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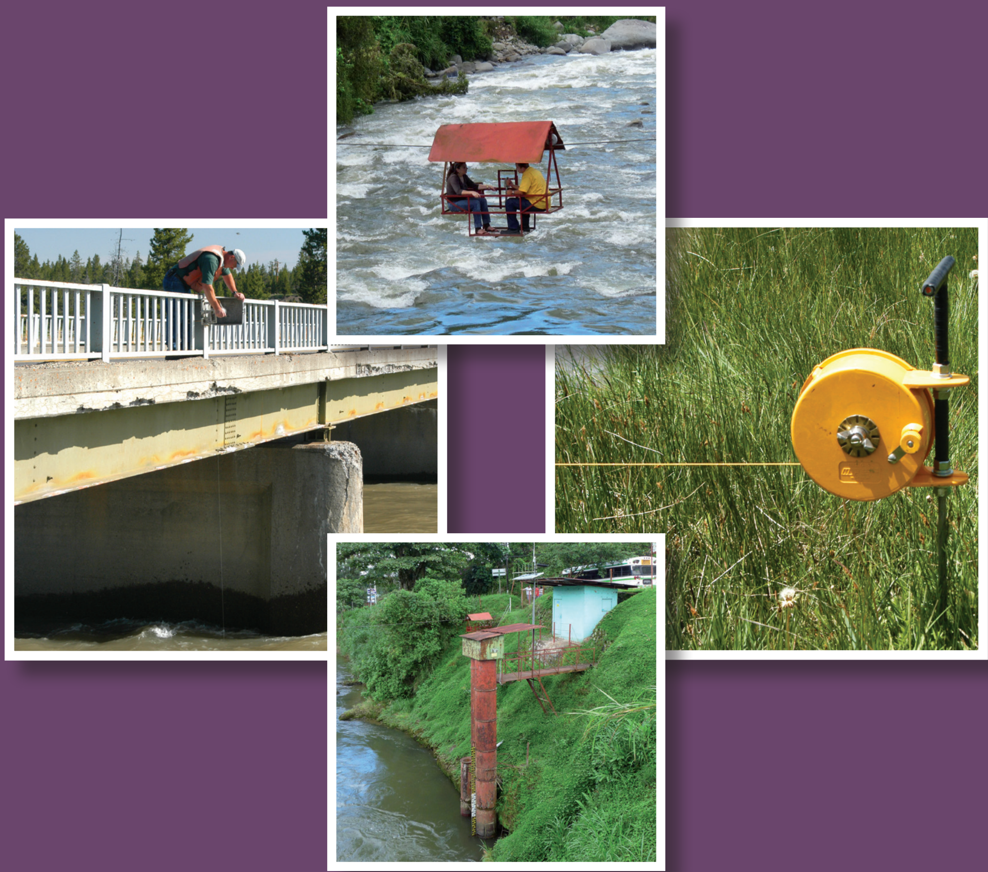

MANUAL ON STREAM GAUGING

VOLUME II – COMPUTATION OF DISCHARGE

World
Meteorological
Organization
Weather • Climate • Water

WMO-No. 1044

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FOREWORD

Increasing pressures on our vital water resources signify that confidence in the quality of streamflow records is today, more than ever, an essential prerequisite for the sustainable management of these critical resources.

The *Manual on Stream Gauging* (WMO-No. 519) was first released in 1980. Since then, however, there have been significant advances both in the approach and the methodologies employed.

Consequently, at its twelfth session (Geneva, October 2004), the WMO Commission for Hydrology (CHy) decided to meet the identified needs of the National Hydrological Services by revising the Manual to include the newer technologies that have been introduced over the period and are currently employed in this crucial field.

In this context, Volume I of the *Manual on Stream Gauging* encompasses the topics of gauge height measurement, stream velocity and stream discharge, whilst Volume II focuses on the discharge rating relationship.

On behalf of WMO, I wish to commend both volumes of this Manual and to express my appreciation to all who contributed to this key update.

(M. Jarraud)
Secretary-General

PREFACE

With the adoption by the WMO Commission for Hydrology of the Quality Management Framework – Hydrology (QMF-H) at its thirteenth session, the Commission demonstrated the importance that National Hydrological Services place on the efficiency, quality and effectiveness with which they perform their functions. The stream gauging programme is one of the fundamental building blocks of the operations of any hydrological service and therefore it is only natural that a thorough revision and update of the WMO *Manual on Stream Gauging* would be among the initial publications to be labeled as a component of QMF-H. For this reason, it is particularly gratifying for me to be able to introduce this important contribution from the Commission to the international hydrological community.

The preparation of the Manual was led by Mr Paul Pilon (Canada), then member of the advisory working group of the Commission for Hydrology. Mr Vernon B. Sauer made the revisions to the Manual. The reviewers of the draft text were Michael Nolan, Larry Bohman and Scott Morlock (United States of America); Stewart Child, James Waters and Reginald Herschy (United Kingdom of Great Britain and Northern Ireland); Pavel Polcar (Czech Republic); Kimmo Ristolainen (Finland); Svein Harsten (Norway); and Julio Llinas (Dominican Republic). The draft was edited by James Biesecker (United States).

The activities were carried out in association with the Open Panel of CHy Experts (OPACHE) on Basic Systems (Hydrometry and Hydraulics).

I express my gratitude to the original authors of the *Manual on Stream Gauging* (Mr R.W. Herschy and Mr S.E. Rantz), the author of this revision (Mr Sauer) and the reviewers for their contributions to the preparation of the revised Manual.

The Commission for Hydrology is planning to organize courses in various regions to train hydrological personnel in its use. The translation of the Manual into other languages will be considered soon by the Commission.

(Bruce Stewart)
President, Commission for Hydrology

SUMMARY

Thirty years after the publication of its first *Manual on Stream Gauging*, WMO is publishing this updated edition, which encompasses the new technologies that have emerged since 1980. The Manual is again being published in two separate volumes (Volume I: Fieldwork and Volume II: Computation of Discharge) to retain the concept of a "working manual". Volume I, which is aimed primarily at the hydrological technician, contains ten chapters. Three major topics are discussed in Volume I: the selection of gauging station sites, measurement of stage and measurement of discharge. Chapter 1 provides an introduction and a brief discussion of streamflow records and general stream-gauging procedures, while presenting some preliminary definitions of the terminology used in the Manual. Chapter 2 – Selection of Gauging station Sites, discusses the general aspects of gauging station network design, taking into account the main purpose for which a network is being set up (for example, flood or low-flow frequency studies), and the hydraulic considerations that enter into specific site selection. The section on the design of gauging station networks is, of necessity, written for the experienced hydrologist who plans such networks. Chapter 3 – Gauging station Controls, reviews the types of control, the attributes of a satisfactory control and artificial controls, as well as the criteria for selection and design of artificial controls. Chapter 4 – Measurement of Stage, discusses the basic requirements for collecting stage data, gauge structures and instrumentations, typical gauging station instrumentation configurations, data retrieval and conversion, new stage station design and operation of stage measurement stations, as well as safety considerations in operational stream gauging. Chapter 5 – Measurement of Discharge by Conventional Current Meter Methods, offers a general description of conventional current meter measurement of discharge, instruments and equipment, measurement of velocity and depth, and the procedure for conventional current meter measurement of discharge. Chapter 6 – Measurement of Discharge by Acoustic and Electromagnetic Methods, reviews three methods of gauging introduced in the first edition and now commonly used in stream gauging, namely the moving boat method using Acoustic Doppler Current Profilers (ADCPs), the Acoustic Velocity Meter (AVM) method and the electromagnetic method. A fourth method, which involves the Acoustic Doppler Velocity Meter (ADVM), is also covered. Chapter 7 – Measurement of Discharge by Precalibrated Measuring Structures, discusses standard measuring structures. The methods considered in Chapter 8 – Measurement of Discharge by Miscellaneous Methods, include velocity index methods, float measurements, volumetric measurement, portable weir and Parshall flume measurements, measurement of unstable flow for roll waves or slug flow, tracer dilution methods, remote-sensing and aircraft measurements, and radar methods for measurements of discharge. Chapter 9 – Indirect Determination of Peak Discharge, provides a general discussion of the procedures used in collecting field data and in computing peak discharge by the various indirect methods after the passage of a flood. Chapter 10, Uncertainty of Discharge Measurements, addresses the uncertainty related to the various methods discussed previously.

Volume II: Computation of Discharge, deals mainly with computation of the stage-discharge relation and computation of daily mean discharge. It is aimed primarily at the junior engineer who has a background in basic hydraulics. Volume II consists of six chapters. Chapter 1 – Discharge Ratings Using Simple Stage-Discharge Relations, is concerned with ratings in which the discharge can be related to stage alone. It discusses stage-discharge controls, the governing hydraulic equations, complexities of stage-discharge relations, graphical plotting of rating curves, rating for artificial and natural section controls, channel control, extrapolation of rating curves, shifts in the discharge rating, effect of ice formation on discharge ratings and sand channel streams. Chapter 2 – Discharge Ratings Using Velocity Index Method, presents the basics of the velocity index method, stage-area rating development and velocity index rating development, and then discusses discharge computation using the ADVM velocity index as an example. Chapter 3 – Discharge Ratings Using Slope as a Parameter, deals with variable slope caused by variable backwater, changing discharge, and a combination of both, as well as shifts in discharge ratings where slope is a factor. It also presents an approach to computing discharge records for slope stations. Chapter 4 – Flow Computation Models for Upland, Branched, and Tidal Streams, starting from one-dimensional unsteady flow equations, covers model formulation and boundary conditions, model applications and other empirical methods. Chapter 5 – Discharge

Ratings for Miscellaneous Hydraulic Facilities, discusses dams with movable gates, navigation locks, pressure conduits, urban storm drains, and automated computation of flow through water control structures. Finally, Chapter 6 – Analysis and Computation of Discharge Records Using Electronic Methods, examines the different problems related to electronic analysis and computation of discharge records, such as entry of field data into the electronic processing system, verification and editing of unit values, verification and analysis of field measurements, entry of rating curves into the electronic processing system, rating tables and curve plots, discharge measurement shift adjustments, primary computations, hydrograph plots, computation of extremes, estimation of missing records, monthly and annual value computations, and station analysis documentation.

RÉSUMÉ

Trente ans après la publication de son premier manuel de jaugeage (*Manual on Stream Gauging*), l'OMM présente une nouvelle édition mise à jour qui prend en considération les nouvelles techniques apparues depuis 1980. Comme la précédente, cette édition se compose de deux volumes distincts (Volume I: *Fieldwork* (Travaux sur le terrain) et Volume II: *Computation of Discharge* (Calcul des débits)) afin de maintenir le principe d'un «manuel pratique». S'adressant essentiellement au technicien en hydrologie, le Volume I traite de trois sujets principaux, à savoir le choix de l'emplacement des stations de jaugeage, les mesures de niveau et les mesures de débit et comprend dix chapitres. Le chapitre 1 – Introduction – donne un bref aperçu des relevés des débits d'un cours d'eau ainsi que des méthodes de jaugeage d'application générale, tout en donnant des définitions préliminaires pour quelques-uns des termes employés dans le Manuel.

Le chapitre 2 – *Selection of Gauging station Sites* (Choix de l'emplacement des stations de jaugeage) – expose les aspects généraux de la conception des réseaux de stations de jaugeage en prenant en considération le but poursuivi par la création d'un réseau donné (étude des crues ou de la fréquence des débits d'étiage, par exemple) et les facteurs hydrauliques qui entrent en ligne de compte dans le choix d'un site particulier. La section consacrée à la conception des réseaux de stations de jaugeage s'adresse naturellement à l'hydrologue expérimenté qui élabore les plans de réseaux de ce type. Le chapitre 3 – *Gauging station Controls* (Contrôles aux stations de jaugeage) – traite des différents types de tronçons et de sections de contrôle, des caractéristiques auxquelles doivent satisfaire de tels tronçons ou sections, des ouvrages de contrôle ainsi que des critères de sélection et de conception de ces derniers. Le chapitre 4 – *Measurement of Stage* (Mesures de niveau) – aborde les exigences de base en ce qui concerne la collecte de données sur le niveau de l'eau, les installations et instruments de mesure, la configuration type des instruments équipant les stations de jaugeage, l'extraction et la conversion des données, la conception des nouvelles stations de mesure du débit et le fonctionnement de ces stations ainsi que la sécurité des opérations de jaugeage. Le chapitre 5 – *Measurement of Discharge by Conventional Current Meter Methods* (Mesure du débit par la méthode classique du moulinet) – donne une description générale de la mesure du débit effectuée au moyen d'un moulinet, présente les instruments et l'équipement utilisés à cet effet et porte aussi sur la mesure de la vitesse d'écoulement et de la profondeur de l'eau ainsi que sur la méthode classique de mesure du débit à l'aide d'un moulinet. Le chapitre 6 – *Measurement of Discharge by Acoustic and Electromagnetic Methods* (Mesure du débit par des moyens acoustiques et électromagnétiques) – passe en revue trois méthodes de jaugeage présentées dans la première édition et aujourd'hui couramment utilisées, à savoir la méthode du bateau mobile utilisant des profileurs de courant à effet Doppler acoustique (ADCP), la méthode du moulinet acoustique (AVM) et la méthode électromagnétique. Une quatrième méthode, qui fait intervenir le moulinet acoustique à effet Doppler (AVDM) est également présentée. Le chapitre 7 – *Measurement of Discharge by Precalibrated Measuring Structures* (Mesure du débit par dispositifs pré-étalonnés) – porte sur des installations de mesure standard. Le chapitre 8 – *Measurement of Discharge by Miscellaneous Methods* (Mesure du débit à l'aide de diverses méthodes) – porte sur les méthodes fondées sur l'indice de vitesse, les méthodes à flotteurs, les méthodes volumétriques, les mesures par déversoir portable et par canal Parshall, la mesure de l'écoulement en ondes de translations brusques (roll waves) dans le cas de régimes instables, le jaugeage par dilution de traceurs, les mesures par télédétection et par aéronef et, enfin, l'utilisation du radar. Le chapitre 9 – *Indirect Determination of Peak Discharge* (Détermination

indirecte des débits de pointe) – donne une description générale des procédures appliquées pour recueillir des données sur le terrain et calculer les débits de pointe par différentes méthodes indirectes après une crue. Le chapitre 10 – *Uncertainty of Discharge Measurements* (Incertitude des mesures de débit) – traite des incertitudes afférentes aux diverses méthodes évoquées précédemment.

Le Volume II – *Computation of Discharge* (Calcul des débits) – traite principalement du calcul de la relation hauteur-débit et du débit journalier moyen. Il est destiné principalement aux jeunes ingénieurs ayant reçu une formation de base en hydraulique et comprend six chapitres. Le chapitre 1 – *Discharge Ratings Using Simple Stage-Discharge Relations* (Étalonnage des débits sur la base d'une simple relation hauteur-débit) – analyse les cas où le débit ne dépend que de la hauteur. Il expose les questions suivantes: contrôles de la relation hauteur-débit, équations hydrauliques de base, complexité de la relation hauteur-débit, pointage des courbes de tarage, sections de contrôle naturelles et artificielles, tronçons de contrôle, extrapolation des courbes de tarage, détarage, effets de la formation de glace sur le tarage et cours d'eau à lit sablonneux. Le chapitre 2 – *Discharge Ratings Using Velocity Index Method* (Étalonnage des débits la méthode fondée sur l'indice de vitesse) – présente les principes de base de cette méthode, l'établissement des courbes hauteur – superficie et des courbes basées sur l'indice de vitesse, puis aborde le calcul du débit en fonction de l'indice de vitesse basé sur le moulinet acoustique à effet doppler, à titre d'exemple. Le chapitre 3 – *Discharge Ratings Using Slope as a Parameter* (Étalonnage des débits utilisant la pente comme paramètre) – porte sur les questions suivantes: pente variable due à des remous variables, à un débit changeant ou à une combinaison des deux facteurs, détarage lorsque la pente entre en ligne de compte et méthode de calcul des relevés de débit pour les stations avec une pente marquée. Le chapitre 4 – *Flow Computation Models for Upland, Branched, and Tidal Streams* (Modèles de calcul de l'écoulement pour les cours d'eau supérieurs, ramifiés et à marée) – porte sur la formulation des modèles et les conditions aux limites, les applications de modèles et d'autres méthodes empiriques.

