkins, December 1962, U.S. Steel Corp.
4. *The Corrugated Metal Conduit as a Compression Ring*, H. L. White and J. P. Layer, Highway Research Board, 1960.
5. *Review of Design Concepts for Aluminum Alloy Corrugated Culverts*, A. H. Koepf, Kaiser Aluminum and Chemical Sales, Inc., 1965.
6. *American Iron and Steel Institute Handbook*, 1966.
7. *Aluminum Alloy Structural Plate Pipe Culverts*, Kaiser Aluminum and Chemical Sales, Inc., 1965.
8. *Field Test for Estimating the Service Life of Corrugated Metal Pipe Culverts*, J. L. Beaton and R. F. Stratful, Highway Research Board Proceedings, 1962.
9. *Culvert Performance Evaluation*, Washington State Highway Commission, Department of Highways, 1964.
10. *A Preliminary Study of Aluminum as a Culvert Material*, Eric Nordlin and R. F. Stratful, Highway Research Record No. 95, 1965.
11. U.S. Steel Corp. Report, 1966.

ACKNOWLEDGMENTS

Acknowledgment is made of the valuable assistance rendered by the National Corrugated Steel Pipe Association and by the Aluminum Association in providing the data used in the preparation of the criteria and design charts.

☆ U.S. GOVERNMENT PRINTING OFFICE: 1966 O—224-693



Multiple 1-m diameter concrete pipe culvert shows damage from washout—Brazil.

REINFORCED CONCRETE PIPE CULVERTS

Criteria for Structural Design and Installation

By the Bridge Division
Office of Engineering and Operations
Bureau of Public Roads

Reported by Merrill Townsend
Bridge Engineer

U.S. DEPARTMENT OF COMMERCE
Luther M. Hodges, Secretary

BUREAU OF PUBLIC ROADS
Rex M. Whitton, Administrator

August 1963

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C., 20402 - Price 30 cents

CONTENTS

FIGURES AND CHARTS

INTRODUCTION

The structural design of a reinforced concrete pipe culvert requires calculation of the probable maximum load on the pipe, determination of the inherent strength of the pipe, and selection of a bedding for the pipe which will insure that the field supporting strength of the completed structure will be adequate. The formulas and load coefficient diagrams necessary to make such calculations are included in these criteria for use when required.

The material contained in these criteria is basically an updating and revision of the Design and Installation Criteria for Reinforced Concrete Pipe Culverts distributed with the Bureau of Public Roads Circular Memorandum dated April 4, 1957, for the purpose of simplifying design methods for reinforced concrete pipe.

Table 1 in that issue has been replaced by Charts I(a) and I(b) which give the load on the pipe at various heights of fill for the four classes of bedding in combination with appropriate projection ratios and provide a visual comparison of the load carrying capacity of pipe for those combinations.

GENERAL COVERAGE

1.1 These criteria cover the determinations of loads on concrete pipe; the determination of pipe strength required for the various classes of bedding and types of installations; the classes of bedding; and recommended installation practices.

1.2 Pipe. Reinforced concrete pipe shall be of the classes specified in AASHO Specification M 170–60 (ASTM C 76–59T). The strength test requirements specified therein are given in table 1 and are expressed in D-loads as determined by the three-edge-bearing method.

TABLE 1.—Strength requirements for pipe

[In pounds per linear foot per foot of internal pipe diameter]

Test D-load to produce	Strength test D-load requirements				
	Class I	Class II	Class III	Class IV	Class V
0.01-inch crack	800	1,000	1,350	2,000	3,000
The ultimate	1,200	1,500	2,000	3,000	3,750

DESIGN

2.1 Factors affecting strength. The strength required for a given rigid type of pipe depends upon its size; the height, character, and weight of fill over the pipe; the character of the foundation; the depth and width of trench (if any) in which the pipe is installed; the class of bedding; and the type of installation.

266

2.2 Definitions of terms used.

D-load is a term used to designate the load per linear foot of pipe for each foot of internal pipe diameter and is expressed in pounds or kips.

Load factor, L_f, is defined as the ratio of the strength of a pipe under a specific condition of loading to its strength when tested by the three-edge-bearing method.

Projection ratio, p, is defined as the ratio of the distance of the existing ground surface below the exterior pipe top to the outside diameter of the pipe B_c. It applies to positive and zero projecting embankment installations [shown in figure 6(a)].

Projection ratio p' is defined as the ratio of the distance of the existing ground surface above the exterior pipe top to the width of the trench B_d. It applies to negative projecting embankment installations [shown in figure 6(b)].

Existing ground surface may be either natural ground surface or top of compacted fill at the time of installation.

Settlement ratio r_{sd} is a value determined by an equation involving deflection of pipe, settlement of pipe flow line, settlement of embankment subgrade adjacent to pipe, and deformation of fill material adjacent to and between top of pipe and existing ground surface. The following values for r_{sd} are suggested (Soil Engineering by M. G. Spangler):

For positive projecting embankment installations:

Rigid pipe on rock or unyielding solid: $+1.0$
Rigid pipe on ordinary soil bed: $+0.5$ to $+0.8$
Rigid pipe on slightly yielding bed: 0 to $+0.5$

For negative projecting embankment installations:

Rigid pipe on average soil bed: -0.3 to -0.5

Trench width B_d is defined as the width of trench measured at the top of the pipe. A value of $B_d=1.35\ B_c$ was used for the negative projecting installation curves in Chart I(b).

2.3 Determination of loads on pipe.

2.3.1 The formulas necessary to compute W_c, the weight of the column of earth carried by the pipe, the load factor L_f, and the D-load of the pipe are given in figure 1. The symbols used in the formulas are defined below.

B=interior (normal) pipe diameter
B_c=exterior pipe diameter
B_d=trench width

C_c=load coefficient for positive and zero projecting installations. See figure 2.

C_d=load coefficient for trench installations. See figure 3.

C_n=load coefficient for negative projecting and imperfect ditch installations. See figure 4.

$F.S.$=factor of safety used in selection of class of pipe.

H=height of fill over top of pipe in feet.

K=Rankin's lateral pressure ratio.

L_f=load factor for determining strength of pipe.

m=fractional part of B_c over which the lateral pressure is assumed to act. It may or may not be the same as p.

p=projection ratio.

q=ratio of total lateral pressure to total vertical load W_c.

w=weight per cubic foot of fill.

W_c=total vertical load on pipe in pounds per linear foot.

N and N'=parameters which are a function of the bedding, see table 2.

x and x'=parameters which are a function of the vertical projection m, over which the lateral pressure is assumed to act, see table 3.

TABLE 2.—Values of parameters x and x'

Value of m	Value of x	Value of x' (for Class A bedding only)
0	0	0. 150
0. 3	0. 217	0. 743
0. 5	0. 423	0. 856
0. 7	0. 594	0. 811
0. 9	0. 655	0. 678
1. 0	0. 638	0. 638

TABLE 3.—Values of parameters N and N'

Bedding class	N	N'
Class A	--------	0. 505
Class B	0. 707	--------
Class C	0. 840	--------
Class D	1. 31	--------

2.3.2 Computation of D-loads by formula. Figure 1 lists under each type of installation the formulas required to compute W_c, L_f, and the D-load. The type of installation must be known or assumed in order to determine which load coefficient diagram (figure 2, figure 3, or figure 4) should be used. Trench installations are seldom used for highway culverts and the formulas (and figure 3) for that type are shown for information only. No further explanation of their use will be given.

With the type of installation known and with a given height of fill and diameter of pipe, an appropriate settlement ratio, r_{sd} (see paragraph 2.2), and value of p (or p') must be selected, the load coefficient C_c or C_n taken from the appropriate diagram, and W_c computed. After computing q from the formula, the class of bedding is selected and load factor, L_f, computed with value x (or x') taken from table 2 and value of N (or N') taken from table 3. The D-load value is then computed from the basic D-load formula. High D-load values beyond the class V pipe range will require recomputation using a more favorable type of installation and/or class of bedding.

2.3.3 Computation of D-loads by charts. Chart I(a), Chart I(b), and Chart II have been developed to simplify and facilitate the determination of D-loads. Charts I(a) and I(b) are based on a pipe diameter B=60 inches, B_c=72 inches, r_{sd}=+0.5 for positive and zero projecting installations, and −0.3 for negative projecting and imperfect ditch installations. A value of B_d=1.35 B_c is used for negative projecting installations. 267

On Charts I(a) and I(b) the actual D-load is determined by taking the value of H (height of fill) on the vertical ordinate and following horizontally to its intersection with the ray line for the class of bedding and projection used. The D-load value may then be read at the bottom (or top) of the chart. D-loads for intermediate values of p may be obtained by interpolating between the ray lines for zero and maximum value of p for that class of bedding. Charts I(a) and I(b) provide a visual comparison of the relative merits of the classes of bedding for various fill heights.

Chart II is a nomographic chart for positive and zero projecting installations and provides for determination of D-loads and selection of class of pipe on the basis of a 1.33 factor of safety, varying values of r_{sd} p, and a fixed value of q=0.18 The chart is a combination of four diagrams described as follows: The upper right quadrant diagram gives the load factor, L_f, in terms of p and the class of bedding. The upper left quadrant

FORMULAS FOR DETERMINATION OF LOADS ON PIPE

Positive and zero projecting embankment installations

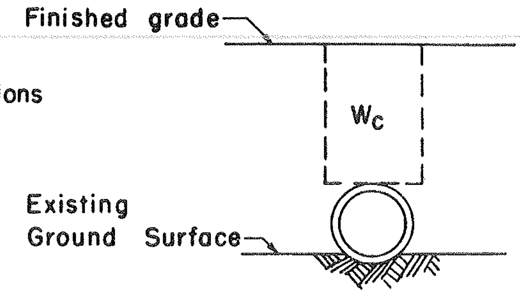
$W_c = C_c w B_c^2$

$L_f = \dfrac{1.431}{N - xq}$ where $q = \dfrac{mk}{C}\left(\dfrac{H}{B_c} + \dfrac{m}{2}\right)$ for positive projections

Use 50% of q value for zero projections

D-load strength required $= \dfrac{W_c}{L_f B}$

Formula applies to values of p from 0 to 1



Negative projecting embankment installation

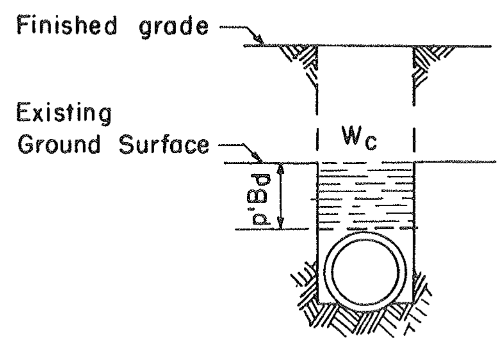
$W_c = C_n w B_d^2$

$L_f = \dfrac{1.431}{N - xq}$ where $q = \dfrac{\text{lateral pressure}}{W_c}$

Lateral pressure $= \left(H + \dfrac{mB_c}{2}\right)\dfrac{wkmB_c}{2}$

D-load strength required $= \dfrac{W_c}{L_f B}$

Formula applies to values of p' greater than 0.



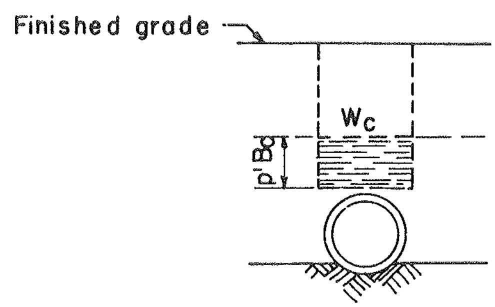
Conventional imperfect ditch installation

$W_c = C_n w B_c^2$

$L_f = \dfrac{1.431}{N - xq}$ where $q = \dfrac{mk}{C}\left(\dfrac{H}{B_c} + \dfrac{m}{2}\right)$

D-load strength required $= \dfrac{W_c}{L_f B}$

Formula applies to values of p' greater than 0.