Le chapitre 5 – *Discharge Ratings for Miscellaneous Hydraulic Facilities* (Étalonnage des débits pour diverses installations hydrauliques) – porte sur les barrages à vannes mobiles, les écluses de navigation, les conduites en charge, les canaux de drainage urbains et le calcul automatique de l'écoulement par des ouvrages hydrauliques. Enfin, le chapitre 6 – *Analysis and Computation of Discharge Records Using Electronic Methods* (Analyse et calcul des relevés de débit par des moyens électroniques) – aborde les différents problèmes liés à l'analyse et au calcul électroniques des relevés de débit tels que l'introduction des données de terrain dans le système de traitement électronique, la vérification et l'ajustement des valeurs unitaires, la vérification et l'analyse des mesures effectuées sur le terrain, l'introduction des courbes de tarage dans le système de traitement électronique, les barèmes et les courbes d'étalonnage, les corrections apportées à la dérive des mesures du débit, les calculs primaires, les hydrogrammes, le calcul des extrêmes, l'évaluation des relevés manquants, le calcul des valeurs mensuelles et annuelles ainsi que la documentation sur l'analyse des données recueillies aux stations.

РЕЗЮМЕ

Тридцать лет спустя после опубликования своего первого *Наставления по измерению расхода воды* ВМО публикует это переработанное издание, в котором нашли отражение новые технологии, появившиеся после 1980 г. *Наставление*, как и в предыдущем случае, будет издано двумя отдельными томами (том I: *Полевые работы*, и том II: *Вычисление расхода воды*), с тем чтобы сохранить концепцию «рабочего наставления». Том I, который предназначен главным образом для техников-гидрологов, состоит из десяти глав. В этом томе обсуждаются три основные темы: выбор мест расположения гидрометрических створов, измерение уровня воды и измерение расхода воды. Глава 1 содержит введение и краткое описание данных наблюдений за речным стоком и процедур измерения стока воды; она также включает некоторые предварительные определения терминов, используемых в *Наставлении*. В главе 2 — Выбор мест расположения гидрометрических створов, рассматриваются общие аспекты проектирования сети гидрометрических станций с учетом основной цели, для которой создается сеть (например, паводки или исследования повторяемости низкого стока), а также аспекты гидравлики, которые следует учитывать при выборе места расположения конкретного створа. Раздел, посвященный проектированию сетей гидрометрических станций, по сути, предназначен для опытных гидрологов,

которые занимаются планированием таких сетей. Глава 3 — Инспекция гидрометрических станций, содержит обзор видов контроля, характеристик удовлетворительного контроля и искусственных контрольных сечений, а также критерии выбора и проектирования искусственных контрольных сечений. В главе 4 — Измерение уровня воды, рассматриваются основные требования к сбору данных об уровне воды, водомерным сооружениям и приборам, типичной конфигурации приборов на водомерных постах, извлечению данных и их преобразованию, проектированию новых станций измерения уровней воды и функционированию станций измерения уровней воды, а также соображения безопасности при оперативном измерении расхода воды. Глава 5 — Измерение расхода воды методами, при которых используются гидрометрические вертушки, содержит общее описание обычного измерения расхода воды, приборов и оборудования, измерения скорости и глубины и процедуры измерения расхода воды с помощью обычных вертушек. В главе 6 — Измерения расхода воды с помощью акустического и электромагнитного методов, дается описание трех методов измерения, представленных в первом издании и широко используемых для измерений расхода воды, а именно: метода измерения с движущегося судна с использованием акустических профилометров Доплера для измерения течения (ADCPs), ультразвукового метода измерения скорости течения (AVM) и электромагнитного метода. Рассматривается и четвертый метод, который предполагает использование акустического доплеровского измерителя скорости (ADVM). Глава 7 — Измерения расхода воды с помощью гидрометрических сооружений, посвящена стандартным сооружениям. Методы, изложенные в главе 8 — Измерения расходов воды разными методами, включают в себя методы скорость-индекс; измерения с помощью поплавков; объемные измерения; измерения с помощью переносных водосливов или лотков Паршаля; измерения неустойчивого потока для боковых волн или пробковых потоков; метод разбавления с использованием трассеров; измерения с помощью дистанционного зондирования и с самолета, а также радиолокационные методы измерения расхода. Глава 9 — Косвенное определение максимального расхода воды, дает общее описание процедур, используемых при сборе данных полевых измерений и при вычислениях максимального расхода с помощью различных косвенных методов после прохождения паводка. В главе 10 — Неопределенность измерений расхода, рассматривается неопределенность, связанная с различными методами, представленными выше.

Том II — *Вычисление расхода воды*, рассматривает главным образом вычисление зависимости расхода от уровня и вычисление среднего суточного расхода. Он предназначен для младших техников, знакомых с основами гидравлики. Том II состоит из шести глав. В главе I — Кривые расхода воды на примере простой зависимости расходов от уровня, рассматриваются кривые, в которых расход ставится в зависимость только от уровня. В ней описываются контрольные сечения для измерения уровня и расхода; основные уравнения гидравлики; сложности, связанные с зависимостью расхода от уровня; графическое построение кривых расхода; кривые расхода для искусственных и естественных контрольных сечений; контрольное русло; экстраполяция кривых расхода; смещения кривых расхода; влияние образования льда на кривые расхода и деформацию русла. Глава 2 — Кривые расхода воды с использованием метода скорость-индекс, описывает основы метода скорость-индекс; построение кривой уровень-площадь и кривой скорость-индекс; а также рассматривает вычисление расхода воды с помощью индекса скорости ADVM в качестве примера. В главе 3 — Кривые расхода воды на примере использования уклона водной поверхности в качестве параметра, рассматривается переменный уклон, вызванный меняющимся подпором, изменением расхода и сочетанием того и другого, а также смещения кривых расхода, в которых уклон является коэффициентом. В ней также представлен подход к вычислению расхода на станциях, измеряющих уклон. Глава 4 — Модели расчета потока воды для нагорных, разветвленных и приливно-отливных течений, начиная с одномерных уравнений неустановившегося потока, посвящена формулированию модели и граничных условий, применениям моделей и другим эмпирическим методам. Глава 5 — Кривые расхода воды для различных гидротехнических сооружений, рассматривает дамбы с раздвижными шлюзами, навигационные шлюзы, напорные водоотводные каналы, городские водостоки, а также автоматическое вычисление потока через водорегулирующие сооружения. Наконец, в главе 6 — Анализ и вычисление расхода воды с использованием электронных методов, рассматриваются различные проблемы, связанные с электронным анализом и вычислением расходов, такие как введение данных полевых измерений в электронную систему обработки данных; верификация и редактирование единичных значений; верификация и редактирование данных полевых измерений; внесение кривых расходов в электронную систему обработки данных; таблицы и графики кривых расхода; поправки на смещение в измерениях расхода; первичные вычисления; графики, представляющие гидрографические данные; вычисления экстремальных значений; расчет недостающих данных; вычисления ежемесячных и годовых значений и документация с результатами анализа данных наблюдений на станции.

RESUMEN

Treinta años después de la primera publicación del *Manual on Stream Gauging* (Manual sobre aforo de caudales), la OMM presenta una edición actualizada del mismo para incluir las nuevas tecnologías que han ido apareciendo desde 1980. De nuevo, y para seguir con el concepto de "manual de trabajo", esta publicación se divide en dos volúmenes (Volumen I "Trabajos sobre el terreno" y Volumen II "Cálculo del caudal"). El Volumen I "Trabajos sobre el terreno" está destinado esencialmente a los técnicos en hidrología y consta de diez capítulos. En dicho Volumen se abordan tres cuestiones principales, a saber: la selección del emplazamiento de las estaciones de aforo, la medición del nivel y la medición del caudal. En el capítulo 1 "Introducción" se exponen brevemente los registros del flujo de la corriente y los métodos de aforo generalmente utilizados, ofreciéndose además algunas definiciones preliminares de la terminología utilizada en el Manual. El capítulo 2 "Selección del emplazamiento de las estaciones de aforo" trata de los aspectos generales del diseño de redes de estaciones de aforo, teniendo en cuenta el objetivo buscado con el establecimiento de dichas redes (por ejemplo, estudios sobre la frecuencia de crecidas o períodos de estiaje) y los factores hidráulicos que han de tenerse en cuenta en la selección de un sitio determinado. La sección dedicada al diseño de redes de estaciones de aforo está destinada, naturalmente, a los hidrólogos experimentados que elaboran los planes de este tipo de redes. El capítulo 3 "Controles en las estaciones de aforo" estudia los distintos tipos de control, las características que deben reunir los controles naturales satisfactorios y los artificiales, así como los criterios para seleccionar y diseñar los controles artificiales. El capítulo 4 "Medición del nivel" analiza los requisitos básicos para la recopilación de datos sobre el nivel de las aguas, las estructuras e instrumentos de medición, las configuraciones típicas de los instrumentos de las estaciones de aforo, la recuperación y conversión de datos, el diseño de las nuevas estaciones de nivel y el funcionamiento de las estaciones de medición del nivel, así como las cuestiones relacionadas con la seguridad durante las operaciones de aforo del caudal. El capítulo 5 "Medición del caudal por el método clásico del molinete" facilita una descripción general de las mediciones del caudal efectuadas con molinete, de los instrumentos y equipo necesarios, de los métodos para medir la velocidad y la profundidad, y de los procedimientos de medición del caudal con el método clásico del molinete. En el capítulo 6 "Medición del caudal por métodos acústicos y electromagnéticos" se exponen tres métodos de aforo del caudal introducidos en la primera edición y cuyo uso se ha generalizado, a saber: el método del bote móvil dotado de perfiladores de corriente de efecto Doppler (ADCP), el método de los medidores ultrasónicos (acústicos) de velocidad (AVM) y el método electromagnético. En este capítulo se presenta además un cuarto método basado en medidores acústicos de velocidad de efecto Doppler (ADVM). El capítulo 7 "Medición del caudal por medio de estructuras de medición precalibradas" se centra en las estructuras de medición normalizadas. El capítulo 8 "Otros métodos de medición del caudal" pasa revista a varios métodos como son los índices de velocidad, las mediciones con flotadores, los métodos volumétricos, los vertederos de aforo portátiles y los medidores Parshall, las mediciones de los flujos inestables de las ondas abruptas de traslación (*roll waves*), los métodos de dilución de trazadores, las mediciones realizadas por teledetección o desde aeronaves y los métodos para medir el caudal mediante radares. El capítulo 9 "Determinación indirecta de caudales máximos instantáneos" ofrece un estudio general de los procedimientos indirectos utilizados para la recopilación de datos sobre el terreno y para el cálculo de caudales máximos instantáneos después de una crecida. El capítulo 10 "Incertidumbre de las mediciones del caudal" analiza las incertidumbres relacionadas con los diversos métodos anteriormente mencionados.

El Volumen II "Cálculo del caudal" se ocupa principalmente del cálculo de la relación altura-caudal y del caudal medio diario. Está destinado principalmente a los ingenieros noveles que tienen conocimientos básicos en hidráulica y consta de seis capítulos. El capítulo 1 "Calibración del caudal mediante una simple relación altura-caudal" analiza los casos en que el caudal se relaciona únicamente con la altura. En este capítulo se abordan cuestiones como los controles de la relación altura-caudal, las ecuaciones fundamentales de la hidráulica, las complejidades de la relación altura-caudal, la trascripción gráfica de las curvas de gasto, el aforo en los controles de las secciones artificiales y naturales, el control del canal, la extrapolación de las curvas de gasto, las fluctuaciones del caudal, los efectos de la formación de hielo en el caudal y los ríos de lecho arenoso. En el capítulo 2 "Calibración del caudal mediante el método del índice de velocidad" se exponen los principios básicos del método del índice de velocidad y del establecimiento de curvas de gasto fundamentadas en la relación altura-superficie y en el índice de velocidad. Además,

se presenta el cálculo del caudal utilizando como ejemplo el índice de velocidad ADVM. El capítulo 3 “Calibración del caudal utilizando la pendiente como parámetro” trata de las cuestiones siguientes: pendiente variable debida a remansos variables, a un caudal variable o a una combinación de ambos, y fluctuación de las curvas de gasto cuando la pendiente constituye un factor. Asimismo, presenta un método de cálculo de los registros del caudal para la estaciones en pendiente. El capítulo 4 “Modelos de cálculo del caudal aguas arriba y en ríos ramificados y con mareas a partir de ecuaciones unidimensionales de flujos variables” estudia la formulación de modelos y condiciones de contorno, así como las aplicaciones de los modelos y otros métodos empíricos. El capítulo 5 “Calibración del caudal en diversas instalaciones hidráulicas” aborda las cuestiones relacionadas con las presas con compuertas móviles, las esclusas de navegación, las tuberías de carga, los canales de drenaje urbano y el cálculo automático de la corriente mediante estructuras de control del agua. Finalmente, el capítulo 6 “Análisis y cálculo de registro del caudal mediante métodos electrónicos” examina diferentes problemas relacionados con el análisis y el cálculo de los registros del caudal como son: la introducción de datos de campo en un sistema de procesamiento electrónico, la verificación y ajuste de valores unitarios, la verificación y análisis de mediciones sobre el terreno, la introducción de curvas de gasto en el sistema de procesamiento electrónico, la transcripción gráfica de curvas y tablas de gasto, los ajustes aportados a las desviaciones de las mediciones del caudal, los cálculos primarios, los hidrográmas, el cálculo de extremos, la estimación de los registros faltantes, los cálculos de valores mensuales y anuales, y los documentos de análisis de la estación.

INTRODUCTION

1.1 STREAMFLOW RECORDS

Streamflow serves man in many ways. It supplies water for domestic, commercial and industrial use; irrigation water for crops; dilution and transport of wastes; energy for hydroelectric power; transport channels for commerce; and a medium for recreation. Records of streamflow are the basic data used in developing reliable surface water supplies because the records provide information on the availability of streamflow and its variability in time and space. The records are therefore used in the planning and design of surface water related projects, and they are also used in the management or operation of such projects after the projects have been completed. Streamflow records are also used for calibrating hydrological models, which are used for forecasting, such as flood forecasting.

Streamflow, when it occurs in excess, can create a hazard – floods cause extensive damage and hardship. Records of flood events obtained at gauging stations serve as the basis for the design of bridges, culverts, dams and flood control reservoirs, and for flood plain delineation and flood warning systems. Likewise, extreme low flow and drought conditions occur in natural streams, and should be documented with reliable streamflow records to provide data for design of water supply systems. It is therefore essential to have valid records of all variations in streamflow.