Trench installation

$W_c = C_d w B_d^2$

D-load strength required $= \dfrac{W_c}{L_f B}$

L_f values have been determined experimentally as follows:

Bedding	L_f
Class A	2.2-3.4
Class B	1.9
Class C	1.5
Class D	1.1

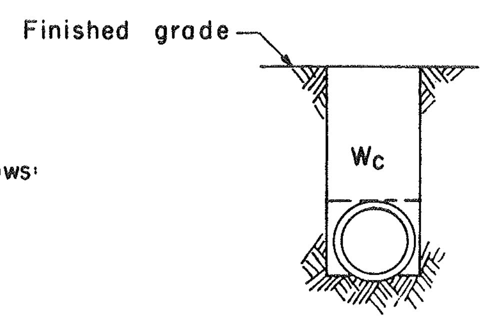
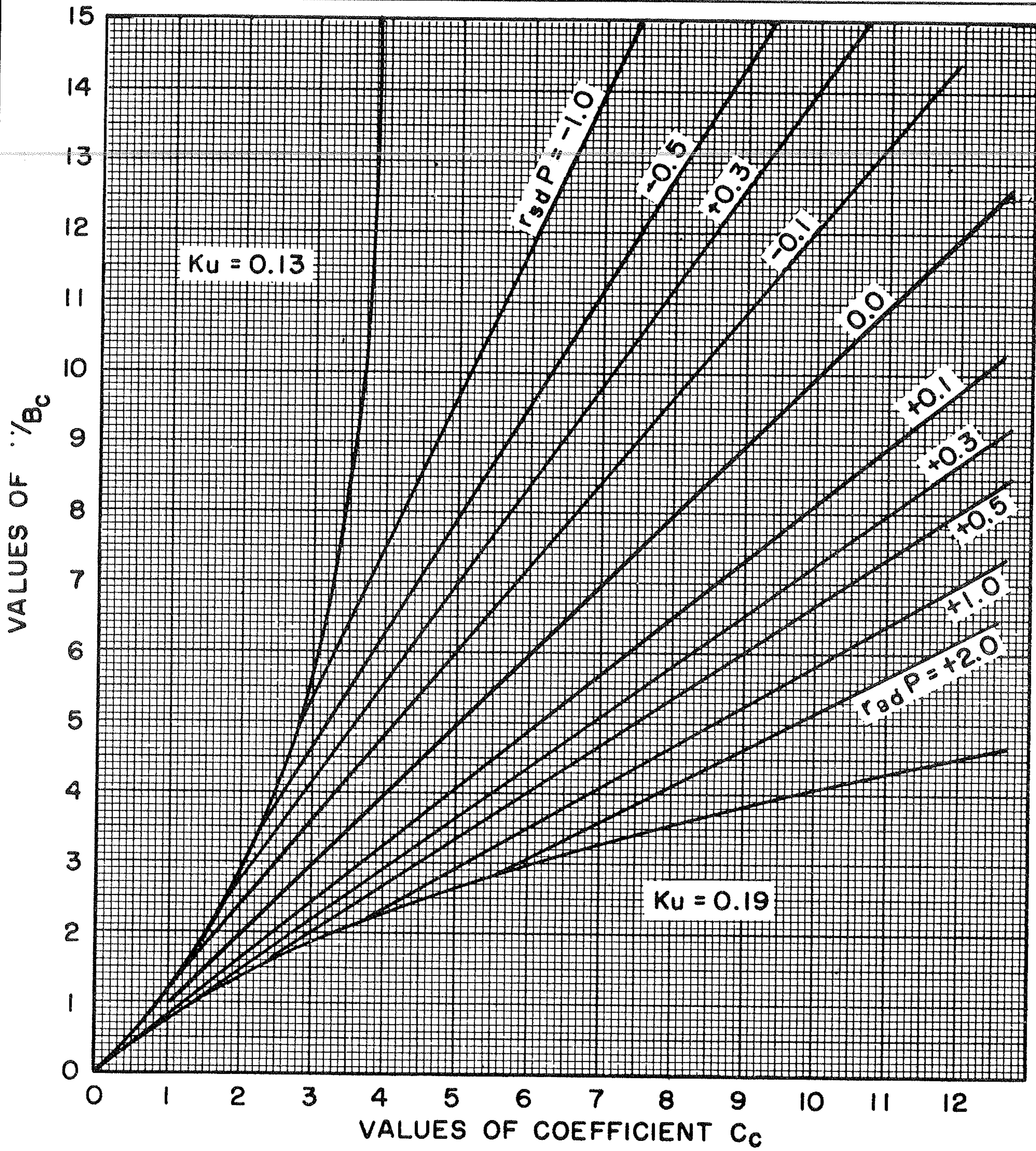


FIGURE 1

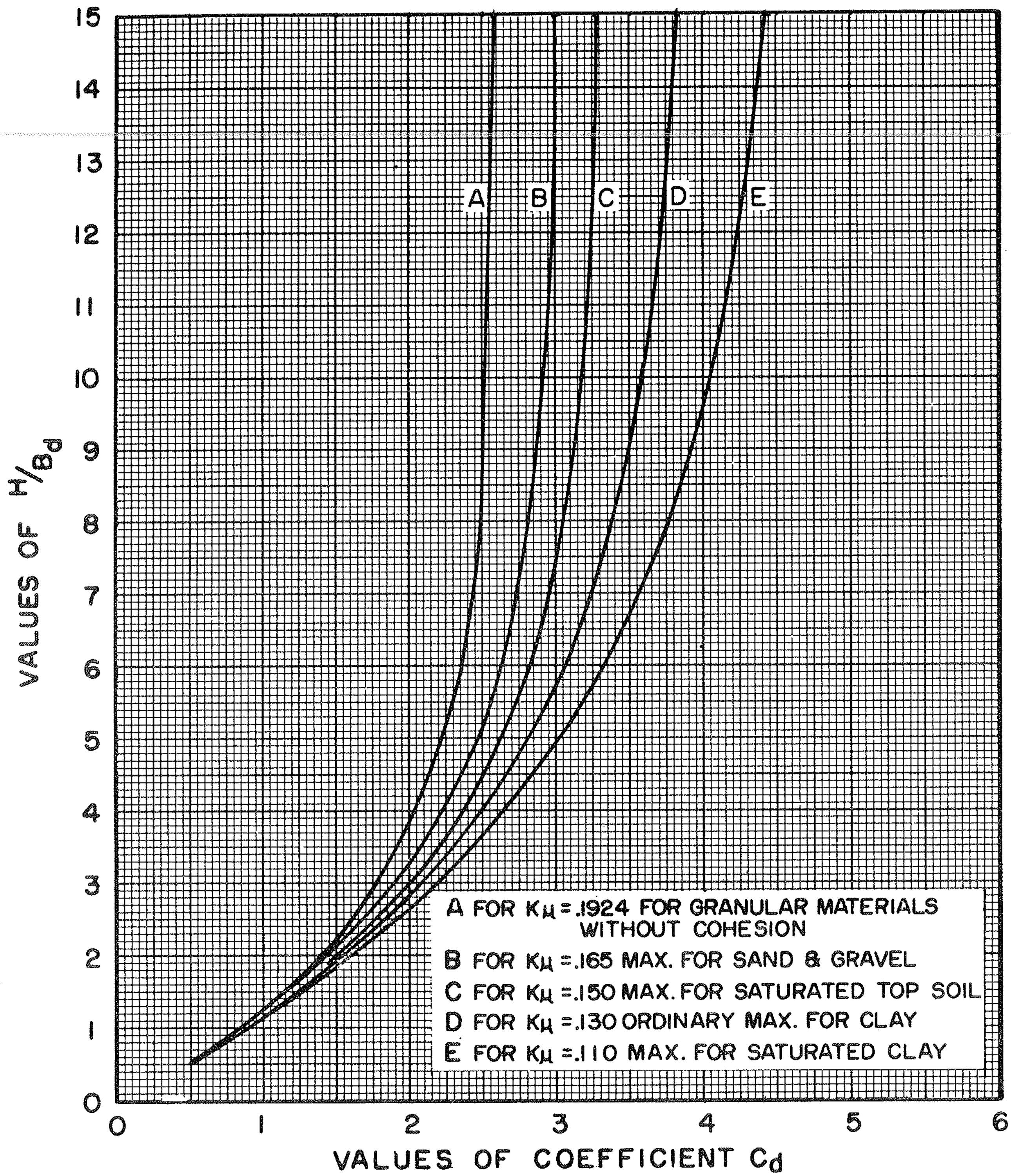


POSITIVE AND ZERO PROJECTING INSTALLATIONS

FIGURE 2

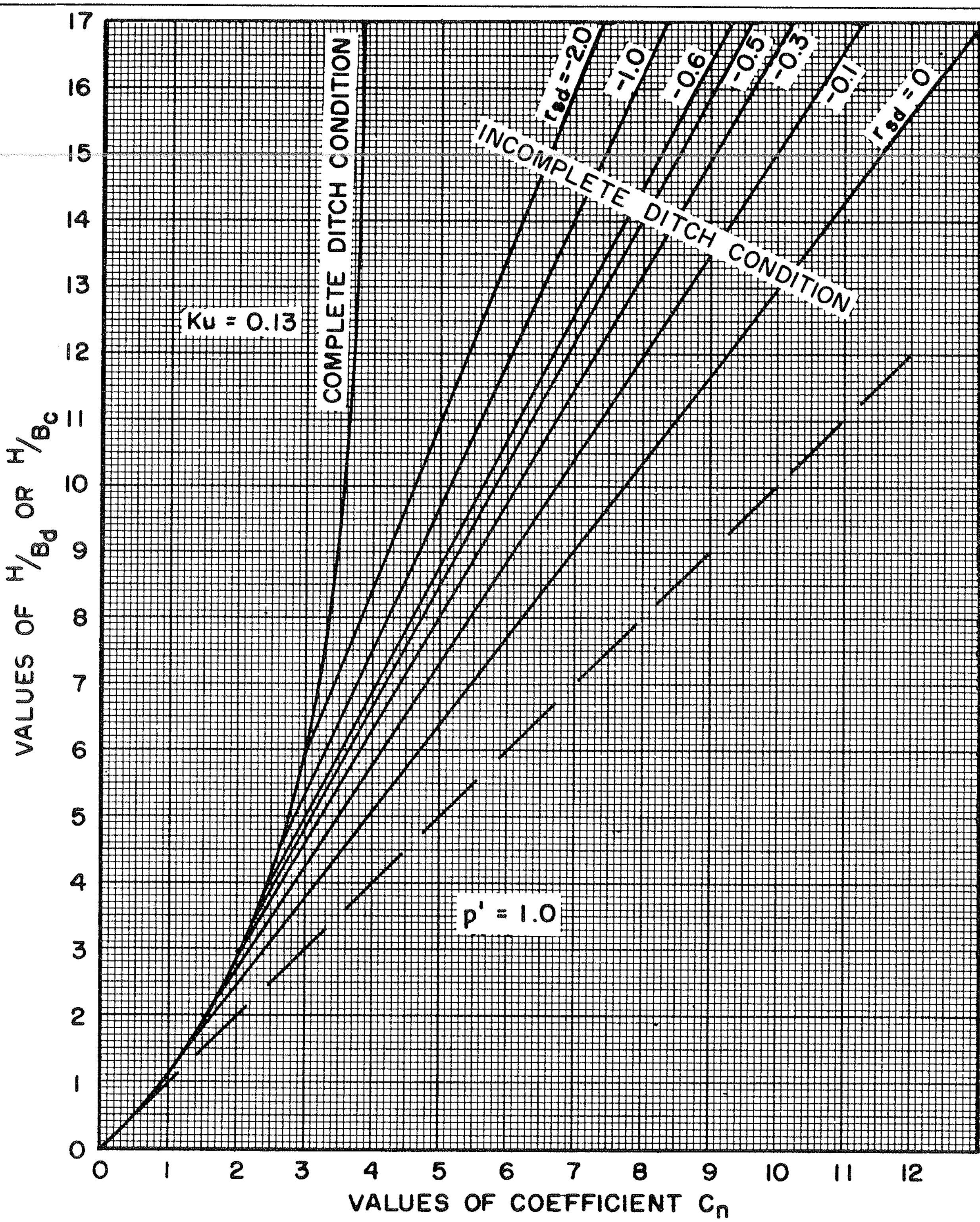
269

270



TRENCH INSTALLATIONS

FIGURE 3



NEGATIVE PROJECTING AND IMPERFECT DITCH INSTALLATIONS
FIGURE 4

272

diagram gives the safe uniform load W_c/B_c for a given strength pipe and load factor, L_f. The lower left quadrant gives C_c in terms of B_c and W_c/B_c and the lower right quadrant contains a load coefficient diagram that gives values of C_c in terms of H/B_c and $r_{sd}\ p$. This diagram is the same as that shown in figure 2 except that it gives the values of H/B_c and C_c up to 20.

The chart does not provide a visual comparison of the relative merits of the classes of bedding but is useful where different values of r_{sd} must be used. The chart may be used to determine required D-loads for trench installations by using curve OT in the lower right quadrant. The rest of the chart is used by substituting B_d for B_c throughout the chart and by changing the D values given in the upper left quadrant by the factor B_d/B_c. Also the appropriate value of the load factor for the class of bedding (given in figure 1 for trench installations) should be used on the L_f ordinate in determining the value of D.

2.3.4 Selection of class of pipe. After the D-load has been computed by the appropriate formula (see figure 1) the class of pipe may be determined either on the basis of the 0.01 inch crack strength or on the basis of the ultimate strength to which a factor of safety has been applied (ultimate strength $\div F.S.$). The factors of safety most commonly used are 1.5 and 1.33, the latter having been used in the 1957 issue of the Bureau of Public Roads Design and Installation Criteria for Reinforced Concrete Pipe. The 0.01 inch crack basis and the 1.5 $F.S.$ basis give identical results for classes I to IV pipe but for Class V pipe the 0.01 inch crack basis permits about 20 percent higher fill because the ultimate strength is only 25 percent higher than the 0.01 inch crack strength. Charts I(a) and I(b) show at the top of the chart the pipe class strength ranges based on three-edge-bearing tests and on factors of safety of 1.5 and 1.33. The class of pipe can be readily determined from the height of fill and the appropriate ray line for class of bedding.

2.3.5 Examples of design

(a) Use of formulas:

Given: $B=5$ feet ($B_c=6$ feet)

$H=30$ feet; then $H/B_c=5$

$w=120$ pounds per cubic foot

Try Class C bedding, zero projection, using figure 2, for $r_{sd}p=0$, and $H/B_c=5$, $C_c=5$

$W_c=5\times120\times6^2=21{,}600$ pounds (figure 1, $W_c=C_c\times wB_c^2$)

$$q=\frac{m\cdot k}{C_c}\,\frac{1}{2}\left(\frac{H}{B_c}+\frac{m}{2}\right)=\frac{.7\times.33}{2\times5}\left(5+\frac{.7}{2}\right)=.124$$

for Class C bedding, $N=.84$ and $x=.594$ for $m=.7$ (tables 2 and 3)

$$L_f=\frac{1.431}{.84-.124\times.594}=1.87$$

$$\text{D-load}=\frac{21{,}600}{5\times1.82}=2{,}320 \text{ pounds}$$

Class V pipe: ultimate strength (3,750 pounds) after applying a $F.S.$ of $1.5=2{,}500$ pounds, so a class V pipe is adequate. (The $F.S.$ may be applied by dividing into the ultimate strength of the pipe or by multiplying the computed D-load.)

(b) Use of Charts I (a) and I (b)

Given: Same problem as in (a)

Use Chart I (a) for positive projections.

Take the 30-foot point on the height of fill ordinate and follow horizontally to the right to the intersection with the ray line for Class B, $p=0$. Reference to the upper part of the chart will show that a class IV pipe is adequate for a $F.S.$ of 1.5 or 1.33. If the 30-foot H is followed farther to its intersection with the ray line for Class C, $p=0$, a class V pipe will be found adequate by either $F.S.$ Following this 30-foot H still farther to its intersection with the ray line for Class B, $p=.7$, a class V pipe will be found adequate for the 1.33 $F.S.$ and about 5 percent under for a 1.5 $F.S.$ This gives the designer a choice of pipe class and class of bedding based on installation economics. If the height of fill was considerably greater, a negative projecting or imperfect ditch installation would be required and Chart I (b) would be used in a similar manner for design.

(c) Use of Chart II.

Given: The problem in (a) $H=30$ feet, $B=5$ feet, $r_{sd}=+0.5$

Try Class B bedding, $p=0.7$

$H/B_c=5$, $r_{sd}\ p=.35$

On the H/B_c axis spot the value of $H/B_c=5$ and follow downward vertically to the intersection with the $+0.35$ ray line, thence horizontally to the left to the intersection with the $B_c=6$ ray line in the lower quadrant.

used. Joints shall be made with: (1) portland cement mortar; (2) rubber gaskets; (3) oakum and portland cement mortar; (4) bituminous sealing compound, hot-poured, or cold applied; or (5) a combination of these materials unless one type or combination is specified by the engineer.

3.3.3.1 Portland cement mortar. The mixture shall be one part portland cement and two parts sand by volume. The quantity of water in the mixture shall be sufficient to produce a soft workable mortar but shall in no case exceed six gallons of water per sack of cement. The sand shall conform to AASHO Specification M45–42 and the cement shall conform to AASHO Specification M85–60. If ordered by the engineer air entraining portland cement conforming to AASHO Specification M134–60 shall be used. The first pipe shall be bedded carefully to the established grade line with the groove upstream. A shallow excavation shall be made underneath the pipe at the joint and filled with mortar to provide a bed for the pipe.

The pipe ends shall be thoroughly cleaned and wetted with water before the joint is made. A layer of mortar shall then be placed in the lower half of the bell or groove of the pipe section already laid. Next, mortar shall be applied to the upper half of the tongue. The spigot or tongue end of this pipe shall then be inserted in the bell or groove end of the pipe already laid until mortar is squeezed out on the interior or exterior surface. Sufficient mortar shall be used to fill the joints of tongue and groove pipe to fill the joint completely and to form a bead on the outside of the pipe. Inside of the joint shall be wiped clean and finished smooth. In pipe too small for a man to work inside, wiping may be done by dragging a swab or long-handled brush through the pipe as work progresses. The mortar bead on the outside shall be protected from air and sun with a proper covering until satisfactorily cured. No backfilling around the joints shall be done until the joints have been fully inspected and approved.

3.3.3.2 Rubber gasket joints. The pipe and gaskets shall conform to the requirements of AASHO Specification M198–62I. Gaskets and jointing materials shall be placed in accordance with the recommendation of the particular manufacturer in regard to the use of lubricants, cements, adhesives, and other special installation requirements. Surfaces to receive lubricants, cements, or adhesives shall be clean and dry. Gaskets and jointing materials shall be affixed to the pipe not more than 24 hours prior to the installation of the pipe, and shall be protected from the sun, blowing dust, and other deleterious agents at all times. Gaskets and jointing materials shall be inspected before installation of the pipe and any loose or improperly affixed gaskets and jointing materials shall be removed and replaced to the satisfaction of the engineer. The pipe shall be aligned with the previously installed pipe, and the joint pulled together. If, while making the joint, the gasket or jointing material becomes loose and can be seen through the exterior joint recess when the joint is pulled up to within one inch of closure, the pipe shall be removed and the joint remade to the satisfaction of the engineer.

3.3.3.3 Oakum and portland cement. A closely twisted gasket of diameter required to support the spigot of the pipe at the proper grade and to make the joint concentric, and conforming to Federal Specification H–H–P–117, shall be used. The joint packing shall be in one piece of sufficient length to pass around the pipe and lap at the top. This gasket shall be thoroughly saturated with neat cement grout. The bell of the pipe shall be thoroughly cleaned with a wet brush and the gasket shall be laid in the bell for the lower third of the circumference and covered with mortar. The spigot of the pipe shall be thoroughly cleaned with a wet brush and inserted in the bell and carefully driven home. A small amount of mortar shall be inserted in the annular space for the upper two-thirds of the circumference. The gasket shall be lapped at the top of the pipe and driven home in the annular space with a calking tool.

The remainder of the annular space shall then be filled completely with mortar and beveled off at an angle of approximately 45 degrees with the outside of the bell. If the mortar is not sufficiently stiff to prevent appreciable slump before setting, the outside of the joint thus made shall be wrapped with cheese cloth. The finishing of this type of joint shall be kept at least five joints behind the laying operation. The portland cement mortar, finish, and protection shall be as specified in paragraph 3.3.3.1.

3.3.3.4(a) Bituminous sealing compound, hot-poured. Before jointing, the inside of the bells and the outside of the spigot shall be dry and clean.

The pipe shall be centered in the annular space. Annular space shall be calked with joint packing conforming to Federal Specification H–H–P–117 or H–H–P–119 and shall be sealed with a joint compound conforming to Federal Specification SS–S–169. The joint packing shall be thoroughly corded and finished and practically free from lumps, dirt, and extraneous matter. The fibers shall be thoroughly impregnated with a hot asphaltic cement. The depth of the packing shall be such as to leave a space between the surface of the packing and the end of the bell as follows: at least one inch for pipes 15 inches and less in diameter, one and one-half inches for pipes 18 to 24 inches in diameter, and two inches for pipes larger than 24 inches in diameter. When the jointing is made with pipe in its final location, a joint runner, previously dipped into thick mud or grout to permit easy removal when the point is cooled, shall be placed around the pipe, leaving an opening at the top of the runner. Molten class 1 bituminous compound shall be poured continuously into this opening until the joint is completely filled and shall be poured as rapidly as possible without entrapping air. After the compound has cooled or set, the runner may be removed. Alternate joints may be poured before

276 the pipe is lowered into the trench. In this case the joint shall be poured with the pipe in a vertical position without the use of the runner. The compound shall have thoroughly set before the pipe is placed in the trench and the pipe shall be handled so as not to cause deformation of the joint. In cold weather special care shall be exercised to assure that the compound is not cooled too rapidly for proper adhesion and, if necessary, the pipe shall be preheated. The temperature of the molten compound shall be be between 350 degrees F. and 450 degrees F. unless otherwise recommended by the manufacturer. The compound shall not be overheated or subjected to such prolonged heating as might cause a change in its physical properties.

3.3.3.4(b) Bituminous sealing compound, cold applied. The annular space between the bell and the spigot of the pipe shall be dry and clean and shall be packed with an asphalt-saturated, cellulose-fiber packing conforming to Federal Specification H–H–P–119. The packing shall be of a size suitable for the annular space and shall be cut in lengths to encircle the pipe completely. The first strands shall be calked solidly against the back of the bell. Additional strands shall be placed and calked solidly in the bell to fill one-third to one-half of the annular space. The annular space shall then be filled completely with a joint sealer conforming to Federal Specification SS–S–168. Overfilling is not required. The sealer shall be mixed on the job in accordance with the manufacturer's recommendations and in small enough quantities so that appreciable setting will not occur before use.

3.3.4 Camber. The invert grade of the pipe shall be cambered sufficiently to prevent the development of a sag or back slope in the flow line as the foundation material settles under the weight of the embankment. The amount of camber shall be determined by the engineer based upon consideration of the flow line gradient, height of fill, compressive characteristics of the foundation material, and depth to rock. In no case shall the camber be sufficient to produce an adverse grade after settlement has occurred.

3.4 Backfilling for pipe. It is essential that the backfill material at the sides and top of pipe be placed and compacted in such a manner as to develop the computed lateral pressures used in design.

3.4.1 Backfill material. Backfill material within a nominal pipe diameter B at the sides of pipe and to one foot above the top thereof shall be fine readily compactible job excavated soil material or granular fill material. It shall not contain stones which would be retained on a two-inch ring, frozen lumps, chunks of highly plastic clay, or other objectionable material. Granular fill material shall be crushed stone or pea gravel with not less than 95 percent passing a one-half-inch sieve and not less than 95 percent being retained on a number 4 sieve.