The streamflow records referred to above are primarily continuous records of discharge at stream-gauging stations; a gauging station being a stream site instrumented and operated so that a continuous record of stage and discharge can be obtained. Networks of stream-gauging stations are designed to meet the various demands for streamflow information including an inventory of the total water resources of a geographic area. The networks of continuous record stations, however, are often augmented by auxiliary networks of partial record stations to fill a particular need for streamflow information at relatively low cost. For example, an auxiliary network of sites, instrumented and operated to provide only instantaneous peak discharge data, is often established to obtain basic information for use in regional flood frequency studies. An auxiliary network of un-instrumented sites for measuring low flow only is often established to provide basic data for use in regional studies of drought and of fish and wildlife management.

This Manual is a revision of the previously published *Manual on Stream Gauging* (WMO-No. 519), Volumes I and II, 1980. Much of the original material is used in this manual where procedures and equipment are still relevant. Likewise, material from a similar 2-volume manual by Rantz (1982) is also used. In many cases, the two manuals are identical.

1.2 GENERAL STREAM-GAUGING PROCEDURES

Once the general location of a gauging station has been determined from a consideration of the need for streamflow data, its precise location is selected to take advantage of the best locally available conditions for stage and discharge measurement and for developing a stable stage-discharge relation, also called a "discharge rating", or simply a "rating".

A continuous record of stage is obtained by installing instruments that sense and record the water surface elevation in the stream. Discharge measurements are initially made at various stages to define the relation between stage and discharge. Discharge measurements are then made at periodic intervals, usually monthly, to verify the stage-discharge relation or to define any change in the relation caused by changes in channel geometry and/or channel roughness.

Artificial controls such as low weirs or flumes are constructed at some stations to stabilize the stage-discharge relations in the low flow range. These control structures are calibrated by stage and discharge measurements in the field.

In recent years, it is increasingly common to have real-time, automatic, transfer of data from gauging stations to hydrological analysis centres. During certain events, such as imminent flood threats, real-time data are used as input to hydrological models to simulate water behaviour and provide flood forecasts for authorities. Real-time data are frequently published on internet sites for immediate use by the general public. Real-time data are used for several purposes and users should be made aware that real-time data are always considered preliminary and have not been quality controlled.

Data obtained at the gauging stations are reviewed and analyzed by engineering personnel throughout the water year. Discharge ratings are established, either by graphical methods or by computer methods. Unit values of recorded gauge heights are used to compute unit and daily values of gauge height and discharge. The mean discharge for each day and extremes of discharge for the year are computed. The data are then prepared for publication and are considered final.

1.3 DEFINITIONS

A few common terms as defined by Sauer (2002) that are used throughout this Manual (Volumes I and II) will be defined in this section. This is not intended to define all stream gauging terms. Additional definitions will be given as needed in other sections of the Manual.

Gauge height, stage, and elevation are interchangeable terms used to define the height of the surface of a water feature, such as a stream, reservoir, lake, or canal. For a stream gauging station, gauge height is the more appropriate terminology, but the more general term "stage" is sometimes used interchangeably. For lakes, reservoirs and tidal streams, the height of the water surface usually is referred to as elevation. Gauge height (also stage) is measured above an arbitrary gauge datum, whereas elevation is measured above an established vertical datum, such as mean sea level. Gauge heights and elevations are principal data elements in the collection, processing, and analysis of surface-water data and information. Gauge heights and elevations are measured in various ways, such as by direct observation of a gauging device, or by automatic sensing through the use of floats, transducers, gas-bubbler manometers and acoustic methods. Gauge heights and elevations should be measured and stored as instantaneous unit values. Subsequent data processing and analysis will provide the means for any required analysis, such as averaging.

Stream velocity is another data element in a stream gauging system. Unit values of stream velocity are measured at some sites for the purpose of computing stream discharge. This is done most commonly where variable backwater conditions are present. Unit values of stream velocity are measured at some sites where variable backwater is not present to improve the calculation of discharge. The three principal instruments for measuring stream velocity are the deflection vane gauge, the electromagnetic velocity meter and the acoustic (ultrasonic or Doppler) velocity meters.

Stream discharge is a very important element, and frequently the ultimate goal in stream gauging. Discharge cannot be measured directly, but must be computed from other measured variables such as gauge height, stream depth, stream width, and stream velocity. Daily mean values of discharge are usually computed from instantaneous unit values of discharge, using computer methods. This differs from some of the methods used in the past where daily mean values of discharge were computed from daily mean values of gauge height. It also differs from procedures where mean values of gauge height for subdivided parts of a day were used to compute discharge.

The term *unit value* is used to denote a measured or computed value of a variable parameter that is associated with a specified instantaneous time and date. In addition, unit values generally are part of a time-series data set. For surface-water records, unit values for all parameters always should be instantaneous values. Some parameters, such as velocity, tend to fluctuate rapidly and a true instantaneous value would be difficult to use in the analysis and processing of the records. Some instruments are designed to take frequent (for example, every second) readings, temporarily store these readings, and then compute and store a mean value for a short time period. For these situations, the field instruments are programmed to record mean unit values for very short time intervals (1 to 2 minutes) so they can be considered for practical purposes to be instantaneous unit values. Data recorded for very short time intervals are sometimes referred to as high time-resolution data.

Daily values are measured or computed values of a parameter for a specific date. The time of the daily value is not required, although for certain daily values, time sometimes is stated. Examples of daily values are daily mean value, maximum instantaneous value for a day, and minimum instantaneous value for a day. In the case of maximum and minimum instantaneous values for a day, the time of the value usually is stated.

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CHAPTER 1

DISCHARGE RATINGS USING SIMPLE STAGE-DISCHARGE RELATIONS

1.1 INTRODUCTION

Continuous records of discharge at gauging stations are computed by applying the discharge rating for the stream to records of stage. Discharge ratings may be simple or complex, depending on the number of variables needed to define the stage-discharge relation. This chapter is concerned with ratings in which the discharge can be related to stage alone. The terms rating, rating curve, station rating, and stage-discharge relation are synonymous and are used interchangeably in this Manual. Parts of this chapter are based on ISO standard 1100-2 (1998), with some sections taken directly from the ISO standard.

Discharge ratings for gauging stations are usually determined empirically by means of discharge measurements made in the field. Notable exceptions are the pre-calibrated ratings used in several countries for the special weirs and flumes discussed in Volume I, Chapter 7. However, even at sites where a weir or flume is used, it is advisable to make some current meter measurements for the purpose of confirming the pre-calibrated rating.

Common practice is to measure the discharge of the stream periodically, usually by current meter, and to note the concurrent stage. Measured discharge is then plotted against concurrent stage on graph paper to define the rating curve. At a new station many discharge measurements are needed to define the stage discharge relation throughout the entire range of stage. Periodic measurements are needed thereafter to either confirm the stability of the rating or to follow changes (shifts) in the rating. A minimum of ten discharge measurements per year is recommended, unless it has been demonstrated that the stage-discharge relation is completely unvarying with time. In that event the frequency of measurements may be reduced. It is of prime importance that the stage-discharge relation be defined for flood conditions, for extreme low flow conditions and for periods when the rating is subject to shifts as a result of ice formation or as a result of the variable channel and control conditions. It is essential that the stream gauging programme provides for the non-routine measurement of discharge at those times.

If the discharge measurements cover the entire range of stage experienced during a period of time when the stage-discharge relation is stable, there is little problem in defining the discharge rating for that period. On the other hand, if, there are no discharge measurements to define the upper end of the rating the defined lower part of the rating curve must be extrapolated to the highest stage experienced. Such extrapolations are always subject to error, but the error may be minimized if the analyst has knowledge of the principles that govern the shape of rating curves. Much of the material in this Chapter is directed toward a discussion of those principles so that when the hydrologist is faced with the problem of extending the high water end of a rating curve he or she can decide whether the extrapolation should be a straight line or whether it should be concave upward or concave downward.

The problem of extrapolation can be circumvented, of course, if the unmeasured peak discharge is determined by use of the indirect methods discussed in Volume I, Chapter 9. In the absence of such peak discharge determinations, some of the uncertainty in extrapolating the rating may be reduced by the use of one or more of several methods of estimating the discharge corresponding to high values of stage. Four such methods are discussed in a subsequent section of this chapter.

In some cases the lower end of the rating curve may also need extrapolation. Procedures for low-water extrapolation are discussed in subsequent sections of this Manual. A most important method involves use of the gauge height of zero flow which will be discussed in detail.

1.2 STAGE-DISCHARGE CONTROLS

The subject of stage-discharge controls was discussed in detail in Volume I, Chapter 3, but a brief summary at this point is appropriate.

The stage-discharge relation for open-channel flow at a gauging station is governed by channel conditions downstream from the gauge, referred to as a control. Two types of controls can exist, depending on channel and flow conditions. Low flows are usually controlled by a section control whereas high flows are usually controlled by a channel control. Medium flows may be controlled by either type of control. At extreme high stages, particularly where significant flow is conveyed in overbanks or flows through multiple roadway bridges, the control will include the influence of these features. At some stages a combination of section and channel control may occur. These are general rules and exceptions can and do occur. Knowledge of the channel features that control the stage-discharge relation is important. The development of stage-discharge curves where more than one control is effective, where control features change and where the number of measurements is limited, usually requires judgment in interpolating between measurements and in extrapolating beyond the highest or lowest measurements. This is particularly true where the controls are not permanent and tend to shift from time to time, resulting in changes in the positioning of segments of the stage-discharge relation.

1.2.1 Section control

A section control is a specific cross-section of a stream channel, located downstream from a water-level gauge that controls the relation between gauge height and discharge at the gauge. A section control can be a natural feature such as a rock ledge, a sand bar, a severe constriction in the channel or an accumulation of debris. Likewise, a section control can be a manmade feature such as a small dam, a weir, a flume, or an overflow spillway. Section controls can frequently be visually identified in the field by observing a riffle, or pronounced drop in the water surface, as the flow passes over the control. Frequently, as gauge height increases because of higher flows, the section control will become submerged to the extent that it no longer controls the relation between gauge height and discharge. At this point, the riffle is no longer observable, and flow is then regulated either by another section control further downstream or by the hydraulic geometry and roughness of the channel downstream (channel control).

1.2.2 Channel control

A channel control consists of a combination of features throughout a reach downstream from a gauge. These features include channel size, shape, curvature, slope and channel roughness. The length of channel reach that controls a stage-discharge relation varies. The stage-discharge relation for a relatively steep channel may be controlled by a relatively short channel reach whereas the relation for a relatively flat channel may be controlled by a much longer channel reach. In addition, the length of a channel control will vary depending on the magnitude of flow. Precise definition of the length of a channel-control reach is usually neither possible nor necessary.

1.2.3 Combination or compound controls

At some stages the stage-discharge relation may be governed by a combination of section and channel controls, sometimes referred to as compound controls. This usually occurs for a short range in stage between section-controlled and channel-controlled segments of the rating. This part of the rating is commonly referred to as a transition zone of the rating and represents the change from section control to channel control. In other instances, a combination control may consist of two section controls, where each has partial controlling effect. More than two controls acting simultaneously are rare. In any case, combination controls, and/or transition zones, occur for very limited parts of a stage-discharge relation and can usually be defined by plotting procedures. Transition zones in particular represent changes in the slope or shape of a stage-discharge relation.

1.3 GOVERNING HYDRAULIC EQUATIONS

Stage-discharge relations are hydraulic relations that can be defined according to the type of control that exists. Section controls, either natural or manmade, are governed by some form of the weir or flume equations. In a very general and basic form, these equations are expressed as:

$$Q = C_D B H^{\beta} \tag{1.1}$$

where Q is discharge, in cubic metres per second ($m^3 s^{-1}$); C_D is a coefficient of discharge and may include several factors; B is cross-section width, in metres (m); H is hydraulic head, in metres, and β is an exponent depending on the shape of the control (for example for V-shaped, $\beta = 2.5$ and for rectangular, $\beta = 1.5$).

Stage-discharge relations for channel controls with uniform flow are governed by the Manning or Chezy equation, as it applies to the reach of controlling channel downstream from a gauge. The Manning equation is:

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \tag{1.2}$$

where A is cross-section area, in square metres (m^2); R is hydraulic radius, in metres (m); S is friction slope, and n is channel roughness.

The Chezy equation is:

$$Q = CAR^{1/2}S^{1/2} \tag{1.3}$$

where C is the Chezy form of channel roughness.

The above equations are generally applicable for gradually varied, uniform flow. For highly varied, non-uniform flow, equations such as the Saint-Venant unsteady flow equations would be appropriate. However, these are seldom used in the development of stage-discharge relations, and are not described in this Manual.

1.4 COMPLEXITIES OF STAGE-DISCHARGE RELATIONS

Stage-discharge relations for stable controls such as a rock outcrop and manmade structures such as weirs, flumes and small dams usually present few problems in their calibration and maintenance. However, complexities can arise when controls are not stable and/or when variable backwater occurs. For unstable controls, segments of a stage-discharge relation may change position occasionally, or even frequently. This is usually a temporary condition which can be accounted for through the use of the shifting-control method.

Variable backwater can affect a stage-discharge relation, both for stable and unstable channels. Sources of backwater can be downstream reservoirs, tributaries, tides, ice, dams and other obstructions that influence the flow at the gauging station control. Methods of developing complex ratings for variable backwater conditions will be discussed in Chapters 2 and 3 of this Manual.

Another complexity that exists for some streams is hysteresis, which results when the water surface slope changes due to either rapidly rising or rapidly falling water levels in a channel control reach. Hysteresis ratings are sometimes referred to as loop ratings, and are most pronounced in relatively flat sloped streams. On rising stages the water surface slope is significantly steeper than for steady flow conditions, resulting in greater discharge than indicated by the steady flow rating. The reverse is true for falling stages. Details on hysteresis ratings will be discussed in a subsequent section of this chapter.

1.5 GRAPHICAL PLOTTING OF RATING CURVES

The relation between stage and discharge is defined by plotting measurements of discharge with corresponding observations of stage, taking into account whether the discharge is steady, increasing or decreasing and also noting the rate of change in stage. This may be done manually by plotting on paper or by using computerized plotting techniques. A choice of two types of plotting scale are available, either an arithmetic scale or a logarithmic scale. Each has certain advantages and disadvantages, as explained in subsequent paragraphs. It is customary to plot the stage as ordinate and the discharge as abscissa, although when using the stage-discharge relation to derive discharge from a measured value of stage, the stage is treated as the independent variable.

1.5.1 List of discharge measurements

The first step before making a plot of stage versus discharge is to prepare a list of discharge measurements that will be used for the plot. At a minimum this list should include at least 12 to 15 measurements, all made during the period of analysis. If the rating is segmented then more measurements may be required. These measurements should be well distributed over the range in gauge heights experienced. It should also include low and high measurements from other times that might be useful in defining the correct shape of the rating and for extrapolating the rating. Extreme low and high measurements should be included wherever possible.

For each discharge measurement in the list it is important that at least the following items are included:

(a) Unique identification number;
(b) Date of measurement;
(c) Gauge height of measurement. If there is a difference between inside and outside gauge readings, list both readings;
(d) Total discharge;
(e) Accuracy of measurement;
(f) Rate-of-change in stage during measurement, a plus sign indicating rising stage and a minus sign indicating falling stage.

Other information might be included in the list of measurements but is not mandatory. For instance, names of hydrographers making the measurement, time of measurement, difference between inside and outside gauge readings (if any), location of measurement, method of measurement and notes

Table II.1.1. Typical list of discharge measurements

ID number	Date	Made by	Width	Area	Mean velocity	Gauge height	Effective depth	Discharge	Method	Number verticals	Gauge height change	Rated
			m	*m*^{2}	*m/s*	m	m	*m*^{3}*/s*			*m/h*	
12	08/04/38	MEF	36.27	77.94	1.272	2.682	2.082	99.12	0.2/0.8	22	– 0.082	GOOD
183	06/02/55	GTC	33.53	78.41	1.405	2.786	2.186	11.02	0.6/0.2/0.8	22	– 0.047	GOOD
201	04/02/57	AJB	28.96	21.92	1.511	2.002	1.402	33.13	0.6/0.2/0.8	21	– 0.013	POOR
260	13/03/63	GMP	26.52	21.46	1.400	1.981	1.381	30.02	0.6	22	– 0.020	GOOD
313	24/08/66	HFR	30.18	42.08	1.602	2.374	1.774	67.40	0.6/0.2/0.8	22	+ 0.006	GOOD
366	21/08/73	MAF	28.96	14.86	0.476	1.557	0.957	7.080	0.6	21	0	GOOD
367	10/10/73	MAF	28.96	13.66	0.361	1.490	0.890	4.928	0.6	21	0	GOOD
368	26/11/73	MAF	29.26	14.21	0.373	1.509	0.909	5.296	0.6	18	0	GOOD
369	19/02/74	MAF	29.87	16.26	1.291	1.838	1.238	20.99	0.6	21	0	GOOD
370	09/04/74	MAF	29.26	21.27	0.805	1.780	1.180	17.13	0.6/0.2/0.8	21	0	GOOD
371	29/05/74	MAF	29.57	19.69	0.688	1.710	1.110	13.54	0.6	21	0	GOOD
372	10/07/74	MAF	28.96	16.81	0.458	1.573	0.973	7.703	0.6	21	0	GOOD
373	22/08/74	MAF	29.26	15.79	0.481	1.570	0.970	7.590	0.6	21	0	GOOD
374	01/10/74	MAF	29.26	13.19	0.264	1.414	0.814	3.483	0.6	21	0	GOOD
375	11/11/74	MAJ	28.96	11.71	0.283	1.396	0.796	3.313	0.6	21	0	GOOD
382	01/10/75	MAF	30.48	43.76	1.598	2.432	1.832	69.95	0.2/0.8	21	+ 0.017	GOOD

about the condition of the control. Table II.1.1 shows a typical list of discharge measurements, including a number of items in addition to the mandatory items. The discharge measurement list may be handwritten for use when hand-plotting is done or the data may be a computer list where a computerized plot is developed.

1.5.2 Use of inside and outside gauge readings

At some recording gauging stations there is often a difference between recorded (inside) gauge heights and outside gauge heights during periods of high stage. When that happens, both inside and outside gauge heights for discharge measurements should be recorded on the form shown in Table II.1.1. When plotting the measurements for rating analysis, the outside gauge readings are used first. The stage-discharge relation is extended to the stage of the outside high water marks that are observed for each flood event. The stage-discharge relation is next transposed to correspond with the inside gauge heights obtained from the stage recorder at the times of discharge measurement and at flood peaks. The rationale behind this procedure is as follows. The outside gauge readings are used for developing the rating because the hydraulic principles on which the rating is based require the use of the true stage of the stream. The transposition of the rating to inside (recorded) stages is then made because the recorded stages will be used with the rating to determine discharge. The recorded stages are used for discharge determination because if differences exist between inside and outside gauge readings, those differences will be known only for those times when the two gauges are read concurrently. If the outside gauge heights were used with the rating to determine discharge, variable corrections, either known or assumed, would have to be applied to the recorded gauge heights to convert them to outside stages. Differences between inside and outside gauge heights do exist for some stations and must be recognized in the development of ratings. However in the discussions that follow no distinction between the two gauges will be made.

1.5.3 Arithmetic plotting scales

The simplest type of plot uses an arithmetically divided plotting scale as shown in Figure II.1.1. This plot uses calibration measurements shown in Table II.1.1. Scale subdivisions should be chosen to cover the complete range of gauge height and discharge expected to occur at the gauging site. Scales should be subdivided in uniform, even increments that are easy to read and interpolate. They should also be chosen to produce a rating curve that is not unduly steep or flat. Usually the curve should follow a slope of between 30° and 50°. If the range in gauge height or discharge is large, it may be necessary to plot the rating curve in two or more segments to provide scales that are easily read with the necessary precision. This procedure may result in separate curves for low water, medium water and high water. Care should be taken to see that, when joined, the separate curves form a smooth, continuous combined curve.

The use of arithmetic co-ordinate paper for rating analysis has certain advantages, particularly in the study of the pattern of rating shifts in the lower part of the rating. A change (shift) in the low-flow rating at many sites results from a change in the gauge height of effective zero flow, which means a constant shift in gauge height. A shift of that kind is more easily visualized on arithmetic co-ordinate paper because on that paper the shift curve is parallel to the original rating curve. The two curves are separated by a vertical distance equal to the change in the value of the gauge height of zero flow. A further advantage of arithmetic co-ordinate paper is the fact that the gauge height of zero flow can be plotted directly on arithmetic co-ordinate paper, thereby facilitating extrapolation of the low water end of the rating curve. That cannot be done on logarithmic paper because zero values cannot be shown on that type of paper.

For analytical purposes arithmetic scales have practically no advantage. For this reason, logarithmic plotting should always be used initially in developing the general shape of the rating. The final curve may be displayed on either type of graph

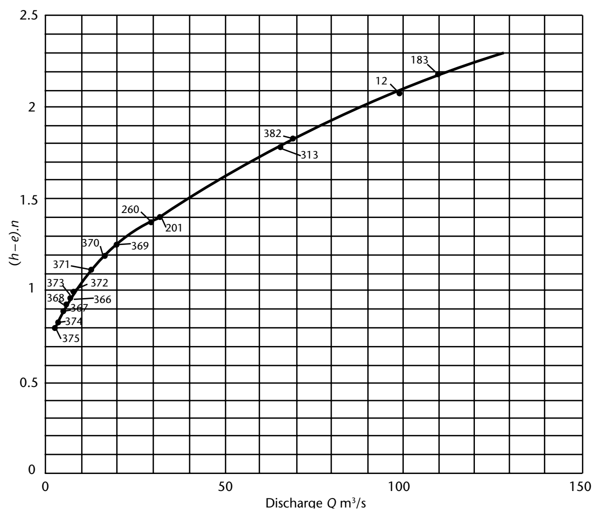


Figure II.1.1. Arithmetic plot of stage-discharge relation

paper and used as a base curve for the analysis of shifts. A combination of the two types of graph paper is frequently used with the lower part of the rating plotted on an inset of rectangular co-ordinate paper or on a separate sheet of rectangular co-ordinate paper.

1.5.4 Logarithmic plotting scales

Most stage-discharge relations, or segments thereof, are best analyzed graphically through the use of logarithmic plotting paper. To utilize fully this procedure, gauge height should be transformed to effective depth of flow on the control by subtracting from it the effective gauge height of zero discharge. Using these conditions, the slope of the rating will conform to the type of control (section or channel), thereby providing valuable information to shape correctly the rating curve segment. In addition, this feature allows the analyst to calibrate the stage-discharge relation with fewer discharge measurements. The slope of a rating curve is the ratio of the horizontal distance to the vertical distance. This non-standard way of measuring slope is necessary because the dependent variable (discharge) is always plotted as the abscissa.

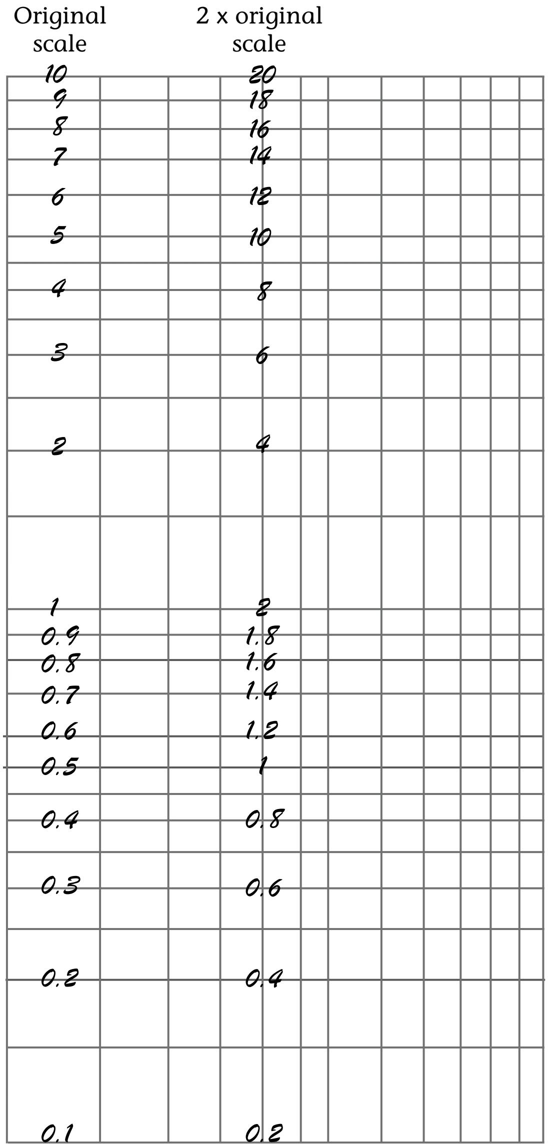


Figure II.1.2. Example showing how the logarithmic scale of graph paper may be transposed

The measured distance between any two abscissas on logarithmic graph paper, whose values are printed or indicated on the sheet by the manufacturer of the paper, represents the difference between the logarithms of those values. Consequently, the measured distance is related to the ratio of the two values. Therefore, the distance between pairs of numbers such as 1 and 2, 2 and 4, 3 and 6, 5 and 10, are all equal because the ratios of the various pairs are identical. Thus the logarithmic scale of either the ordinates or the abscissas is maintained if all printed numbers on the scale are multiplied or divided by a constant. This property of the paper has practical value. For example, assume that the logarithmic plotting paper available has 2 cycles, as shown in figure 1.2 and that ordinates ranging from 0.3 to 15.0 are to be plotted. If the printed scale of ordinates is used and the bottom line is called 0.1, the top line of the paper becomes 10.0, and values between 10.0 and 15.0 cannot be accommodated. However, the logarithmic scale will not be distorted if all values are multiplied by a constant. For this particular problem, 2 is the constant used in Figure II.1.2, and now the desired range of 0.3 to 15.0 can be accommodated. Examination of Figure II.1.2 shows that the change in scale has not changed the distance between any given pair or ordinates; the position of the ordinate scale has merely been transposed.

A rating curve that plots as a straight line on logarithmic paper has the equation:

$$Q = C\left(h - e\right)^{\beta} \tag{1.4}$$

where Q is discharge; h is gauge height of the water surface; e is gauge height of zero flow for a control of regular shape, or of effective zero flow for a control of irregular shape; $(h - e)$ is head or depth of water on the control. This value is indicated by the ordinate scale printed by the manufacturer or by the ordinate scale that has been transposed, as explained in the preceding paragraph; C is the discharge when the head $(h - e)$ equals 1.0; β is slope of the rating curve. Slope in equation 1.4 is the ratio of the horizontal distance to the vertical distance.

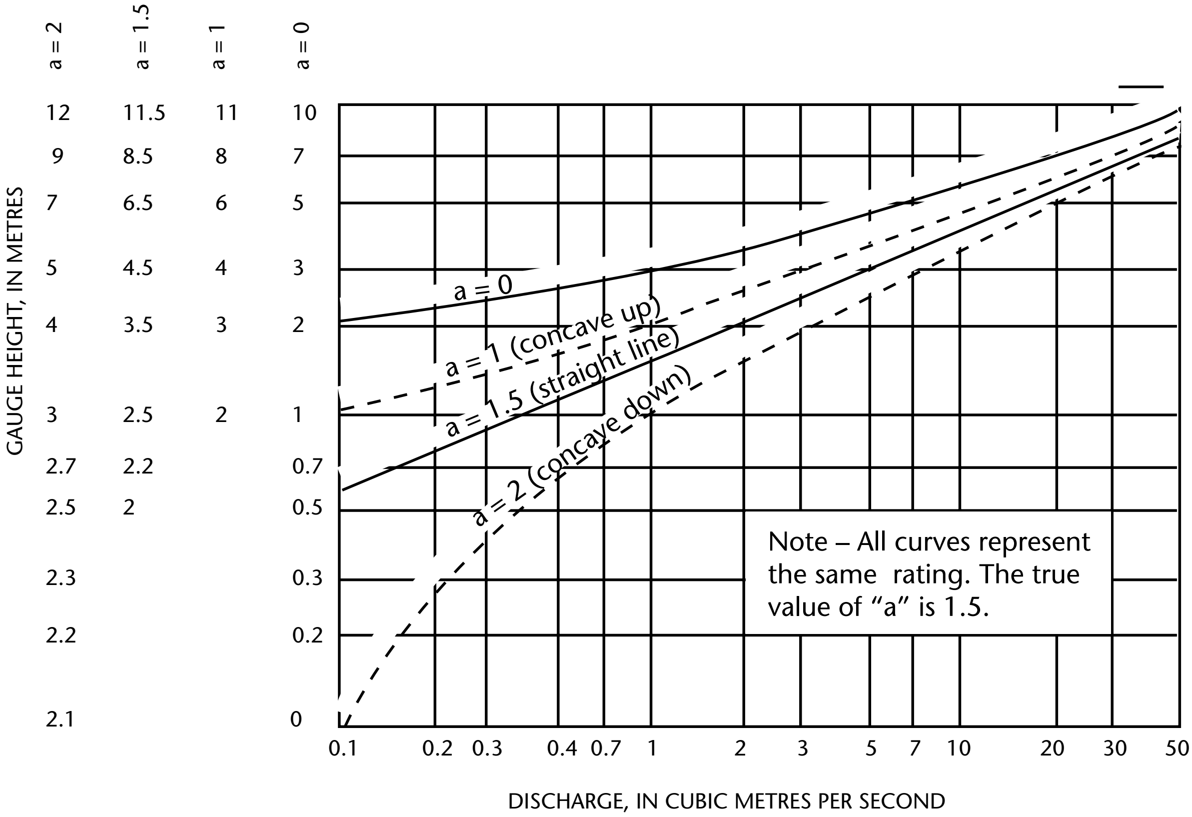


Figure II.1.3. Rating curve shapes resulting from the use of differing values of effective zero-flow

Changes to the control will have predictable results on the logarithmic rating plot. If the width of the control increases, C increases and the new rating will be parallel to and to the right of the original rating. If the width of the control decreases, the opposite effect occurs; C decreases and the new rating will be parallel to and to the left of the original rating. If the control scours, e decreases and the depth $(h - e)$ for a given gauge height increases; the new rating moves to the right and will no longer be a straight line but will be a curve that is concave downward. If the control becomes built up by deposition, e increases and the depth $(h - e)$ for a given gauge height decreases; the new rating moves to the left and is no longer linear, but is a curve that is concave upward.

When discharge measurements are originally plotted on logarithmic paper no consideration is usually given to values of e. The gauge height of each measurement is plotted using the ordinate scale provided by the manufacturer or, if necessary, an ordinate scale that has been transposed as illustrated in Figure II.1.2. Referring to Figure II.1.3, the inside scale (e = 0) is the scale printed by the paper manufacturer. Assume that the discharge measurements have been plotted to that scale and that they define the curvilinear relation between gauge height h, and discharge Q, that is shown in the topmost curve. For the purpose of extrapolating the relation, a value of e is sought, which when applied to h, will result in a linear relation between $(h - e)$ and Q. For a section control of regular shape, the value of e will be known; it will be the gauge height of the lowest point of the control (gauge height of zero flow). For a channel control or section control of irregular shape, the value of e is the gauge height of effective zero flow; that may be determined by successive approximations.