3.4.2 Backfilling operation for positive projecting installations, figure 6(a). Backfill material shall be compacted at near optimum moisture content in layers not exceeding 6 inches (compacted) for a width on each side of the pipe equal to 2 B_c or 12 feet, whichever is less, and shall be brought up evenly on both sides of the pipe for its full length up to one foot above top of pipe. The fill material under the haunches and adjacent to the sides of pipe shall be thoroughly compacted by pneumatic tampers or hand methods to develop lateral pressure. The remainder of fill may be compacted by rolling (or other approved methods) in a direction parallel with the sides of pipe, but care must be exercised to avoid displacement of, thence upward vertically into the upper left quadrant to the intersection with the ray line for the D-loads. Also, take the $p=0.7$ value on the p axis in the upper right quadrant, proceed upward vertically to the interesction with the Class B curve, thence to the left horizontally to the interesection with the previously projected vertical line in the upper left quadrant and read a D-load (by interpolation between heavy D-load ray lines), based on the ultimate D-load strength of the pipe and a class V pipe is found adequate.

INSTALLATION

3. Installation of concrete pipe covers the construction of the bedding, laying of pipe, and the backfilling around and over the pipe.

3.1 Bedding of pipe. The contact between a pipe and the foundation upon which it rests is the pipe bedding. It has an important influence on the ability of the pipe to support loads. Class of bedding is defined by the width of the band of contact between the pipe and its foundation and four classes of bedding are designated. The class of bedding to be provided shall be determined by the designer and specified on the plans.

3.1.1 Bedding material. Where granular materials specified for bedding, it shall be fine granular material meeting the sand grading requirements found in AASHO Specification M6–51 (or ASTM C33–59).

3.1.2 Class A bedding. In this class of bedding (often called concrete cradle bedding) the lower exterior part of the pipe shall be bedded in a continuous cradle constructed of class *B* concrete (2,200 pounds per square inch) or better, having a minimum thickness under the pipe of one-fourth the internal diameter B and extending up the sides of the pipe for a height equal to one-fourth the exterior diameter B_c. If the cradle is on reasonably sound rock (not boulders or fragmented shale) the minimum thickness under the pipe may be reduced from one-fourth B to 6 inches. The cradle should, however, extend into the rock several inches to develop resistance against lateral pressure. The cradle shall have a width at least equal to B_c plus 8 inches and shall be constructed monolithically without horizontal construction joints. Backfill shall be placed as specified in 3.4, Backfilling. A typical Class A bedding is illustrated in figure 5(a).

3.1.3 Class B bedding. With this class of bedding, the projection ratio shall not exceed 0.7. The pipe shall be carefully bedded on fine granular materials or sand over an earth foundation accurately shaped by means of a template to fit the lower 15 percent of its height B_c. Compactible soil material shall then be rammed and tamped in 6-inch layers around the pipe for the remainder of the lower 30 percent of B_c. Backfill shall then be compacted as specified in 3.4, Backfilling. A typical Class B bedding is illustrated in figure 5(b).

3.1.4 Class C bedding. With this class of bedding, the projection ratio shall not exceed 0.9. The pipe shall be bedded with ordinary care in a soil foundation shaped to fit the lower part of the pipe exterior with reasonable closeness for at least 10 percent of its overall height. Backfill shall then be completed as specified in 3.4, Backfilling. A typical Class C bedding is illustrated in figure 5(c). 273

3.1.5 Class D bedding. This class of bedding requires no shaping of the bed but the gradient of the bed shall be smooth and true to established grade. The backfill shall be placed as specified in 3.4, Backfilling.

3.1.6 Bedding for pipe arches, horizontal elliptical, and vertical elliptical pipe. It is equally important that such pipe be adequately bedded and backfilled and Class B or Class C bedding specified for circular pipe is recommended basing the projection ratio p on the exterior vertical diameter. Backfill should be as specified in 3.4, Backfilling.

3.2 Types of installations. The type of installation constructed in conjunction with the class of bedding also has an important influence on the ability of the pipe to support loads. For example, with a Class B bedding a zero projection ($p=0$) type provides about 33 percent increase in height of fill the pipe can carry over that for $p=0.7$; a negative projecting ($p'=1$) type provides about 62 percent increase, and an imperfect ditch type provides about 160 percent increase above that for $p=0.7$. The types of installations are illustrated in figure 6.

3.2.1 Imperfect ditch (imperfect trench) installation. This method is applicable only for pipe bedded as positive projecting. The pipe shall first be bedded in accordance with the requirements for the class of bedding specified. The fill shall then be placed and compacted on both sides of the pipe as specified in 3.4, Backfilling, for a lateral distance equal to 12 feet or 2 B_c, whichever is less, and up to an elevation equal to $p'B_c$ plus one foot above the top of pipe. Next a trench of B_c width shall be dug in the fill directly over the pipe down to an elevation of one foot above the pipe top. The trench should not be constructed over the entire length of the installation but only where needed for the section under the higher fill. Care should be exercised to keep the sides of this re-excavated trench as nearly vertical as possible. This trench shall then be filled with loose highly compressible soil material. Straw, hay, corn stalks, leaves, brush, or sawdust may be used to fill the lower one-fourth to one-third of the trench to insure maximum compressibility of this backfill. After the special backfill has been completed to the top of trench the fill above that elevation is placed and compacted in accordance with normal methods for embankment fill. A typical imperfect ditch installation ($p'=1.0$) is illustrated in figure 6(c). This method is usually used in conjunction with Class B or Class C bedding but may be used with Class A bedding.

274

3.2.2 Negative projecting embankment installations. In this type the top of pipe must be below the top of existing ground surface. A trench is dug in the earth to the required elevation and the pipe is installed in accordance with the requirements for the class of bedding specified and backfilled in accordance with 3.4, Backfilling, up to one foot above the pipe top. The remainder of the trench shall then be filled with loose material spread evenly up to the top of trench. Above that elevation fill shall be placed as a normal embankment fill. Care should be exercised in digging the trench, to keep its width B_d to the minimum width consistent with providing adequate bedding and backfilling. For excessive widths of trench the installation should also be computed as a positive projecting type.

3.2.3 Rock or incompressible foundation. Where ledge rock, rocky or gravelly soil, hard pan, or other unyielding material is encountered, the pipe shall be bedded in accordance with the requirements of one of the bedding classes but with the following additions: the hard unyielding material shall be excavated below the bottom of the pipe or pipe bell to a depth of 12 inches or one-half inch for each foot of fill over the pipe, whichever is greater, but need not exceed three-quarters of the nominal pipe diameter B. For Class D bedding, the depth of excavation shall be 8 inches. The excavation shall be one foot wider than the exterior pipe diameter B_c and shall be refilled with fine compressible material such as silty clay, loam, or sand and lightly compacted and shaped as required for the specified class of bedding. A typical bedding on an incompressible foundation is illustrated in figure 5(e).

3.3 Laying pipe. The necessary facilities shall be provided for lowering and properly placing the pipe sections. Pipe laying shall begin at the downstream end of the installation with the bell or groove end of the first section upstream. The pipe shall be laid to the lines and grades specified with the pipe sections closely jointed. When bell and spigot pipes are used bell holes shall be dug in the subgrade to accommodate the bells. They shall be deep enough to insure that the bell does not bear on the bottom of the hole but shall not be excessively wide in the longitudinal direction of the installation.

When the pipe sections are laid the barrel of each section shall be in contact with the quadrant shaped bedding throughout its full length exclusive of the bell. Where lift holes in the pipe have been provided such holes shall be refilled with an acceptable grade of concrete after laying and the concrete shall be thoroughly cured before backfill material is placed.

3.3.1 Elliptical pipe. When elliptical pipe with circular reinforcement or circular pipe with elliptical reinforcement is used, the pipe shall be installed in a position that the manufacturer's marks designating "top" and "bottom" of the pipe shall be not more than 5° from the vertical plane through the longitidinal axis of the pipe.

3.3.2 Multiple pipe installations. Where multiple lines of pipe are used, they shall be spaced far enough apart to permit thorough tamping of the earth between the pipe. To this end, the adjacent sides of the pipe shall be at least one-half the nominal pipe diameter apart or three feet, whichever is less.

3.3.3 Jointing pipe. Unless otherwise specified, one of the following methods of jointing bell and spigot pipe and tongue and groove pipe shall be or damage to, the pipe. Backfill not within one pipe diameter B at sides of pipe may be regular embankment fill material.

3.4.3 Backfilling operation for zero and negative projecting installations, figures 6(a) and 6(b). After the pipe has been installed the backfill material shall be placed and compacted in 6-inch layers at near optimum moisture content on both sides of the pipe up to one foot above the pipe top with pneumatic or hand tampers. Care shall be exercised to thoroughly compact the fill under the haunches of the pipe. For negative projecting installations the remainder of the trench shall be filled with a loose compressible material spread evenly up to the top of the trench with no compaction thereof.

3.4.4 Backfilling operations: general. In all backfilling operations care should be exercised and it shall be the contractor's responsibility to see that the pipes are not damaged by vertical or lateral forces imposed during installation and by compaction of backfill. Circular pipe with elliptical reinforcement, and elliptical pipe with circular reinforcement, are particularly vulnerable to damage by careless compaction of backfill and it may be necessary to install horizontal timber struts until the fill over the pipe has been completed. All pipe after being bedded and backfilled as specified should be protected by a 4-foot cover of fill before heavy construction equipment is permitted to cross during construction of embankment fill.

3.5 Care in handling pipe. In transporting concrete pipe from truck to its final location, reasonable care should be exercised in unloading to avoid damage to the pipe. Concrete pipe should not be rolled down embankments in such a way that it rolls without control and if pipe sections are moved along the ground with bulldozers or tractors great care should be taken to avoid damage thereto.

3.6 Inspection. Installation conditions have a very important effect on both the load and the supporting strength of the pipe and a satisfactory installation requires attainment of the design conditions in the field. Consequently the engineer on the job should not only be familiar with good installation practices but should also keep a close check on the contractor's operations to insure fulfillment of that objective.

277

ACKNOWLEDGMENTS

Acknowledgment is made to the valuable assistance rendered by M. G. Spangler, Research Professor, Iowa State University, and by the American Concrete Pipe Association and their Washington Representative, John A. Ruhling, in the preparation of the criteria.

REFERENCES

Spangler, M. G., Soil Engineering, International Text Book Co., Second Edition, 1960.

Babcock, Dudley P., Bureau of Public Roads, Simplified Design Method for Reinforced Concrete Pipe Under Earth Fills, in Proceedings of Highway Research Board, volume 35, 1956.

American Association of State Highway Officials, Standard Specifications for Highway Materials: M6–51, M45–42, M85–60, M134–60, M170–60, and M198–62I.

American Society for Testing and Materials, Specification C33–59 and C76–59T.

Federal Specifications:
H–H–P–117, H–H–P–119, and SS–S–168.