In successive trials, the ordinate scale in Figure II.1.3 is varied for e values of 1 m, 1.5 m and 2 m, each of which results in a different curve, but each new curve still represents the same rating as the top curve. For example, a discharge of 3 $m^3 s^{-1}$ corresponds to a gauge height h of 4 m on all four curves. The true value of e is 1.5 m, and thus the rating plots as a straight line if the ordinate scale numbers are increased by that value. In other words, while even on the new scale a discharge of 3 $m^3 s^{-1}$ corresponds to a gauge height h of 4 m, the head or depth on the control for a discharge of 3 $m^3 s^{-1}$ is $(h - e)$, or 2.5 m; the linear rating marked e = 1.5 m crosses the ordinate for 3 $m^3 s^{-1}$ at 4 m on the new scale and at 2.5 m on the manufacturer's, or inside scale. If values of *e* smaller than the true value of 1.5 m are used, rating curve will be concave upward; if values of *e* greater than 1.5 m are used, curve will be concave downward. The value of *e* to be used for a rating curve, or a segment of a rating curve, can thus be determined by adding or subtracting trial values of *e* to the numbered scales on the logarithmic plotting paper until a value is found that results in a straight line plot of the rating. It is important to note that if the logarithmic ordinate scale must be transposed by multiplication or division to accommodate the range of stage to be plotted, that transposition must be made before the ordinate scale is manipulated for values of *e*.

1.5.5 Computer plotting of discharge measurements and rating curves

Plotting of discharge measurements and rating curves, either arithmetic plots or logarithmic plots, is best done by computer. These plots can be viewed on the computer monitor and/or plotted on paper forms. Advantages of computer plots are:
(a) Selection of measurements for plotting can be made quickly and easily;
(b) Scale changes can be made and measurements replotted quickly;
(c) Various values of *e* can be easily tried for the purpose of defining a straight-line rating on logarithmic plots;
(d) Separate rating segments, representing different control conditions, can be easily and quickly plotted;
(e) Rating analysis, as described in the subsequent section, is accomplished easily;
(f) Plotting errors are virtually eliminated.

Logarithmic plots of rating curves must meet the requirement that the log cycles are square. That is, the linear measurement of a log cycle, both horizontally and vertically, *must* be equal. Otherwise, it is impossible to hydraulically analyze the resulting plot of the rating. This requirement for square log cycles should always be tested because some computer programs do not include this as an automatic feature.

1.5.6 Analysis of rating curves

Rating curves for section controls such as a weir or flume conform to equation 1.1, and when plotted logarithmically the slope will be 1.5 or greater depending on control shape, velocity of approach and minor variations of the coefficient of discharge. Logarithmic rating curves for most weir shapes will plot with a slope of 2 or greater. An exception is the sharp-crested rectangular weir, which plots with a slope slightly greater than 1.5. Logarithmic ratings for section controls in natural channels will almost always have a slope of 2 or greater. This characteristic slope of 2 or greater for most section controls allows the analyst to identify easily the existence of section control conditions simply by plotting discharge versus effective depth, $(h - e)$, on logarithmic plotting paper.

Rating curves for channel controls, on the other hand, are governed by equation 1.2 or 1.3, and when plotted as effective depth versus discharge the slope will usually be less than 2. Variations in the slope of the rating when channel control exists are the result of changes in channel roughness and friction slope as depth changes.

The above discussion applies to control sections of regular shape (triangular, trapezoidal and parabolic). When a significant change in shape occurs, such as a trapezoidal section control with a small V-notch for extreme low water, there will be a change in the rating curve slope at the point where the control shape changes. Likewise, when the control changes from section control to channel control, the logarithmic plot will show a change in slope. These changes are usually defined by short curved segments of the rating, referred to as transitions. This kind of knowledge about the plotting characteristics of a rating curve is extremely valuable in the calibration and maintenance of the rating, and in later analysis of shifting control conditions. By knowing the kind of control (section or channel) and the shape of the control, the analyst can more precisely define the correct hydraulic shape of the rating curve. In addition, these kinds of information allow the analyst to extrapolate accurately a rating curve, or conversely, know when extrapolation is likely to lead to significant errors.

Figure II.1.4 gives examples of a hypothetical rating curve showing the logarithmic plotting characteristics for channel and section controls and for cross-section shape changes. Insert A in Figure II.1.4 shows a trapezoidal channel with no flood plain and with channel control conditions. The corresponding logarithmic plot of the rating curve, when plotted with an effective gauge height of zero flow *e* that results in a straight line rating, has a slope less than 2. In insert B a flood plain has been added which is also channel control. This is a change to the shape of the control cross section and results in a change in the shape of the rating curve above bankfull stage. If the upper segment (above the transition curve) were re-plotted to the correct value of effective gauge height of zero flow, it too would have a slope less than 2. In the third plot, insert C, a section control for low flow has been added. This results in a change in rating curve shape because of the change in control. For the low water part of the rating, the slope will usually be greater than 2.

Figure II.1.5 is a logarithmic plot of an actual rating curve, using the measurements shown inTable II.1.1. This rating is for a real stream where section control exists throughout the range of flow, including the high flow measurements. The effective gauge height of zero flow *e* for this stream is 0.6 metres, which is subtracted from the gauge height of the measurements to define the effective depth of flow at the control. The slope of the rating below 1.4 m is about 4.3, which is greater than

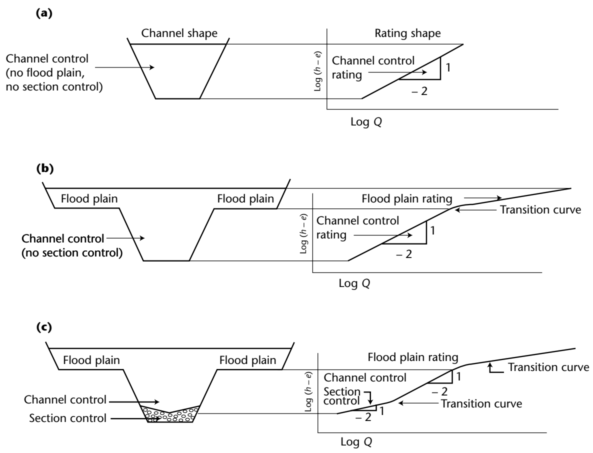


Figure II.1.4. Relation of channel and control properties to rating curve shape

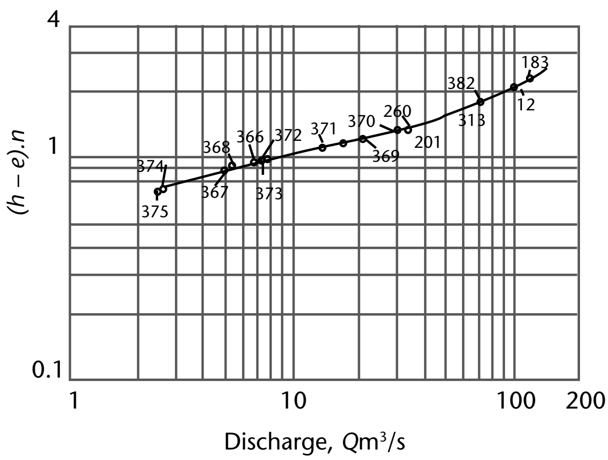


Figure II.1.5. Logarithmic plot of stage-discharge relation

2 and conforms to a section control. Above 1.5 m, the slope is 2.8, which also conforms to a section control. The change in slope of the rating above about 1.5 m is caused by a change in the shape of the control cross-section. Below about 1.4 m the control section is essentially a triangular shape. In the range of 1.4 to 1.5 m the control shape is changing to trapezoidal, resulting in the transition curve of the rating. And above about 1.5 m the control cross-section is basically trapezoidal.

The examples of Figures II.1.4 and II.1.5 are intended to illustrate some of the principals of logarithmic plotting. The analyst should try to use these principals to the best extent possible, but should always be aware that there are probably exceptions and differences that occur at some sites. Mathematical derivation and additional examples of rating analysis is given in subsequent sections of this chapter.

1.6 RATINGS FOR ARTIFICIAL SECTION CONTROLS

Knowledge of the rating characteristics of artificial section controls of standard shape is necessary for an understanding of the rating characteristics of natural controls, almost all of which have irregular shape. The structures detailed in Volume I, Chapter 7 may be used as controls for velocity area stations, the low or medium flows being measured by the structure using the laboratory rating, the medium and high flows being measured by current meter. For such dual purpose stations the laboratory rating is used up to the modular limit and the rating continued by current meter. For structures such as the Crump weir or Flat-V weir where a crest tapping is used the range of the structure is increased into the non-modular region until complete drowning takes place. Flows above this limit will be dependent on a downstream control or a channel control.

If the weirs described in Volume I, Chapter 7 are compounded without divide walls being incorporated in the design a loss of accuracy will occur if the laboratory ratings are used. This loss of accuracy may not be serious but it is recommended that all such non-standard variations be field calibrated. The calibration of a non standard broad-crested weir follows.

1.6.1 Notched flat-crested rectangular weir

Figure II.1.6 shows the notched flat-crested rectangular weir that is the control for a gauging

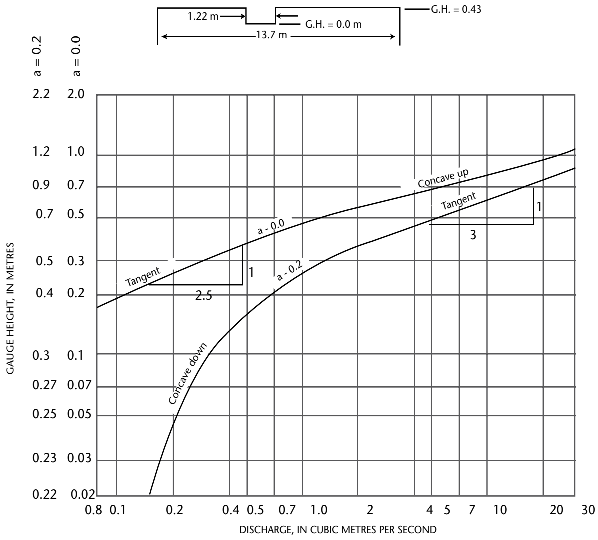


Figure II.1.6. Rating curve for a notched broad-crested control

station on Great Trough Creek near Marklesburg, Pennsylvania, United States of America. Because there is a sharp break in the cross-section at gauge height 0.43 m a break occurs in the slope of the rating curve at that stage. The gauge height of zero flow for stages between 0.0 and 0.43 m is 0.0 m; for stages above 0.43 m the effective gauge height of zero flow is at some point between 0.0 and 0.43 m. If the low end of the rating is made a tangent, the gauge height of zero flow (*e*) is 0.0 m and the slope of this tangent turns out to be 2.5, which, as now expected, is greater than the theoretical slope of 1.5. The upper part of this rating curve is concave upward because the value of *e* used (0.0 m) is lower than the effective value of zero flow for high stages.

If the upper end of the rating is made a tangent, it is found that the value of *e*, or effective zero flow, must be increased to 0.2 m. Because we have raised the value of *e*, this will make the low water end of the curve concave downward. The high water tangent of the curve, principally because of increased rate of change of velocity of approach, will have a slope that is greater than that of the low water tangent of the curve previously described. Its slope is found to have a value of 3.0.

The low water tangent for the notched control, which is defined by discharge measurements, warrants further discussion. Its slope of 2.5 is higher than one would normally expect for a simple flat-crested rectangular notch. Reasons for this may be:

(a) The velocity of approach factor is included in the rating;
(b) A thin-plate weir is fixed to the downstream edge of the notch with its elevation about 0.03 m above the base of the notch;
(c) Probably more important, the width of the notch is small compared to the total width of the control; this may alter the flow characteristics to the extent that the notch may in fact be operating between rectangular and v-notch conditions.

These observations are mentioned here only to warn the reader not to expect a slope as great as 2.5 in the rating for a simple flat-crested rectangular notch. In fact, the sole purpose here of discussing the low water tangent of the rating curve is to demonstrate the effect exerted in the curve by varying the applied values of *e*. It should also be noted that the slope of the rating for section controls will almost always be greater than 2, as discussed in previous sections of this chapter.

The low water end of a rating curve is usually well defined by discharge measurements. If it is necessary to extrapolate the rating downward it is best done by re-plotting the low water end of the curve on arithmetic coordinate graph paper and extrapolating the curve down to the point of zero discharge.

1.6.2 **Trenton-type control**

The so-called Trenton-type control is a concrete weir that is popular in the United States. The dimensions of the cross-section of the crest are shown in Figure II.1.7. The crest may be constructed so as to be horizontal for its entire length across the stream or for increased low flow sensitivity the crest may be given the shape of an extremely flat V. For a horizontal crest, the equation of the stage discharge relation, as obtained from a logarithmic plot of the discharge measurements, is commonly of the order of:

$$Q = 2.31bh^{1.65} \tag{1.5}$$

where b = top width of water surface, in metres.

The precise values of the constants will vary with the height of the weir above the stream-bed because that height affects the velocity of approach. The constants of the equation are greater than those for a flat-crested rectangular weir (see Volume I, Chapter 7) because the cross-sectional shape of the Trenton-type control is more efficient than a rectangle, with regard to the flow of water.

When the Trenton-type control is built with its crest in the shape of a flat V, the exponent of h in the discharge equation is usually 2.5 or more, as expected for a triangular notch where velocity of approach is significant. The precise values of the constants in the discharge equation are dependent on the geometry of the installation.

1.6.3 **Columbus-type control**

One of the most widely used controls in the United States is the Columbus-type control, as shown in Figure II.1.8. This control is a concrete weir with a parabolic notch that is designed to give accurate measurements of a wide range of flows.

The notch accommodates low flows; the main section, whose crest has a flat upward slope away from the notch, accommodates higher flows. The throat of the notch is convex along the axis of flow to permit the passage of debris. For stages above a head of 0.2 m, which is the elevation of the top of the notch, the elevation of effective zero flow is 0.06 m, and the equation of discharge is approximately:

$$Q = 12.14\left(h - 0.061\right)^{3.3} \tag{1.6}$$

The precise values of the constants in the equation will vary with conditions for each installation. The shape of the crest above a stage of 0.2 m is essentially a flat V, for which the theoretical exponent of head is 2.5 in the discharge equation. However, the actual value of the exponent is greater than 2.5 principally because of the increase of velocity of approach with stage.