278

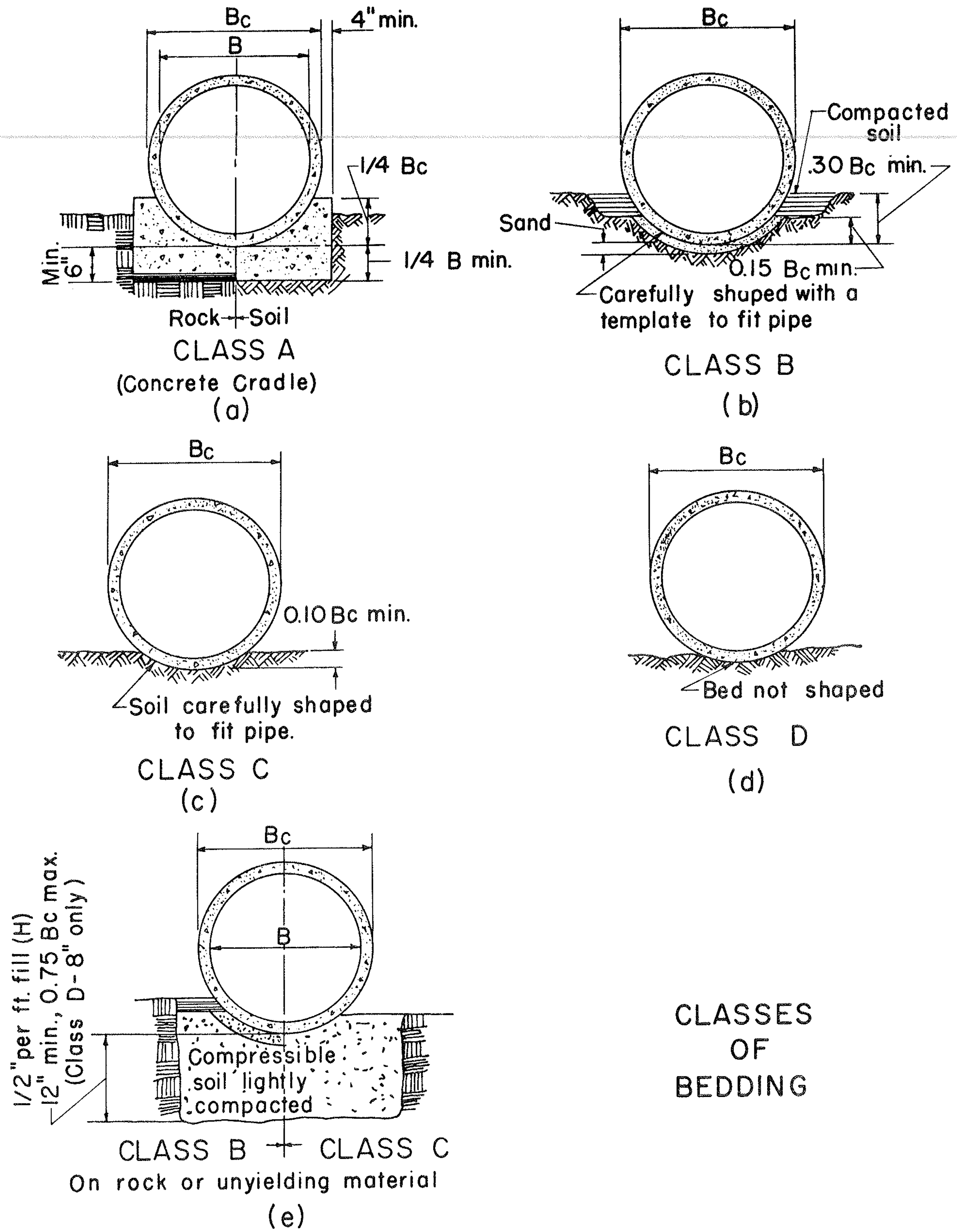
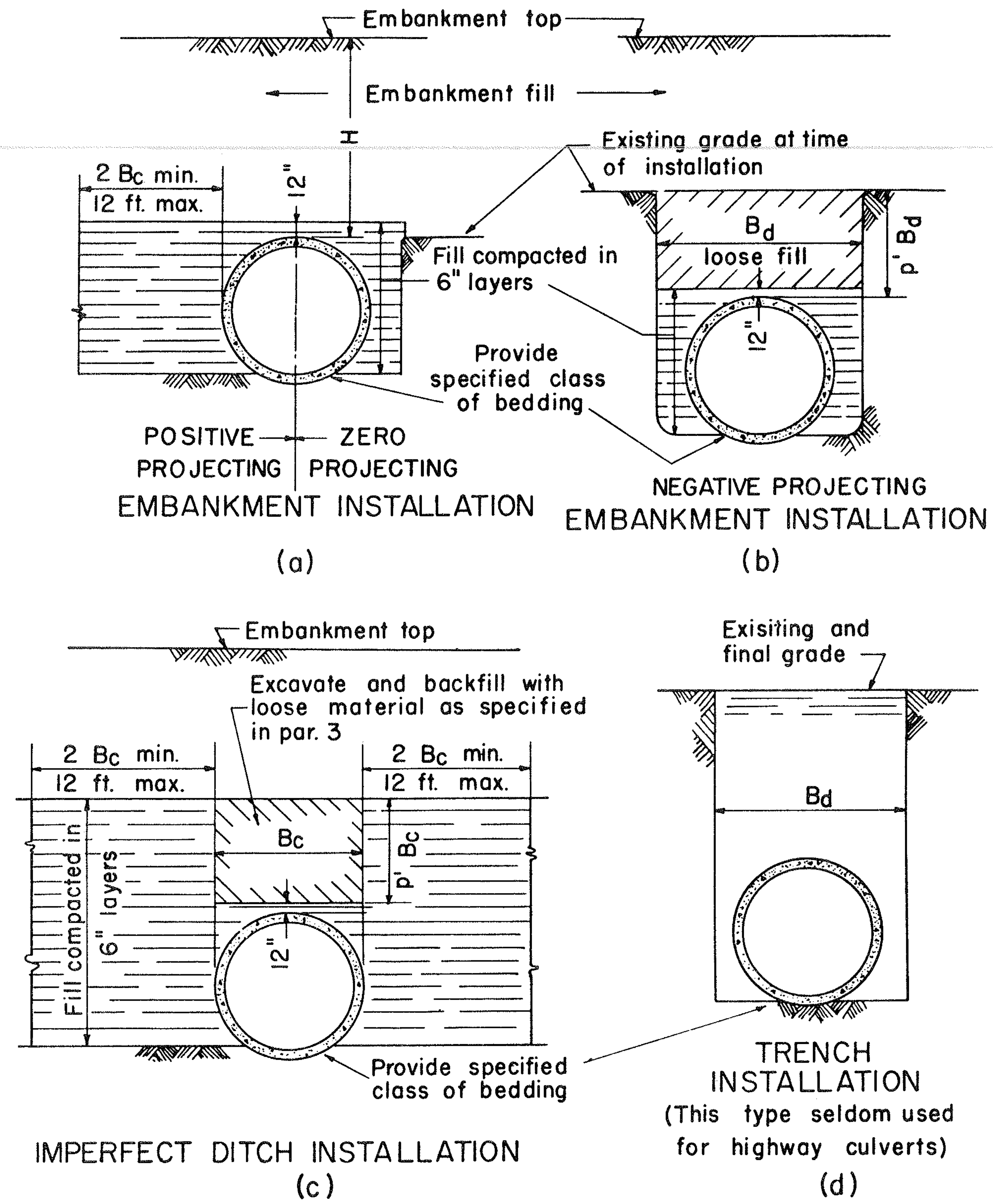

Bc
B
4" min.
1/4 Bc
1/4 B min.
Min. 6"
Rock Soil
CLASS A
(Concrete Cradle)
(a)
Bc
Compacted soil
.30 Bc min.
Sand
0.15 Bc min.
Carefully shaped with a template to fit pipe
CLASS B
(b)
Bc
0.10 Bc min.
Soil carefully shaped to fit pipe.
CLASS C
(c)
Bc
Bed not shaped
CLASS D
(d)
Bc
B
1/2" per ft. fill (H)
12" min., 0.75 Bc max.
(Class D-8" only)
Compressible soil lightly compacted
CLASS B CLASS C
On rock or unyielding material
(e)
CLASSES OF BEDDING

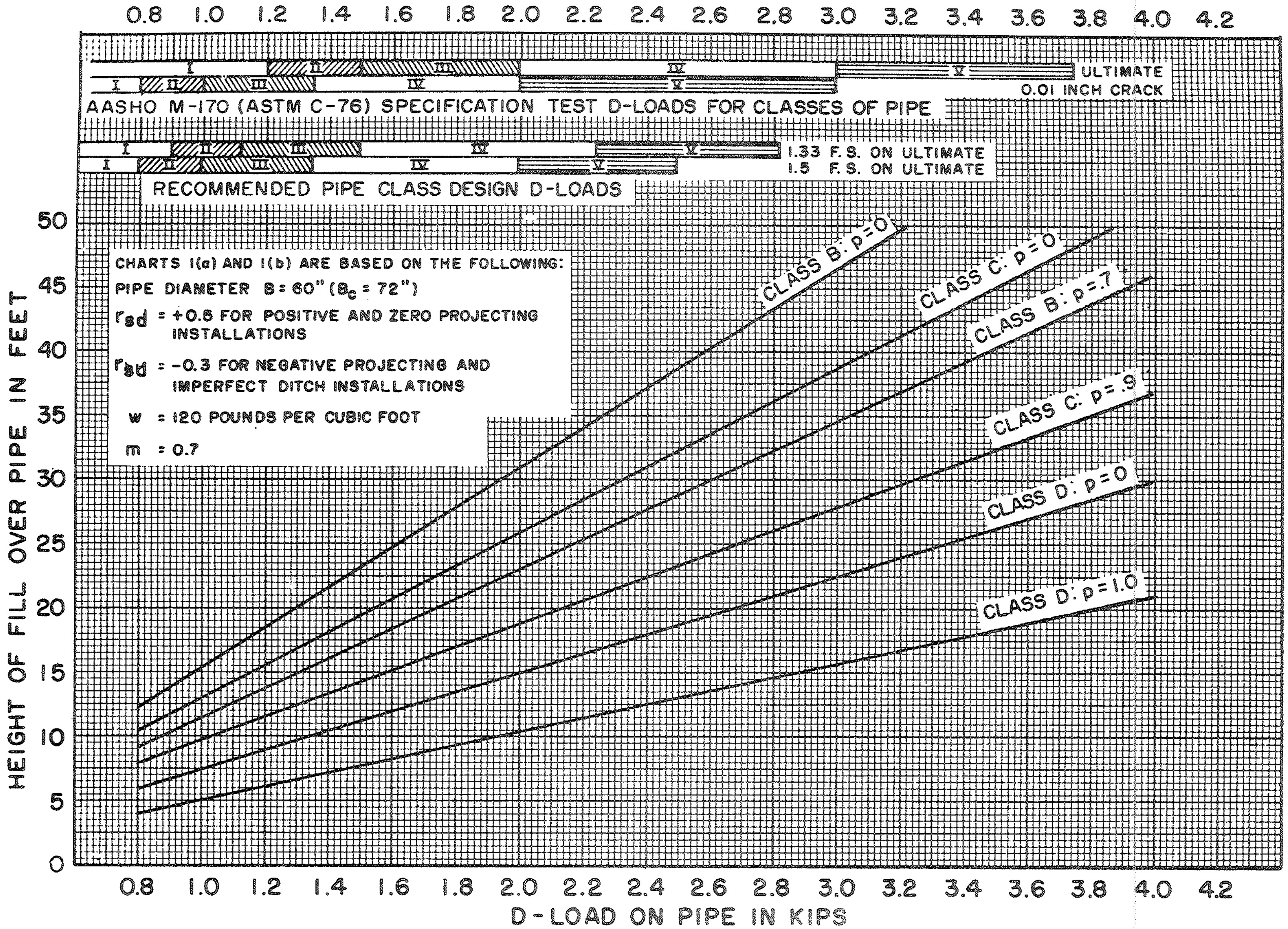
FIGURE 5



TYPES OF INSTALLATION

FIGURE 6

279



POSITIVE AND ZERO PROJECTING INSTALLATIONS
CHART I(a)

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

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 de referencias a publicaciones adicionales que fueron mencionadas en los textos seleccionados o que se asocian intimamente con el material que se presentó en la Vista General y los Textos Seleccionados. Cada referencia tiene cinco partes que se explican e ilustran abajo

(a) Número de referencia: Este número dá la posición de la referencia dentro de este bibliografía en particular. Se utiliza en el índice del compendio pero *no* deberá utilizarse al pedir publicaciones.

(b) Título: Este es el título de la publicación completa o el título de un artículo o sección dentro de una revista, informe, o libro.

(c) Datos bibliográficos: Este parágrafo dá los nombres de autores personales o organizacionales (si hay alguno), el nombre del editor y su direccion, la fecha de publicacion, y el numero de paginas representadas por el título en la parte (b). En algunas referencias el parágrafo termina con

Bibliographie

La bibliographie qui suit contient deux catégories de références. La première catégorie consiste en une référence pour chaque texte choisi qui est inclus dans la partie précédente de ce recueil. La deuxième catégorie contient des références pour des documents qui ont soit été cités dans les textes choisis, ou soit sont étroitement associés avec des écrits qui sont présentés dans l'Exposé ou les Textes Choisis. Chaque référence est composée de cinq parties qui sont expliquées et illustrées ci-dessous

(a) Numéro de la référence: Ce numéro indique la position de cette référence dans cette bibliographie. Ce numéro est indiqué dans l'index du recueil mais *ne doit pas* être utilisé pour les commandes de publications.

(b) Titre: Cela indique ou le titre du livre entier, ou le titre d'un article ou d'une section d'une revue, un rapport, ou un livre.

(c) Données bibliographiques: Ce paragraphe indique les noms des auteurs personnels (quand il y en a) ou des auteurs collectifs (organisation), le nom de l'éditeur et son adresse, la date de l'édition, et le nombre de pages qui sont incluses sous le titre dans (b). Certaines with an order number for the publication in parentheses.

(d) Availability information: This paragraph tells how the referenced publication is available to the reader. If the publication is out-of-print but may be consulted at a particular library, the name of the library is given. If the publication can be ordered, name and address of the organization from which it is available are given. ***The order should include all information given in parts (b) and (c) above.***

(e) Abstract: This paragraph contains an abstract of the publication whose title was given in part (b).

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un número de pedido para la publicación en paréntesis.

(d) Disponibilidad de la información: Este parágrafo explica que la publicación referenciada está disponible al lector en una de dos formas como sigue. (1) La publicación esta agotada pero puede ser consultada en la biblioteca indicada donde se sabe que se posee una copia. (2) La publicación puede ser pedida de la organización cuyo nombre y dirección están indicados. ***El pedido deberá incluir toda la información dada en las partes (b) y (c).***

(e) Resumen. Este parágrafo es un resúmen de la publicación cuyo título se dió en la parte (b).

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références se terminent par un numéro entre parenthèses qui indique le numéro de commande.

(d) Disponibilité des Documents: Ce paragraphe indique les deux façons dont le lecteur peut acquérir les documents: (1) L'édition est épuisée, mais une certaine bibliothèque détient ce document et il peut être consulté. (2) Le document peut être commandé à l'organisation dont le nom et l'adresse sont indiqués ici. ***L'ordre de commande doit inclure toutes les informations données dans les parties (b) et (c).***

(e) Analyse: Ce paragraphe est une analyse du texte dont le titre est cité dans la partie (b).

Illustration Ilustración Illustration

(a) Reference number
(a) Número de referencia
(a) Numéro de la référence

Reference 1

(b) Title
(b) Título
(b) Titre

LOW COST ROADS; DESIGN, CONSTRUCTION AND MAINTENANCE

(c) Bibliographic data
(c) Datos bibliográficos
(c) Données bibliographiques

Odier, L.; Millard, R.S.; dos Santos, Pimentel; Mehra, S. R. London: Butterworths; 1971. 158 p. (Sponsored by UNESCO).

(d) Availability information
(d) Disponibilidad de la información
(d) Disponibilité des documents

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

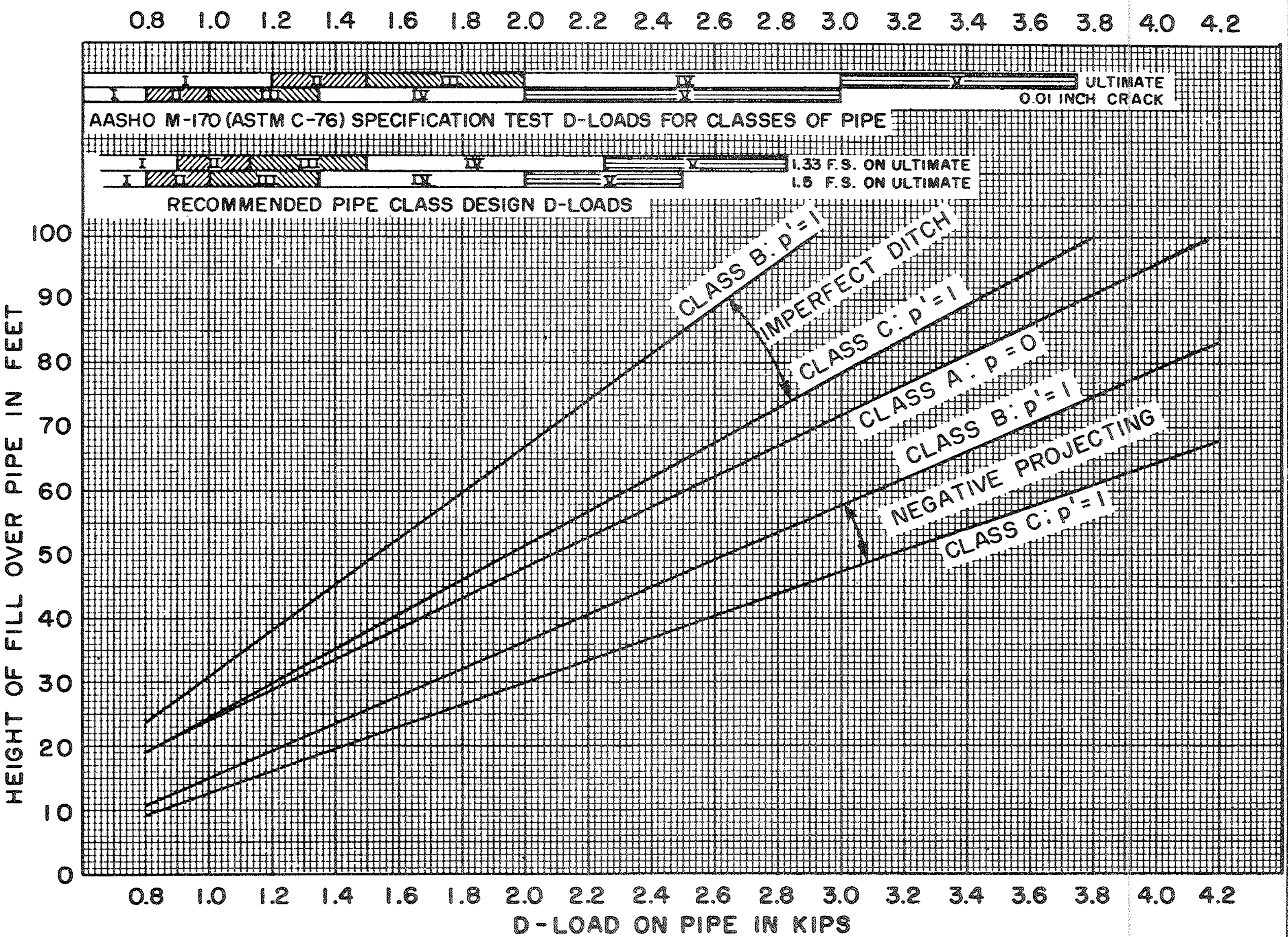
(e) Abstract
(e) Resumen
(e) Analyse

Engineering codes relating to road planning, design, construction, materials, and maintenance for use in the developing countries are presented here. Basic principles of road construction and maintenance policy including social and economic aspects, master plan and feasibility studies, as well as stage construction, are covered. Traffic and design speeds, design related to vertical alignment and horizontal alignment, and cross- section elements are discussed; design principles for unimproved roads, improved roads, roads with permanent surfaces, and flexible pavements are set forth. The drainage of the road is considered including the control of erosion, and the stability of embankments and cuttings. Defensive measures during wet weather construction are noted. The location and waterway requirements for bridges and culverts are discussed, and the principal factors to be considered in the design of bridge foundations and structures are indicated. Notes are provided on construction operations and plant which include preliminary and detailed surveys, setting out, earthworks, compaction, quarrying, soil stabilization, bituminous surfacing and concreting. The discussion of road maintenance distinguishes between short-term, largely manual maintenance and long-term maintenance usually involving the use of mechanical equipment. Methods of estimating costs are outlined and special consideration is given to the choice between manual and mechanized methods or combinations of the two.

The order should include all information given in parts (b) and (c) above.

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).



CLASS A, IMPERFECT DITCH AND NEGATIVE PROJECTING INSTALLATIONS

CHART I(b)

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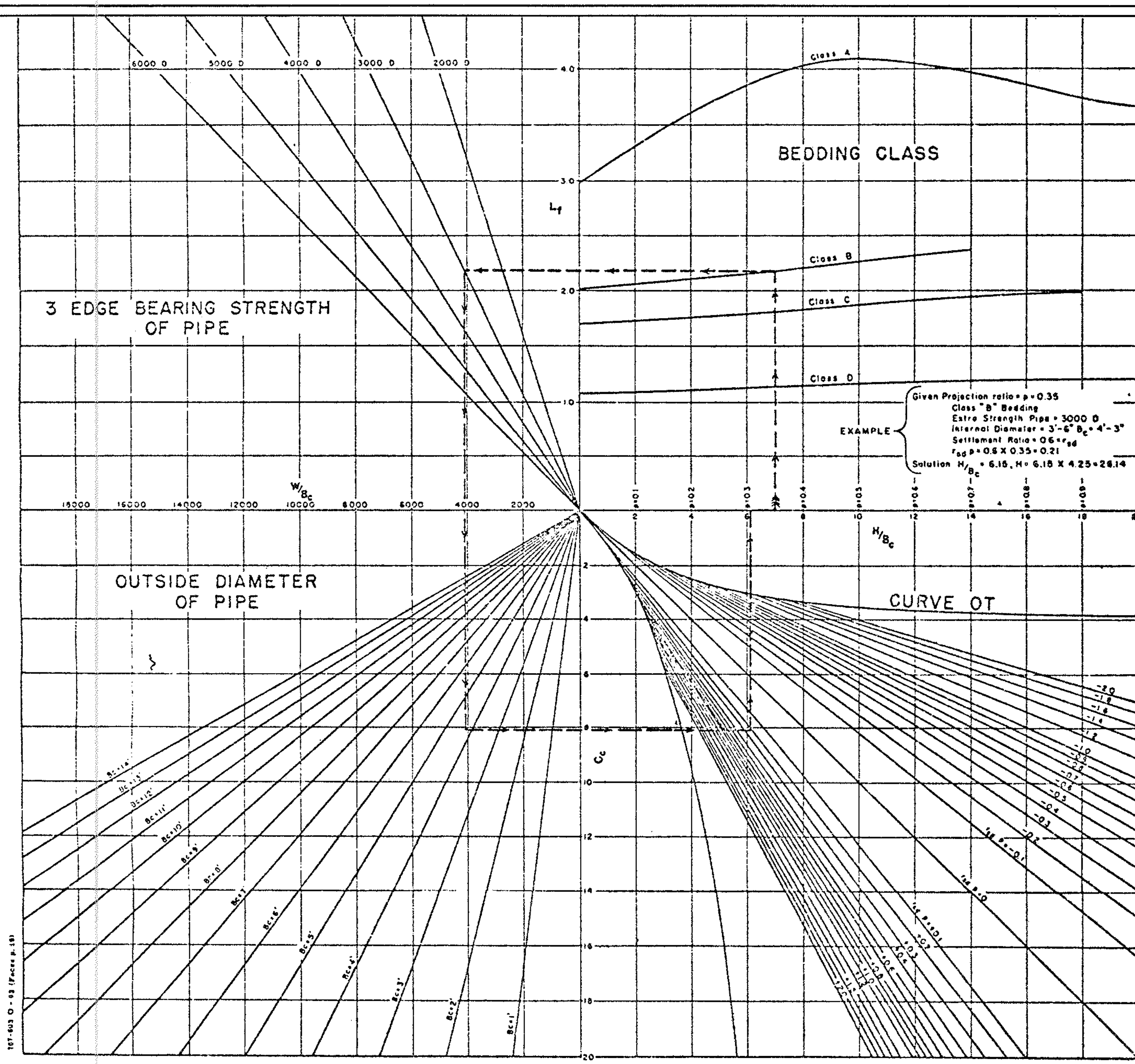

Chart II: Concrete Pipe Chart—Jan. 1937. Giving the maximum allowable height of fill (H) in terms of class of bedding (A, B, C, & D, Amer. Concr. Pipe Assn.) projection ratio (p), settlement ratio (r_{sd}), outside diameter of pipe (B_c) and strength of pipe (D). All dimensioned quantities are in feet—safety factor = 1.33

SELECTED TEXT REFERENCES

Reference 1
LOW COST ROADS; DESIGN, CONSTRUCTION AND MAINTENANCE

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Reference 2
HIGHWAY DRAINAGE GUIDELINES; VOLUME IV: GUIDELINES FOR THE HYDRAULIC DESIGN OF CULVERTS

American Association of State Highway and Transportation Officials, Operating Subcommittee on Design, Task Force on Hydrology and Hydraulics. Washington, DC: American Association of State Highway and Transportation Officials; 1975. 45 p.