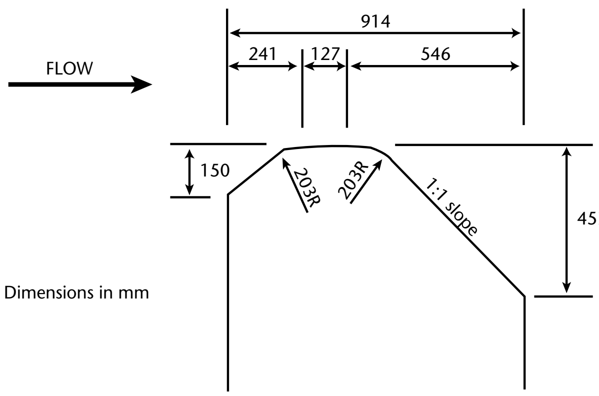
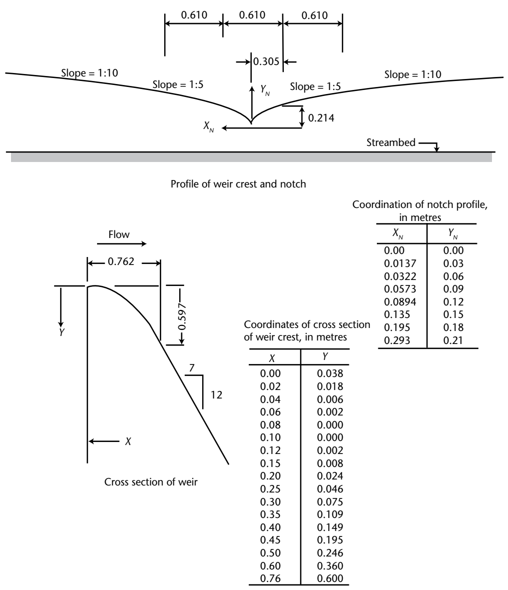


Figure II.1.7. Cross section of Trenton-type control



Coordination of notch profile, in metres

X_N	Y_N
0.00	0.00
0.0137	0.03
0.0322	0.06
0.0573	0.09
0.0894	0.12
0.135	0.15
0.195	0.18
0.293	0.21

Coordinates of cross section of weir crest, in metres

X	Y
0.00	0.038
0.02	0.018
0.04	0.006
0.06	0.002
0.08	0.000
0.10	0.000
0.12	0.002
0.15	0.008
0.20	0.024
0.25	0.046
0.30	0.075
0.35	0.109
0.40	0.149
0.45	0.195
0.50	0.246
0.60	0.360
0.76	0.600

Figure II.1.8. Dimensions of Columbus-type control

1.7 **RATINGS FOR NATURAL SECTION CONTROLS**

Natural section controls, listed in order of permanence, are usually a rock ledge outcrop across the channel, a riffle composed of loose rock, cobbles and gravel or a gravel bar. Less commonly, the section control is a natural constriction in width of the channel, or is a sharp break in channel slope, as at the head of a cascade or brink of a falls. The equation for ratings of natural section controls follows the form of equation 1.4.

Where the control is a rock outcrop, riffle or gravel bar, the stage-discharge relation, when plotted on logarithmic paper, conforms to the general principles discussed for broad-crested artificial controls. If the natural control is essentially horizontal for the entire width of the control the head on the control is the difference between the gauge heights of the water surface and the crest of the control. The exponent β of the head in the equation of discharge, (equation 1.4) will be greater than the theoretical value 1.5 primarily because of the increase in velocity of approach with stage. If the crest of the control has a roughly parabolic profile, as most natural controls have (greater depths on the control near midstream), the exponent β will be even larger because of the increase in width of the stream with stage, as well as the increase in velocity of approach with stage. The value of β will almost always exceed 2.0 and a range of β from 1 to 4 is common for natural controls. If the control is irregularly notched, as is often the case, the gauge height of effective zero flow e for all but the lowest stages will be somewhat greater than that for the lowest point in the notch.

The above principles are also roughly applicable to the discharge equations for an abrupt width contraction or an abrupt steepening of bed slope. The exponent and the gauge height of effective zero flow are influenced by the transverse profile of the stream-bed at the control cross-section.

1.8 RATINGS FOR NATURAL COMPOUND SECTION CONTROLS

If a natural control section is a local rise in the stream-bed, such as at a rock outcrop riffle or gravel bar, that cross-section is invariably a control only for low flows. The gauging station in that circumstance has a compound control with the high flows being subject to channel control. Occasionally there is a second outcrop or riffle down stream from the low water riffle that acts as a section control for flows of intermediate magnitude. When the control for intermediate stages is effective it causes submergence of the low water control. At high flows the section control for intermediate stages is in turn submerged when channel control becomes effective. An example of a compound control involving two section controls is shown in Figure II.1.9.

Figure II.1.8 shows the rating for the compound section control at the gauging station on Muncy Creek near Sonestown, Pennsylvania, United States. The control consists of two rock riffles, effective

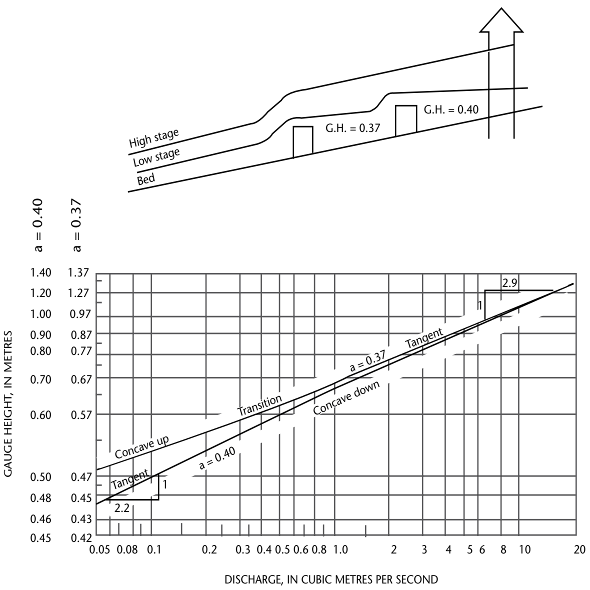


Figure II.1.9. Rating curve for a compound section control at Muncy Creek near Sonestown, Pennsylvania, United States

zero flow *e* for very low stages being at gauge height 0.40 m and for higher stages at gauge height 0.37 m. If the low end of the rating is made a tangent it means that too large a value of *e* is used for the high end of the rating (0.40 m versus 0.37 m) and the high water end of the curve becomes concave downward. Conversely, if the high end of the curve is made a tangent, the low water end of the curve becomes concave upward. The high water tangent of the one curve has a greater value of β than the low water tangent of the other curve. This difference in the values of β reflects the effect of differences in the geometries of the two controls as well as the effect of increased rate of change of approach velocities at the higher stages. The slopes of the two tangents are 2.9 and 2.2, both values being greater than the theoretical slope of 1.5.

1.9 RATINGS FOR STABLE CHANNEL CONTROLS

The term stable channel, as used in this Manual is a relative term. Virtually all natural channels are subject to at least occasional change as a result of scour, deposition or the growth of vegetation. But some alluvial channels, notably those whose bed and banks are composed of sand, have movable boundaries that change almost continuously, as do their stage-discharge relations. For the purpose of this manual stable channels include all but sand channels. Sand channels are discussed in section 1.14, Sand Chanel Streams.

Almost all streams that are unregulated by man have channel control at the higher stages. Among those with stable channels, all but the largest rivers have section control at low stages. Because this section of the manual discusses only stable channels that have channel control for the entire range of stage experienced, the discussion is limited to the natural channels of extremely large rivers and to artificial channels constructed without section controls. The artificial channels may be concrete lined, partly lined, rip-rapped or unlined. Streams that have compound controls involving channel control are discussed in section 1.10.

The discharge equation for the condition of channel control is the Manning equation, as discussed in Volume I, Chapter 9, and is shown as equation 1.2 in this Volume. In analyzing an artificial channel of regular shape, whose dimensions are fixed, flow at the gauge is first assumed to be at uniform depth. Consequently for any stage all dimensions on the right side of the equations are known except *n*.

A value of n can be computed for a single discharge measurement or an average value of n can be computed from a pair of discharge measurements. Thus a preliminary rating curve for the artificial channel can be computed for the entire range of stage from the results of a pair of discharge measurements. If subsequent discharge measurements depart from the computed rating curve it is likely that the original assumption of flow at uniform depth was erroneous. That means that the energy slope, S, is not parallel to the bed slope but varies with stage, and that the value of n, which was computed on the basis of bed slope, is also in error. The rating curve must be revised to fit the plotted discharge measurements, but the preliminary rating curve may be used as a guide in shaping the required extrapolation of the rating curve. The extrapolation should also be checked by application of the conveyance-slope method of rating extrapolation, which is described in section 1.11.2.

To understand the principles that underlie the stage-discharge relation of channel control in a natural channel of irregular shape assume, that the roughness coefficient, n, in the Manning equation is a constant at the higher stages and that the energy slope, S, tends to become constant. Furthermore, area, A, is approximately equal to depth, H, times width, W. By making the substitution for A in the equation 1.7 and 1.8, and by expressing $S^{1/2}/n$ as a constant, C_1, the following equation is obtained:

$$Q = C_1 HWR^{2/3} \text{ (approx)} \qquad (1.7)$$

If the hydraulic radius, R, is considered equal to H and W is considered a constant the equation becomes:

$$Q = CH^{1.67} = C\left(h - e\right)^{1.67} \text{(approx)} \qquad (1.8)$$

However, unless the stream is exceptionally wide, R is appreciably smaller than H. This has the effect of reducing the exponent in the last equation, although this reduction may be offset by an increase of S or W with discharge. Changes in roughness with stage will also affect the value of the exponent. The net result of all these factors is a discharge equation of the form:

$$Q = C\left(h - e\right)^{\beta} \qquad (1.9)$$

where β will commonly vary between 1.3 and 1.8 and seldom reach a value as high 2.0.

An example of a discharge rating for channel control in a natural stream is in the following section, where compound controls that involve channel control are discussed.

1.10 RATINGS FOR COMPOUND CONTROLS INVOLVING SECTION AND CHANNEL CONTROL

A compound control of the stage-discharge relation usually exists in natural channels. Section controls are more effective for the lower stages and channel control are more effective for the higher stages. An example of that situation is given in Figure II.1.10, the rating curve for the Susquehanna River at Harrisburg, Pennsylvania, United States. The low water control is a low weir with zero flow at gauge height 0.67 m. At a stage of 1.19 m this control starts to drown out and channel control becomes effective. If the low end of the rating is made a tangent, a value of $e = 0.67$ m must be used. Because the value of e for the upper end of the rating is something less than 0.67 m, the high end becomes concave downward. If the high end of the curve is made a tangent, the effective value of e is found to be 0.0 m. Because this is too low a value of e for the lower end of the curve, the low end becomes concave upward.

Where the rating for a section control (low end of the curve) is a tangent, the value of β is expected to be greater than 2.0. In this example β = 2.3. Where the rating for a channel control (high end of the curve) is a tangent, the value of β is expected to be less than 2.0 and probably between 1.3 and 1.8. In this example β = 1.3. Should overbank flow occur the rating curve will bend to the right. It can be demonstrated, non-rigorously, that straight line rating curves for section controls

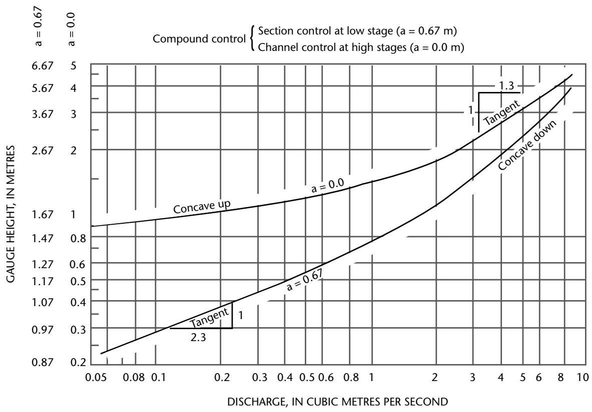


Figure II.1.10. Rating curve for a compound control at Susquehanna River at Harrisburg, Pennsylvania, United States

almost always have a slope greater than 2.0 and that those for channel controls have a slope less than 2.0. This was mentioned in the beginning of this Chapter in the section on logarithmic plotting of rating curves.

1.11 EXTRAPOLATION OF RATING CURVES

More often than not, rating curves must be extrapolated beyond the range of measured discharges. The preceding material in this Chapter explained the principles governing the shape of logarithmic rating curves to guide the hydrologist in shaping the extrapolated segment of a rating. However, even with knowledge of those principles, an element of uncertainty exists in the extrapolation. The purpose of this section of the manual is to describe methods of analysis that will reduce the degree of uncertainty.

1.11.1 Low flow extrapolation

Low flow extrapolation is best performed on arithmetic coordinate graph paper because the coordinates of zero flow can be plotted on such paper. Zero discharge cannot be plotted on logarithmic graph paper. An example of such extrapolation is shown in Figure II.1.11, where the circled points represent discharge measurements plotted on the coordinate scales of gauge height versus discharge. The rating in the example is defined by the measurements down to a gauge height of 0.090 m, but an extrapolation to a gauge height of 0.040 m is required. Field observation has shown the low point on the control (gauge height of zero flow) to be at gauge height 0.027 m.

The method of extrapolation in Figure II.1.11 is self-evident. A curve has been drawn between the plotted points at gauge heights 0.027 m and 0.090 m, to merge smoothly with the rating curve above 0.090 m. There is no assurance that the extrapolation is precise. Low flow discharge measurements are required for that assurance but the extrapolation shown is a reasonable one.

The weir equation 1.1 will apply to low-water extrapolations where section control exists. Values of the discharge coefficient, C, can be calculated from discharge measurements and cross-section data of the control and then extrapolated to the range of interest below the lowest discharge measurement. This is a good technique for defining the shape of the rating curve in the range where discharge measurements are not available.

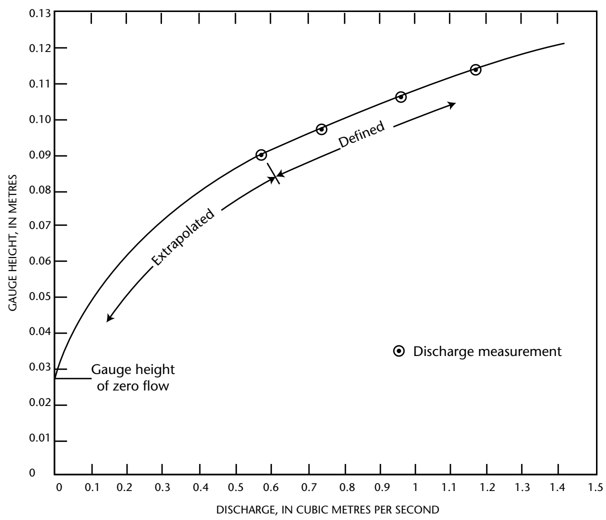


Figure II.1.11. Example of low-flow extrapolation on arithmetic-coordinate graph paper

1.11.2 High flow extrapolation

As mentioned in a previous section of this chapter, the problem of high flow extrapolation can be avoided if the unmeasured peak discharge for the rating is determined by the use of the indirect methods discussed in Volume I, Chapter 9. In the absence of such peak discharge determinations, estimates of the discharges corresponding to high values of stage may be made by using one or more of the following four techniques:
(a) Conveyance slope method;
(b) Areal comparison of peak runoff rates;
(c) Flood routing;
(d) Step backwater method using models such as HEC-2 or WSPRO.

The knowledgeable reader of this Manual may notice the absence from the above list of two techniques that were once standard practice: the velocity-area method and Q vs $Ad^{1/2}$ method. The Q vs $Ad^{1/2}$ method was superior to the velocity-area method and largely supplanted it. Similarly, the conveyance-slope method, because of its superiority, has largely supplanted s Q vs $Ad^{1/2}$ method. Of the three somewhat similar methods only the conveyance-slope method is described here because a description of the two earlier methods (Corbett, 1945) would have only academic rather than practical value.

Conveyance slope method

The conveyance slope method is based on equations of steady flow, such as the Chezy or Manning equation. In those equations:

$$Q = KS^{1/2} \tag{1.10}$$

In the Chezy equation:

$$K = CAR^{1/2} \tag{1.11}$$

and in the Manning equation:

$$K = \frac{1}{n} AR^{2/3} \quad \text{(metric units)} \tag{1.12}$$

Values of cross section area, A, and hydraulic radius, R, corresponding to any stage can be obtained from a field survey of the discharge measurement cross-section. Values of the coefficient C or n can be estimated in the field. Thus, the value of K, which embodies all the elements that can be measured or estimated, can be computed for any given stage. It can also be shown that errors in estimating C or n are usually not critical. Values of gauge height vs K, covering the complete range of stage up to the required peak gauge height, are computed and plotted on arithmetic graph paper. A smooth curve is fitted to the plotted points.

Values of slope, S, which is actually the energy gradient, are usually not available even for measured discharges. However for the measured discharges, $S^{1/2}$ can be computed by dividing each measured discharge by its corresponding K value. Slope, S, is then obtained by squaring the resulting value of $S^{1/2}$. Values of gauge height versus S for measured discharges are plotted on arithmetic graph paper and a curve is fitted to the plotted points. The curve is extrapolated to the required peak gauge height. The extrapolation is guided by the knowledge that S tends to become constant at the higher stages. That constant slope is the normal slope, or slope of the streambed. The upper end of the defined part of the curve of gauge height versus S indicates that a constant or near constant value of S has been attained and the extrapolation of the curve can be made with confidence. The discharge for any particular gauge height will be obtained by multiplying the corresponding value of K from the K curve by the corresponding value of $S^{1/2}$ from the S curve. We see that errors in estimating n will have a minor effect because the resulting percentage error in computing K is compensated by a similar percentage error in the opposite direction in computing $S^{1/2}$. In other words, the constancy of S is unaffected, but if K is, say, 10 per cent high, $S^{1/2}$ will be 10 per cent low and the two discrepancies are cancelled when multiplication is performed. However, if the upper end of the define part of the curve of gauge height versus S has not reached the stage where S has a near constant value, the extrapolation of the curve will be subject to uncertainty. In that situation the general slope of the streambed, as determined from a topographic map, provides a guide to the probable constant value of S that should be attained at high stages.

As mentioned in the preceding paragraph, the discharge for any particular gauge height is obtained by the multiplication of appropriate values of K and $S^{1/2}$, and in that manner the upper end of the stage-discharge relation is constructed.

Figure II.1.12 provides an example of the slope conveyance method, as used for rating curve extrapolation at the gauging station on Klamath River at Somes Bar California, United States. The conveyance curve is based on values of K computed from the geometry of the measurement cross-section. The slope curve is defined to a gauge height of 9 m by discharge measurements (circled points) and extrapolated as the solid line to the peak gauge height of 20 m. It appears highly unlikely that the slope curve at a gauge height of 20 m will fall outside the limiting dashed curves shown in Figure II.1.12. In other words, it is highly unlikely that the value of S at 20 m (0.00095) is in error by more than ± 10 per cent. If that is true, when the square root of S is computed and then used in a computation of peak discharge, the error for both $S^{1/2}$ and Q reduces to ± 5 per cent. One can place

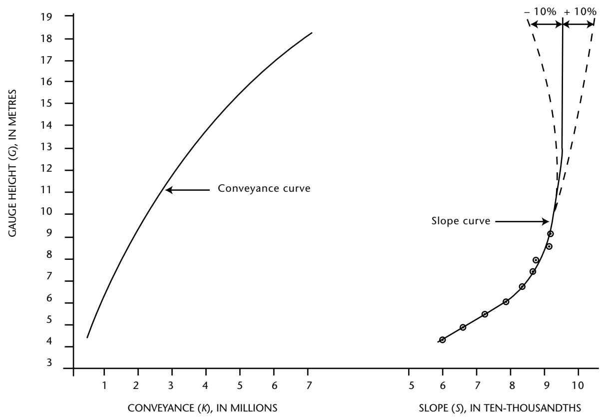


Figure II.1.12. High-flow extrapolation by use of conveyance-slope method, Klamath River at Somes Bar, California, United States

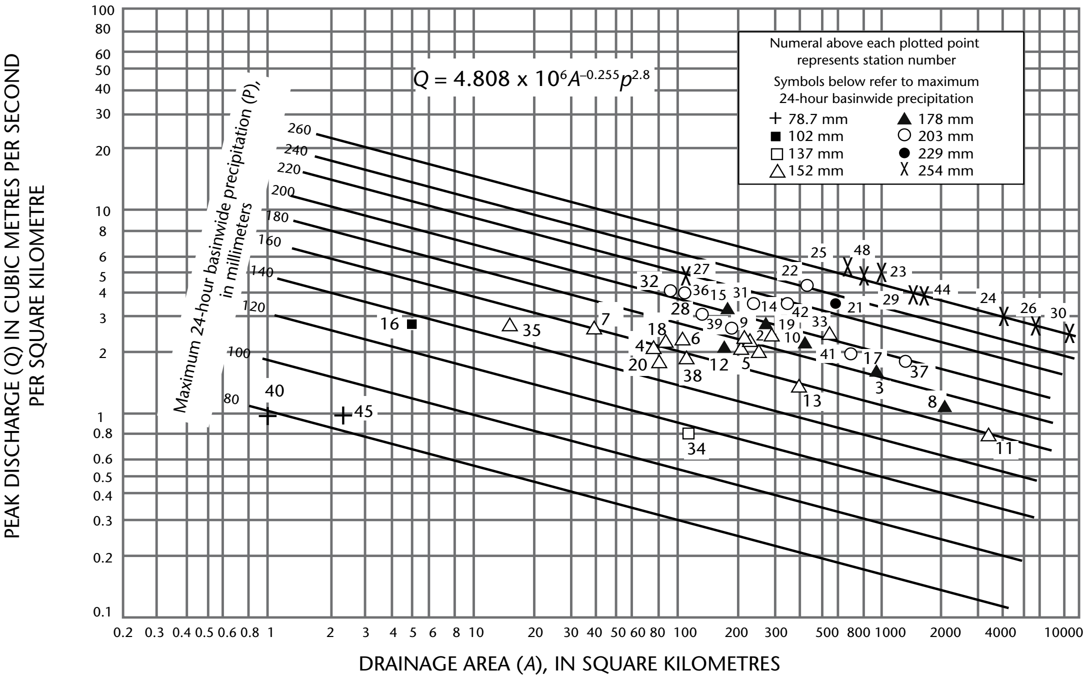


Figure II.1.13. Relation of peak discharge to drainage area and maximum 24-hour basin-wide precipitation in north coastal California, United States, December 1964

considerable confidence in the discharge computed for a gauge height of 20 m in this example. It should be mentioned here that the likelihood of a decrease in slope at high stages, as shown by the dashed curve on the left of the slope curve, is greatest when overbank flows occurs.

In the above example, conditions were ideal for application of the conveyance-slope method and may be misleading with regard to the general accuracy of the method. The conveyance-slope method assumes first that the geometry of the cross-section used for discharge measurements is fairly representative of that of a long reach of downstream channel. The need to meet this assumption immediately eliminates from consideration those gauging stations where discharge measurements are made at constricted cross sections, such as occur at many bridges and cableways.

The conveyance-slope method also assumes that slope tends to become constant at the higher stages. That is strictly true only for long, straight channels of uniform cross section but natural channels that meet that description are virtually nonexistent. Consequently, the slope-stage relation may be anything but a vertical line at the upper stages. In summary, the conveyance-slope method is a helpful adjunct in extrapolating rating curves but its limitations must be understood so that it is not misused.

Areal comparison of peak runoff rates

When flood stages are produced over a large area by an intense general storm, the peak discharges can often be estimated at gauging stations where they are lacking from the known peak discharges at surrounding stations. Usually each known peak discharge is converted to peak discharge per unit of drainage area before making the analysis. In other words, peak discharge is expressed in terms of cubic metres per second per square kilometre.

If there has been relatively little difference in storm intensity over the area affected, peak discharge per unit area may be correlated with drainage area alone. If storm intensity has been variable, as in mountainous terrain, the correlation will require the use of some index of storm intensity as a third variable. Figure II.1.13 illustrates a multiple correlation of that type where the independent variables used were drainage area and maximum 24-hour basin-wide precipitation during the storm of December 1964 in north coastal California, United States.

The peak discharges estimated by the above method should be used only as a guide in extrapolating the rating curve at a gauging station. The basic principles underlying the extrapolation of logarithmic rating curves are not to be violated to accommodate peak discharge values that are relatively gross estimates, but the estimated discharges should be given proper consideration in the extrapolation process.

Step backwater method

The step backwater method is a technique in which water surface profiles for selected discharges are computed by successive approximations, as described by Davidian (1984). Computer programs such as HEC-2 or WSPRO by Sherman (1990) are available for making the step backwater computations. Although the computations can be made by hand it is advisable to use a standard computer program designed for this purpose. The computations are very detailed and tedious, involving trial-and-error methods that can be performed quickly and efficiently by computer. This section will describe the data requirements and general procedure for evaluating the results.

The computations start at a cross-section where the stage-discharge relation is known or assumed and they proceed to the study site, which is the gauge site whose rating requires definition or extrapolation. If flow is in the sub-critical regime, as it usually is in natural streams, the computations must proceed in the upstream direction. Computations proceed in the downstream direction where flow is in the supercritical regime. In the discussion that follows, the usual situation of sub-critical flow will be assumed. Water surface profile computations are based on equation 1.2 (Manning equation) or equation 1.3 (Chezy equation). Irregular cross-section shape, roughness coefficients that vary laterally and vertically and velocity head adjustments can be properly accounted for in the computer programs.

Under conditions of sub-critical flow, water surface profiles converge upstream to a common profile. For example, the stage for a given discharge at a gated dam may have a wide range of values depending on the position of the gates. At a study site far enough upstream to be out of the influence of the dam, the stage for that discharge will be unaffected by the gate operations. Consequently, when the water surface profile is computed for a given discharge in the reach between the dam and the study site, the segment of the computed profile in the vicinity of the study site will be unaffected by the value of stage that exists at the dam. However, it will be necessary that the computations start at the dam and proceed upstream, subreach by subreach (in steps). It follows, therefore, that if an initial cross-section for the computation of the

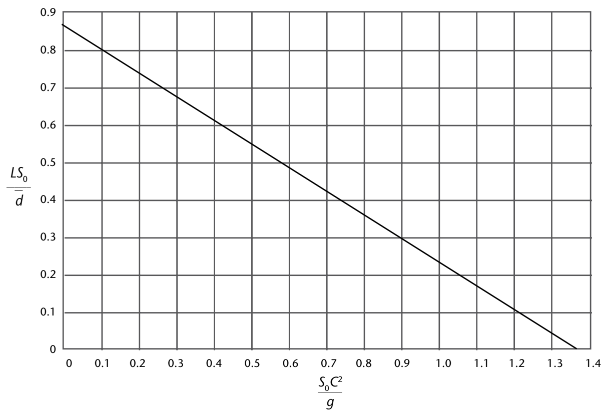


Figure II.1.14. Dimensionless relation for determining distance required for backwater profiles to converge

water surface profile is selected far enough downstream from the study site, the computed water surface elevation at the study site, corresponding to any given discharge, will have a single value regardless of the stage selected for the initial site.

A guide for determining the required distance, L, between the study site and the initial section is found in the dimensionless graph in Figure II.1.14. The graph defined by Bailey and Ray (1966), has for its equation:

$$\frac{LS_0}{\bar{d}} = 0.86 - 0.64\,\frac{S_0C_2}{g} \qquad (1.13)$$

where L = the distance required for convergence; S_0 = bed slope; d = mean depth for the smallest discharge to be considered; g = acceleration of gravity, and C = the Chezy coefficient.

If a rated cross-section is available downstream from the study site, that cross-section would be used as the initial section and there would be no need to be concerned with the above computation of L. As a general rule, it can be said that the steeper the water surface slope the shorter the reach length required for profile convergence. Conversely, very flat sloped streams require relatively long reaches of channel to obtain water surface profile convergence.

After the initial site is selected the next step is to divide the study reach, that is, the reach between the initial section and the study site, into sub-reaches. That is done by selecting cross-sections where major breaks in the high water profile would be expected to occur because of changes in channel geometry or roughness. Those cross-sections are the end sections of the sub-reaches. The cross-sections are surveyed to a common datum and roughness coefficients are selected for each sub-reach. That completes the field work for the study.

The first step in the computations is to select a discharge, *Q*, for study, and obtain a stage at the initial section for use with that value of discharge. If the initial section is a rated cross-section, that stage will be known. If the initial section is not a rated cross section, an estimated stage can be computer using the Chezy or Manning equations using the cross section properties and an estimated water surface slope. A technique such as the conveyance slope method can be used, as described in a previous section of this Manual.

Step backwater computations begin at the subreach farthest downstream using a known or estimated stage at the most downstream cross section for the value of discharge being studied. The stage at the upstream end of the subreach is computed by balancing the energy equation between the two cross sections at each end of the subreach. The computations proceed from subreach to subreach until the uppermost cross section is reached. When doing these computations by hand it is a trial-and-error procedure. Computer programs perform these computations quickly and virtually eliminate the possibility of mathematical errors which are common in step backwater computations performed by hand. Profiles for several different discharges and starting elevations can be computed quickly and easily to analyze several points on a rating curve.

If the stage corresponding to the study discharge at the initial (downstream) cross section was known, the stage computed for the upstream study site is satisfactory. If the stage at the initial cross section was estimated it is necessary to repeat the computations using other values of stage at the initial cross section for the same discharge. This is done to assure convergence of the water surface profiles at the study site. Most computer programs, such as WSPRO by Shearman (1990), will allow entry of several starting elevations for a given discharge, thus providing several profile computations simultaneously. All sets of computations for a given discharge should result in almost identical values of stage at the study site. If they do not, the reach should be extended in the downstream direction, which will involve inclusion of one or more new cross sections. All computations previously described must be repeated for the longer reach. The entire procedure can be repeated for other discharges until enough data are obtained to define the high water rating for the study site.

The step backwater method can be used to prepare a preliminary rating for a gauging station before a single discharge measurement is made. A smooth curve is fitted to the logarithmic plot of the discharge values that are studied. The preliminary rating can be revised, as necessary, when subsequent discharge measurements indicate the need for such revision. If the step backwater method is used to define the high water end of an existing rating curve, the discharge values investigated should include one or more of the highest discharges previously measured. By doing so, selected roughness coefficients can be verified, or can be modified so that step backwater computations for the measured discharges provide stages at the study site that are in agreement with those observed. The computations for the high water end of the rating can then be made with more confidence, in the knowledge that reasonable values of the roughness coefficients are being used. There will also be assurance of continuity between the defined lower part of the rating and the computed upper part.

Flood routing

Flood-routing techniques may be used to test and improve the overall consistency of records of discharge during major floods in a river basin. The number of direct observations of discharge during such flood periods is generally limited by the short duration of the flood and the inaccessibility of certain stream sites. Through the use of flood-routing techniques, all observations of discharge and other hydrological events in a river basin may be combined and used to evaluate the discharge hydrograph at a single site. The resulting discharge hydrograph can then be used with the stage hydrograph for that gauge site to construct the stage-discharge relation for the site; or, if only a peak stage is available at the site, the peak stage may be used with the peak discharge computed for the hydrograph to provide the end point for a rating curve extrapolation.

Flood-routing techniques, of which there are many, are based on the principle of the conservation of mass, where inflow plus or minus change in storage equals outflow. It is beyond the scope of a stream gauging manual to treat the subject of flood routing. It is discussed in most standard hydrology text books such as Linsley, Kohler, and Paulhus, 1949, and Chow, 1964.