Order from: American Association of State Highway and Transportation Officials, 444 North Capitol Street, N.W., Suite 225, Washington, DC 20001.

These guidelines, which make reference to appropriate publications, cover all aspects of the location, design, construction and use of culverts. The section on surveys includes, among others, topographic features, drainage area, channel characteristics, high water information, and existing structures. Plan and profile of culvert location, as well as shape/cross section, materials and end treatments of culvert type are described. Hydraulic design is covered in detail, and special hydraulic considerations are reviewed. Multiple use culverts such as utilities, stock and wildlife passage, land access, and fish passage are described. Culverts for irrigation water and culvert construction in irrigation canals are briefly considered. Debris control, service life, and safety are other aspects covered. The compilation of data and the detention of records are described and hydraulic related construction considerations are reviewed. Hydraulic related maintenance considerations are also discussed.

Reference 3
DRAINAGE STUDIES FROM AERIAL SURVEYS

Sternberg, Irwin. Photogrammetric Engineering, Vol.27, No. 4, 1961 September; pp. 638-44.

Order from: The American Society of Photogrammetry, 105 North Virginia Avenue, Falls Church, Virginia 22046.

Vertical aerial photographs examined stereoscopically provide a useful three-dimensional medium whereby drainage areas can be successfully determined with sufficient accuracy for the design of culverts for highway drainage. Discussed in the paper is the use of large-scale photographs for determining the placement of these culverts and other items concerned with the collection and dispersal of surface water during run-off periods. Methods, corrections to be applied, and techniques which have been successfully employed, all of which are within the capabilities of the average field engineer with limited photogrammetric training and equipment, are described. Examples are given to show the degree of accuracy which can be expected.

Reference 4
HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS

Herr, Lester A.; Bossy, Herbert G. Washington, DC: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, Hydraulics Branch; 1965 December. 54 p. (Hydraulic Engineering Circular No. 5; stock number 050-002-00010-1).

Order from: Superintendent of Documents, U.S. Government Printing Office, Washington, DC 20402.

The hydraulics of conventional culverts (circular, arch and oval pipes, both metal and concrete, and concrete box culverts, all with a uniform barrel cross section) and charts for selecting a culvert size for a given set of conditions are discussed, and instructions for using the charts are provided. Hydraulics are discussed with reference to both inlet control and outlet control types of flow, and the details are provided for computing the depth of tailwater, the velocity of culvert flow and the performance curves. Inlet control and outlet control nomographs are included, and entrance loss coefficients and Manning's n for natural stream channels are tabulated. Illustrative examples are provided.

Reference 5
DEBRIS-CONTROL STRUCTURES

Reihsen, G.; Harrison, L.J. Washington, DC: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, Hydraulics Branch; 1971 March. 38 p. (Hydraulic Engineering Circular No. 9).

Order from: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, HNG-31, Washington, DC 20590.

This circular describes a system of classifying the type of debris expected from any drainage basin, and lists the various types of debris control structures. The basis for choosing the type of control structure is given and details of design are discussed. It is noted that the preliminary field survey data should include information on the classification of the type of expected debris, the quantity of expected debris, future changes in debris type or quantity due to potential changes in land use, information from which the designer can estimate streamflow velocities in the vicinity of the culvert, topographic map or cross sections of the area, available for storage of debris at the site, accessibility of the storage area for debris removal and probable frequency of clean-out, and information on the possible damage that would result from debris clogging the drainage structure. The control structures discussed here include debris deflectors, debris racks, debris risers, debris cribs, debris fins, debris dams and basins, floating drift boom, and combination devices. Comments are also made on the standard and frequency of maintenance.

Reference 6
PRACTICAL GUIDANCE FOR DESIGN OF LINED CHANNEL EXPANSIONS AT CULVERT OUTLETS; HYDRAULIC MODEL INVESTIGATION

Fletcher, Bobby P.; Grace, John L., Jr. Vicksburg,Mississippi: U.S. Army, Waterways Experiment Station, Hydraulics Laboratory; 1974 October. 26 p. plus tables, photographs and appendices. (Report # AD/A-000612).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

The results are reported of specific research to develop practical guidance for the design of channel expansions lined with either sack revetment, cellular blocks, or rock riprap to prevent localized scour at culvert outlets. The research results provide guidance in the use of either of the three lining materials in lieu of rigid concrete channel expansions to provide effective and more economical plans of protection at culvert outlets. Potentially unstable channels that do not warrant the conventional type of rigid concrete structures due to the cost of such protection may be reconsidered in the light of the guidance and alternatives developed from this research. An appendix is provided which summarizes the results of related research efforts to develop practical guidance for estimating and controlling erosion downstream of culvert and storm-drain outlets. Empirical equations and charts are presented for estimating the extent of localized scour to be anticipated downstream of culvert and storm drain outlets and the size and extent of various natural and artificial type revetments and energy dissipators that may be used to control localized scour. With these results, designers can estimate the extent of scour to be expected, and select appropriate and alternative schemes of protection for controlling erosion downstream of culvert and storm-drain outlets.

Reference 7
CORRUGATED METAL PIPE CULVERTS; STRUCTURAL DESIGN CRITERIA AND RECOMMENDED INSTALLATION PRACTICES

U.S. Bureau of Public Roads, Office of Engineering and Operations, Bridge Division; Townsend, Merrill. Washington, DC: U.S. Bureau of Public Roads; 1966 June. 26 p.

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

Criteria relating to the design of corrugated steel and corrugated aluminum pipe culverts of riveted, resistance spot-welded, helical, and bolted fabrication are provided. The design charts for a rapid determination of the maximum allowable fill height for given pipe diameters are also provided. The criteria presented here cover the deflection of pipe, the critical buckling of pipe wall, longitudinal seam strength, and handling and installation strength. Pipe arch design, the effect of line load on pipe, and the durability of corrugated metal pipe are also discussed. Installation aspects discussed here include assembly, bedding, pipe foundation, and side fill. Comments are also made on alignment, camber, multiple installations, cover over pipe during construction and inspection.

Reference 8
REINFORCED CONCRETE PIPE CULVERTS; CRITERIA FOR STRUCTURAL DESIGN AND INSTALLATION

U.S. Bureau of Public Roads, Office of Engineering and Operations, Bridge Division; Townsend, Merrill. Washington, DC: U.S. Bureau of Public Roads; 1963 August. 16 p. plus chart.

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

The criteria presented here cover the determination of loads on concrete pipe, the determination of pipe strength required for the various classes of beddings and types of installations, the classes of bedding and recommended installation practices. The class of reinforced pipe recommended here is in AASHO (American Association of State Highway Officials) Specification M 170-60 (ASTM 79-59T). The determination of loads on pipe and the computation of D loads by charts are described in detail. The selection of class of pipe as well as examples of design (by use of formulas and use of charts) are also detailed. Details of bedding, laying, and backfilling around and over the pipe and the laying of elliptical, multiple pipe and jointing pipe are described.

ADDITIONAL REFERENCES

Reference 9
HYDROLOGY FOR ENGINEERS

2nd ed. Linsley, Ray K., Jr.; Kohler, Max A.; Paulhus, Joseph L.H. New York, New York: McGraw-Hill Book Company; 1975. 482 p. (McGraw-Hill Series in Water Resources and Environmental Engineering).

Order from: McGraw-Hill Book Company, 1221 Avenue of the Americas, New York, New York 10020.

The basic processes of hydrology are stressed in the second edition which represents an extensive revision of the earlier text. The importance of the digital computer as a tool for hydrologic analysis is recognized, but older methods are also discussed. The concept of the hydrologic cycle is described. The factors which affect a region's hydrology are covered in detail and include weather (solar and earth radiation, temperature, humidity, wind), and precipitation (measurement, interpretation of data, variations in precipitation, snowpak and snowfall). Streamflow (water stage, discharge, interpretation of streamflow data) aspects are detailed, and features of subsurface water (occurrence, moisture in the Vadose zone, moisture in the phreatic zone, potential of a groundwater reservoir) are described. Streamflow hydrographs (characteristics, hydrograph synthesis) are discussed, and the relations between precipitation and runoff (runoff phenomena, estimating volume of storm runoff, estimating snowmelt runoff, seasonal and annual runoff relations) are explored. Details of streamflow routing are given and computer simulation of streamflow is examined. The techniques are described for defining probability from a given set of data and with special methods employed for determining the spillway-design flood for major dams. Special methods for probability analysis using synthetically generated data are also discussed in the chapter on stochastic hydrology. Sedimentation and the morphology of river basins are covered in detail.

Reference 10
PROCEEDINGS OF A SYMPOSIUM ON FLOOD HYDROLOGY HELD IN NAIROBI IN OCTOBER 1975

Great Britain Transport and Road Research Laboratory, Transport Systems Department, Environment Division. Crowthorne, U.K.; 1977. 463 p. (TRRL Supplementary Report 259).

Order from: Transport and Road Research Laboratory, Crowthorne, Berkshire RG 11 6AU, U.K.

This symposium was planned jointly by the Economic Commission for Africa (ECA), the East African Meteorological Department of the East African Community (EAC) and the UK Transport and Road Research Laboratory (TRRL). Its purpose was to present to engineers, hydrologists and meteorologists working within Tropical Africa the results of recent research into problems of flood estimation in both urban and rural areas. Papers were presented from both urban and rural areas. Papers were presented from both East and West Africa to cover experience and conditions throughout Tropical Africa and covered not only recent research findings, but also current design methods and the needs of the engineer. The symposium was divided into three sections: a) a rainfall section discussing the rainfall models required as inputs to the flood methods, b) a rural flood estimation section describing techniques for the design of bridges and culverts in rural highway schemes, c) an urban flood estimation section describing techniques for surface water sewer design in towns. These Proceedings include the papers presented at the symposium and a summary of the discussions which followed.

Reference 11
OPEN CHANNEL HYDRAULICS

Chow, Ven Te. New York, New York: McGraw-Hill Book Company; 1959. 680 p. (McGraw-Hill Civil Engineering Series).

Order from: McGraw-Hill Book Company, 1221 Avenue of the Americas, New York, New York 10020.

This book which gives a broad coverage of recent developments is organized into five parts: Basic principles, uniform flow, gradually varied flow, rapidly varied flow, and unsteady flow. In Part I the type of flow in open channels is classified according to the variation in the parameters of flow with respect to space and time. The state of flow is classified according to the range of the invariants of flow with respect to viscosity and gravity. Four coefficients for velocity and pressure distributions are introduced. The energy and momentum principles which constitute the basis of interpretation for most hydraulic phenomena are treated thoroughly. In Part II, several uniform flow formulas are introduced. The design for uniform flow covers nonerodible, erodible and grassed channels. In the third Part, several methods for the computation of flow profiles are discussed. A new method of direct integration is introduced which requires the use of a varied flow function table. The method of singular points for the analysis of flow profiles is also discussed. Part IV on rapidly varied flow discusses problems supported by experimental data. The use of the flow-net method and the method of characteristics is mentioned. In Part V on unsteady flow, the treatment is general but practical.

Reference 12
GRAFICOS HIDRAULICOS PARA EL DISENO DE ALCANTARILLAS (Hydraulic Charts for the Selection of Highway Culverts)

Herr, Lester A.; Bossy, Herbert G. Washington, DC: U.S. Department of Transportation, Office of the Assistant Secretary for Policy, Plans and International Affairs; 1974 June. 52 p. (Hydraulic Engineering Circular No. 5 in Spanish; report # TPI-43-74-01).

Order from: U.S. Agency for International Development, Office of Development Information and Utilization, Development Support Bureau, Washington,DC 20523.

This is a translation into Spanish of the popular and useful publication Hydraulic Charts for the Selection of Highway Culverts, published in 1965 but still considered pertinent today. A current bibliography has been added. Measurements have been given in the metric system. The report includes a brief discussion of the hydraulics of conventional culverts and charts for the selection of a culvert size for a given set of conditions. Instructions for using the charts are provided. No attempt is made to cover all phases of culvert design.

Reference 13
CAPACITY CHARTS FOR THE HYDRAULIC DESIGN OF HIGHWAY CULVERTS

Herr, Lester A.; Bossy, Herbert G. Washington, DC: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, Hydraulics Branch; 1978 March. 90 p. (Hydraulic Engineering Circular No. 10).

Order from: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, HNG-31, Washington, DC 20590.

This circular contains a series of hydraulic capacity charts which permit the direct selection of a culvert size for a particular site without making detailed computations. The procedures given in this circular supplement those given in Hydraulic Engineering Circular (HEC) No. 5 (Ref. 4) by providing a solution for most designs likely to be encountered. This circular discusses the requirements and limitations for the use of the capacity charts and the instructions tell when HEC No. 5 must be used. Two groups of charts are included here. The first group is for box culverts with headwalls at 90 degrees to the culvert axis and sufficiently long to retain the fill slopes clear of the waterway opening. These headwall charts may also be used for culverts with wing walls flared from 10 degrees to 20 degrees with the culvert axis. The headwater-discharge relation is based upon small chamfers at all exposed edges at the entrance. The second group of charts is for box culverts with wing walls flared from 30 degrees to 75 degrees with the culvert axis and chamfered edge at the top of the entrance. The culverts with wing walls flared 30 degrees or more require less headwater depth for a given size and discharge rate than do those with just a headwall or 15 degrees wing walls. All charts are based upon an entrance face at right angles to the barrel axis.

Reference 14
HYDRAULIC DESIGN OF IMPROVED INLETS FOR CULVERTS

Morris, Johnny L.; Harrison, Lawrence J.; Normann, J. M. Washington, DC: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, Hydraulics Branch; 1972 August (reprinted 1974 March). 172 p. (Hydraulic Engineering Circular No. 13).

Order from: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, HNG-31, Washington, DC 20590.

Conventional culvert hydraulics are reviewed, the types of improved inlets are discussed, terms are defined, and design procedures for box and pipe culverts with improved entrances are presented. Inlet geometry refinements described here include bevel-edged, side-tapered, and slope-tapered inlets. Performance curves are presented and discussed. Four inlet control charts for culverts with beveled edges are included, and box culvert multibarrel installations (bevel-edged inlets) are described. The selected configurations of the side-tapered inlet are shown. Side taper ratios may range from 6:1 to 4:1. The latter is recommended as it results in a shorter inlet. The Vertical Face and Mitered Face are variations of the slope tapered inlet which provide additional improvements in hydraulic performance by increasing the head on the control section. For each degree of pipe culvert inlet improvement, there are many possible variations using bevels, tapers, drops, and combinations of the three. The tapered inlets are classified as either side-tapered (flared) or slope-tapered. The side tapered inlet for pipe culverts is designed in a manner similar to that used for a side tapered box culvert inlet. The slope-tapered design for pipes utilizes a rectangular inlet with a transition section between the square and round throat sections. Details of the design procedure are described and design charts are provided.

Reference 15
HYDRAULIC DESIGN OF ENERGY DISSIPATORS FOR CULVERTS AND CHANNELS

Thompson, Philip L.; Corry, Murray L.; Watts, F.J.; et al. Washington, DC: U.S. Federal Highway Administration, Office of Engineering, Bridge Division, Hydraulics Branch ; 1975 December. Various paging. (Hydraulic Engineering Circular No. 14; stock number 050-002-00102-7).

Order from: Superintendent of Documents, U.S. Government Printing Office, Washington, DC 20402.

This aid to selecting and designing an energy dissipator which will meet the requirements indicated by an erosion hazard assessment, details the design concept, and the erosion hazards, and discusses such aspects as culvert outlet velocity, velocity modification, flow transitions and the erosion of culvert outlets. Forced hydraulic jump and increased resistance, and details of impact basins, drop structures, stilling wells and riprap are also discussed. Preliminary energy dissipator selection is made by comparing the input constraints or design criteria flow regime, debris problems, location, channel characteristics, allowable scour, etc., with the attributes of the various energy distributors. The attributes of individual dissipators include: Froude number (Fr) range for best performance; discharge velocity or other limitations; possible maintenance; operational or location problems; maximum size; and limiting features such as culvert slope or shape. Dissipator designs fall into 2 general groups: those with Fr less than 3 (most designs are in this group), and those with Fr greater than 3. The designs are treated as illustrated by the conceptual models, and are related to actual situations through example problems.

Reference 16
HANDBOOK OF CONCRETE CULVERT PIPE HYDRAULICS

Portland Cement Association. Skokie, Illinois; 1964. 267 p.

Order from: Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60076.

This handbook is intended to assist the culvert designer in understanding hydrology and hydraulics and in applying these principles to the design of circular culverts. The material is presented in a form that is practical and usable for both the practicing engineer and the student. The principles of operation are fully explained and easy-to-use design aids are provided. Individual chapters cover the various major phases of hydraulic design: location and alignment of culverts; hydrology; hydraulics of culverts; culvert operation; entrances and headwalls; endwalls and outlet structures; drop inlets and sag culverts. The advantages of concrete pipe culverts are discussed. The appendix provides further information on culvert capacity charts, inlet control nomographs, nomographs for outlet control, discharge factors and functions for circular sections, references for determining earth loads and supporting strengths, and the use of forms in culvert design.

Reference 17
HANDBOOK OF STEEL DRAINAGE & HIGHWAY CONSTRUCTION PRODUCTS

2nd ed. American Iron and Steel Institute, Committee of Galvanized Sheet Producers and Committee of Hot Rolled and Cold Rolled Sheet and Strip Producers, Highway Task Force. New York, New York: American Iron and Steel Institute; 1973. 348 p.

Order from: American Iron and Steel Institute, 1000

16th Street, N.W., Washington, DC 20036.

This book presents a comprehensive discussion of the applications of storm drainage and special drainage problems, and provides a new simplified approach to hydraulic design and the selection of materials to meet various service conditions. The applications covered here include storm drainage, subdrainage, special drainage problems, underpasses and service tunnels, and aerial conduits. A value analysis for the objective evaluation of corrugated steel products for specific uses is also discussed. Product details and fabrication are reviewed and include pipe, pipearches and arches, couplings and fittings, and end finish. Culvert location factors such as structural design, hydraulics, corrosion and abrasion are detailed. Installation instructions are provided. Design specifications and installation notes are provided for tunnel liner plates, sheeting (lightweight), retaining walls, guardrail and median barriers, bridge railing, signs and supports, luminaire supports, bridge plank, and bridge forms.

Reference 18
SAFETY TREATMENT OF ROADSIDE CULVERTS ON LOW VOLUME ROADS

Kohutek, T.L.; Ross, H.E. Jr. College Station, Texas: Texas A&M University, Texas Transportation Institute; 1978 March. 36 p. plus appendices. (Research report 225-1; study 2-8-77-225).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Current American Association of State Transportation and Highway Officials (AASHTO) criteria for safety-treating fixed roadside hazards on high-speed facilities suggest that all culverts within a certain distance of the edge of the traveled way be shielded by a roadside barrier. In a restricted highway funding environment, this is not necessarily a cost-effective solution for low-volume highways. Using a cost-effectiveness model currently recommended by AASHTO, warrants for safety-treating culverts have been developed for 36 in. diameter pipe, 4 ft x 6 ft (4 ft height x 6 ft width) single box, and 4 ft x 6 ft multi-box (double box) culverts located on low-volume, rural highways. Each culvert design was evaluated on fill section embankments with 2½:1 and 6:1 slopes and for end offsets of 12, 18, and 24 ft. The treatments considered for each culvert design and embankment slope were: 1) do nothing (i.e., leave the culvert unprotected); 2) extend the culvert end 30 ft from the edge of the traveled way; 3) provide guardrail protection; and 4) provide grate protection. Figures are given to identify cost-effective treatments for the range of variables (ADT, embankment slope, offset, culvert design and safety treatment, etc.) considered in this study. For traffic volumes less than 750 and offset distances greater than 12 ft, the most cost-effective alternative is to leave the culvert unprotected. At higher traffic volumes the most cost-effective safety treatments are extending the culvert end to 30 ft or grating. Guardrail was found to be cost-effective only for larger culvert sizes and higher traffic volumes. However, guardrail protection was not the most cost-effective alternative for these situations. All supporting data and a discussion of the cost-effectiveness model used in the study are included in this report. Examples are given to illustrate the use of the criteria developed and to show the techniques used to develop these criteria. Other examples are included to enable the user to develop warrants for situations other than those considered in this study.

Reference 19
DRAINAGE STRUCTURES; DESIGN AND PERFORMANCE, 1960

Highway Research Board. Washington, DC; 1961. 31 p. (Bulletin 286).

Order from: University Microfilms International, 300 North Zeeb Road, Ann Arbor, Michigan 48106.

These papers are included in this bulletin. The first paper-Laboratory Study of Spur Dikes for Highway Bridge Protection- demonstrates the effectiveness of the spur dike for reducing scour and discusses its location and shape. The study also established criteria for determining the length of dike required at a particular location. The second paper - Culvert Inlet failures: A Case History - discusses the installations and failures (bent-up ends) on three large structural plate culverts installed with the upstream ends square and projecting to the fill toe. In the discussion which follows the paper, brief accounts are presented of other failures due to buoyancy of culverts, and comments are made on the determinancy of uplift and ballast and on the economy of alternative safeguards. The paper on "New Developments for Erosion Control at Culvert Outlets" presents information on a very promising and inexpensive method of controlling erosion at culvert outlets. The method consists of excavating a hole downstream from the culvert outlet and lining it with a graded layer of protective material consisting of coarse sand, gravel and boulders up to a size that will resist erosion at the peak flow. The development of design criteria for an armorplated, pre-shaped stilling basin is described.

Reference 20
DURABILITY OF DRAINAGE PIPE

Transportation Research Board. Washington, DC: 1978. 37 p. (NCHRP Synthesis of Highway Practice 50).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418.

The proper analysis of soil and water at the drainage site and its watershed should form the basis for selection of materials and types of pipe that should have the required service life. The main corrosion medium affecting drainage facilities is water and chemicals dissolved in or transported by water. Field and laboratory tests have been used to predict deterioration rates for a given environment. Materials used for drainage pipe include steel, aluminum, concrete, vitrified clay, stainless-steel, cast iron and plastic. Pipe protection measures include extra material thickness, coatings of various types, linings, and cathodic protection. Detection of corrosion-abrasion deterioration in culverts requires periodic inspection: at intervals of 10 or more years in less aggressive environments, and every 3 years in more aggressive environments. Trained inspectors should determine the nature of the electrolyte, flow rate, bedload, soil and water resistivity and ph, location, extent and type of corrosion, thickness loss, preventive measures used and reason for deficiencies. A rating system may be helpful and adequate records of all inspections should be kept. Several pipe protective measures are reviewed here. The broad assortment of repair techniques should be analyzed from standpoints of practicality, compatibility with existing installation, prospective performance and economics. Principles which must be considered in the location, design, construction and maintenance of culverts are discussed. The selection of an anticorrosion system would include: hydrologic and hydraulic considerations, structural considerations, availability and suitability of pipe types and sizes for the site, and the durability of the commonly used drainage materials that are satisfactory for the first 3 steps.

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 may 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 or name ap-

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 el Vista General, Textos Seleccionados, o Bibliografía. Los vocablos del tema que se listean son aquellos básicos 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 están listeados como apellido seguido por las iniciales. Las organizaciones nombradas son las que han producido información sobre la materia del compendio y que siguen siendo una fuente de información sobre alguna parte o el alcance total del compendio. Por esta razón se dan las direcciones postales para cada organización listeada.

Los números que siguen a un vocablo del tema, nombre personal, o nombre de organización son 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 cités 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 de personnes identifiées avec les sujets de ce recueil. Le nom de famille est suivi des initiales des prénoms. Les organisations citées sont celles qui ont écrit sur le sujet de ce recueil et qui continueront d'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 écrits en chiffres romains se rapporpears. Roman numerals refer to pages in the Overview, Arabic numerals refer to pages in the Selected Texts, and reference numbers (e.g., Ref.12) 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 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. 12) 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 compendio 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 indicadas bajo los términos que siguen a las palabras *see also.*

La explicación anterior esta subsiguientemente ilustrada.

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tent 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. 12) 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, les références qui les touchent se trouveront citées après les mots-clés qui suivent la notation *see also.*

Ces explications sont illustrées ci-dessous.