1.12 SHIFTS IN THE DISCHARGE RATING

Shifts in the discharge rating reflect the fact that stage-discharge relations are not permanent but vary from time to time, either gradually or abruptly, because of changes in the physical features that form the control for the station. If a specific change in the rating stabilizes to the extent of lasting for more than a month or two, a new rating curve is usually prepared for the period of time during which the new stage-discharge relation is effective. If the effective period of a specific rating change is of shorter duration, the original rating curve is usually kept in effect, but during that period shifts or adjustments are applied to the recorded stage, so that the new discharge corresponding to a recorded stage is equal to the discharge from the original rating that corresponds to the adjusted stage. For example, assume that vegetal growth on the control has shifted the rating curve to the left (minus shift), so that in a particular range of discharge, stages are 0.015 m higher than they originally had been. To obtain the discharge corresponding to a recorded stage of, say, 0.396 m the original rating is entered with a stage of 0.381 m (0.396 – 0.015) and the corresponding discharge is read. The period of time during which such stage adjustments are used is known as a period of shifting control.

Frequent discharge measurements should be made during a period of shifting control to define the stage-discharge relation, or magnitude(s) of shifts, during that period. However, even with infrequent discharge measurements the stage-discharge relation can be estimated during the period of shifting control if the few available measurements are supplemented with knowledge of shifting control behaviour. This section of the Manual discusses such behaviour.

That part of the discussion that deals with channel control shifts does not include alluvial channels, such as sand channels, whose boundaries change almost continuously. Sand channels are discussed in a subsequent section. Likewise, the formation of ice in the stream and on section controls causes shifts in the discharge rating. Ice is discussed separately in a subsequent section.

1.12.1 Detection of shifts in the rating

Stage-discharge relations are usually subject to minor random fluctuations resulting from the dynamic force of moving water and from the collection of debris or aquatic vegetation on section controls. Because it is virtually impossible to sort out those minor fluctuations, a rating curve that averages the measured discharges within close limits is considered adequate. Furthermore, it is recognized that discharge measurements are not error free, and consequently an average curve drawn to fit a group of measurements is probably more accurate than any single measurement that is used to define the average curve. If a group of consecutive measurements subsequently plot to the right or left of the average rating curve it is usually clearly evident that a shift in the rating has occurred. An exception to that statement occurs where the rating curve is poorly defined or undefined in the range of discharge covered by the subsequent measurements. In that circumstance the indication is that the original rating curve was in error and requires revision. If, however, only one or two measurements depart significantly from a defined segment of the rating curve, there may be no unanimity of opinion on whether a shift in the rating has actually occurred, or whether the departure of the measurement(s) results from random error that is to be expected occasionally from measurements.

Two schools of thought exist with regard to identifying periods of shifting control. In some countries, notably the United States, a pragmatic approach is taken that is based on certain guidelines and on the judgment of the analyst. In other countries, notably the United Kingdom of Great Britain and Northern Ireland, the approach used is based on statistical theory. It is reiterated that the discussion that follows excludes the constantly shifting alluvial channels that are discussed in a subsequent section.

In the United States, if the random departure of a discharge measurement from a defined segment of the rating curve is within ± 5 per cent of the discharge value indicated by the rating, the measurement is considered to be a verification of the rating curve. If several consecutive measurements meet the 5 per cent criterion, but if they all plot on the same side of the defined segment of the rating curve, they may be considered to define a period of shifting control. It should be mentioned that when a discharge measurement is made, the measurement is computed before the hydrologist leaves the gauging station and the result is plotted on a rating curve that shows all previous discharge measurements. If the discharge measurement does not check a defined segment of the rating curve by 5 per cent or less, or if the discharge measurement does not check the trend of departures shown by recent measurements, the hydrologist is expected to repeat the discharge measurement. In other words, make a check measurement.

In making a check measurement, the possibility of systematic error is eliminated by changing the measurement conditions as much as possible. The meter and stopwatch are changed, or the stopwatch is checked against the movement of the second hand of a standard watch. If an Electronic Field Notebook (EFN) is used for counting and timing meter revolutions then manual checks should be made to insure that the EFN is functioning as required. If the measurements are being made from a bridge, boat or cableway, the measurement verticals are changed by measuring at verticals between those originally used; if wading measurements are being made, a new measurement section is sought, or the measurement verticals in the original section are changed. If the check measurement checks the original rating curve or current rating trend by 5 per cent or less, the original discharge measurement will be given no consideration in the rating, although it is still entered in the records. If the check measurement checks the original discharge or the trend of that measurement if the stage has changed, by 5 per cent or less, the two measurements are considered to be reliable evidence of a new shift in the stage-discharge relation. If the check measurement fails to check anything that has gone before, a second check measurement is made and the most consistent two of the three measurements are used for rating analysis. The need for a second check measurement is a rarity, but may possibly occur.

Thus, in the United States, a single discharge measurement and its check measurement, even if unsupported by later measurements, may mark a period of shifting control. The engineer who analyses the rating does have the responsibility of explaining the reason for the short-lived shift. It can often be explained as having started as a result of fill (or scour) on a preceding stream rise and as having ended as a result of scour (or fill) on the recession or on a following rise.

In the United Kingdom, the analysis of the rating starts in the usual way. The chronologically numbered discharge measurements are plotted on logarithmic graph paper and are fitted by eye with a smooth curve and the rating equations established by computer. Where compound controls exist there may be one or more points of inflection in the curve. In the statistical analysis that follows, each segment of the rating curve between inflection points is treated separately.

1.12.2 Statistical analysis of the stage-discharge relation

The stage discharge relation, being a line of best fit, should be more accurate than any of the individual discharge measurements. The equation of the relation may be computed by the method of least squares, or regression analysis, which assumes that the relation plots as a straight line on logarithmic paper. Computer programs are readily available for performing these computations.

In the United States the above statistical approach is not favored for several reasons. First, it is felt that the limiting criteria of 2 S_e per cent will usually exceed the 5 per cent criteria preferred in the United States. Second, any statistical approach gives equal weight to all discharge measurements used in the analysis. In theUnited States, hydrologists rate the probable accuracy of the measurements they make on the basis of measuring conditions at the time, without reference to how closely the measurements plot on the rating curve. The feeling in the United States is that more weight in the analysis should be given to measurements rated good to excellent than to measurements rated fair to poor. Third, while it is agreed that in general an average curve drawn to fit a group of measurements is probably more accurate than any single measurement that is used to define the average curve, it is also felt in the United States that any subsequent measurement that is verified by a check measurement is more accurate than the rating curve value of discharge, particularly at a station that is historically known to have rating curve shifts.

1.12.3 Rating shifts for artificial controls

Weirs

Artificial controls are not subject to scour and fill by high flows, but the stream-bed immediately upstream from the weir may be so affected. If scour occurs in the pool formed by the weir, the pool is deepened and velocity of approach decreases. The net result is a smaller discharge for a given stage than under pre-scour conditions. That is, the rating curve for the period of scour will shift to the left of the rating curve for pre-scour conditions. The converse occurs if the weir pool has been subjected to deposition or fill.

The effect of such scour and fill on the stage-discharge relation is usually relatively minor and usually can be expressed by a parallel shift of most of the section control portion of the rating curve that is plotted as a straight line on logarithmic graph paper. If only a single discharge measurement is available for defining the parallel shift curve, the shift curve is drawn to pass through that measurement. If more than one discharge measurement is available, and there is no evidence of a progressive rating shift with time, the parallel shift curve is drawn to average the discharge measurements. If the discharge measurements indicate a progressive rating shift with time, shifts are prorated with time. However, what may appear to be a gradually progressive shift may in fact be several discrete shifts caused by individual peak flows whose occurrences are not widely separated in time. The shift in stage to be applied to recorded gauge heights during the period of shifting control is determined from the vertical spacing between the original rating curve and the shift curve.

The shift, if attributable to fill, is considered to start after the peak discharge of a stream rise that preceded the first of the variant discharge measurements. Shift adjustments are therefore started on the recession of that rise. The shift, if attributable to scour, is considered to start during the high stages of a stream rise that preceded the first of the variant discharge measurements. Because those high stages generally occur when the section control is drowned out by channel control, the shift in the section control segment of the rating is again commonly first applied after the peak discharge of the rise. The shifts are ended on a stream rise that follows the last variant discharge measurement, using the general principle that scour in the gauge pool usually occurs during high stages and fill usually occurs during the recession of a stream rise.

The parallel shift discussed in a preceding paragraph requires some elaboration. A parallel shift of the rating curve on logarithmic graph paper indicates that for all stages the discharge changes by a fixed percentage and that the difference in stage between the two lines increases with stage. However, it is not quite true that the discharge changes by a fixed percentage when the weir pool has scour or fill. At extremely low flows there will be no effect because velocity of approach is negligible; that section of the original rating has a break in slope and the lower end of the parallel shift curve above the break in slope should be smoothly merged with the extreme low water curve. The effect of scour or fill on the percentage change in discharge increases rapidly with stage to a maximum value and then slowly decreases to a per cent change that does not differ greatly from the maximum percentage. The parallel shift drawn through the available discharge measurement(s) will adequately fit those relatively large percentage changes in discharge at the higher stages. The merged section of the shift curve at the lower stages will adequately fit the rapidly increasing percentage change in discharge at those lower stages. Figure II.1.15 illustrates the above discussion.

It has been mentioned frequently in this manual that section controls are usually submerged at high stages as a result of channel control becoming effective. The parallel shift curve described above should be extended to the stage where it either intersects the actual rating for channel control (in the case of scour in the weir pool) or can be smoothly merged into the rating for channel control (in the case of fill in the weir pool). If a shift has occurred simultaneously in the channel control, the shift curves for the section control and channel control segments of the rating are drawn to form a continuous curve.

The preceding paragraphs discuss changes in the velocity of approach that are caused only by scour and fill in the weir pool. The velocity of approach may also be affected by aquatic vegetation growing in the weir pool. Usually such an occurrence will reduce the velocity of approach by greatly increasing the friction loss, and the rating curve will shift to the left. However, the shift will not be abrupt, but will gradually increase as the growing season progresses. The aquatic growth in the pool may also encroach on the weir to the extent that the effective length, b, of the weir is reduced. The effect of a reduction in effective length of the weir is a parallel shift of the rating to the left when plotted on logarithmic graph paper. At all stages the discharge will be reduced by a percentage that is equal to the percentage change in effective length of the weir. The shift will either decrease gradually as the vegetation dies in the dormant season, or the shift may terminate abruptly if the vegetation is washed out by a stream rise.

Aquatic vegetation may sometimes attach itself to a weir crest and thereby reduce the effective head on the weir for any given gauge height, h. The effective head will be reduced by a constant value that is equal to the thickness of the growth. In other words, the effective head, $(h - e)$, is reduced because the value of e is increased by the thickness of the growth. The reduction in effective head causes the rating to shift to the left, it being displaced vertically by an amount equal to the thickness of the growth. It the shift rating is plotted on arithmetic coordinate

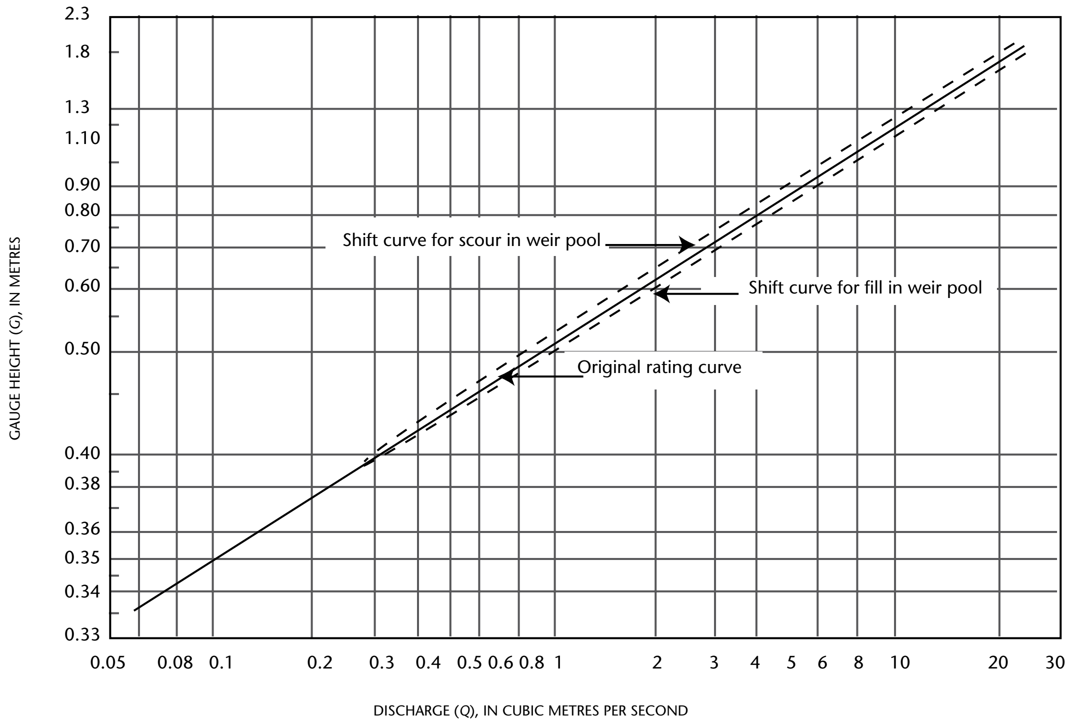


Figure II.1.15. Rating curve for hypothetical rectangular thin-plate weir with shift curves for scour and fill in the weir pool

graph paper it will be parallel to the original rating. If the shift rating is plotted on logarithmic graph paper, it will be a curve that is concave upward and asymptotic to the original linear rating curve at the higher stages. The aquatic vegetation on the weir should be removed with a wire brush before it becomes heavy enough to affect the stage-discharge relation. The effect of the shift caused by the algae growth disappears during stages when channel control becomes effective.

Flumes

Shifts in the stage-discharge relation for flumes are most commonly caused by changes in the approach section, either in the channel immediately upstream from the flume or in the contracting section of the flume upstream from the throat. In either event the change is caused by the deposition of rocks and cobbles that are too large to pass through the flume. The flume is self cleaning with regard to sediment of smaller size. Manual removal of the large debris should restore the original discharge rating of the flume.

The deposition of rocks and debris upstream from the flume may divert most of the flow to the gauge side of the flume and the build up of water at the gauge will result in a shift of the discharge rating to the left. Conversely, if most of the flow is diverted to the side of the flume opposite the gauge, the discharge rating will shift to the right. If rocks and cobbles are deposited at the entrance to the throat of the flume, they will cause the discharge rating to shift to the left, because the stage at the gauge will be raised higher than normal for any given discharge. A similar backwater effect will result from the growth of aquatic vegetation at the entrance to the throat.

The backwater effect, or decrease in head for a given gauge height, caused by deposition or algal growth at the entrance to the throat of the flume, has the effect of increasing the value of e in a linear logarithmic plot of the rating. The shifted rating on logarithmic graph paper will be a curve that is concave upward and asymptotic to the original linear rating curve at the higher stages. The deposition or rocks and debris will usually be associated with a high water event, whereas the growth of aquatic vegetation will increase gradually with time. It is therefore essential that measuring structures be well maintained and kept free of debris and accretion.

1.12.4 **Rating shifts for natural section controls**

The primary cause of changes in natural section controls is the high velocity associated with high discharge. Of those controls, a rock ledge outcrop will be unaffected by high velocities, but boulder, gravel and sand bar riffles are likely to shift, boulder riffles being the most resistant to movement and sand bars the least resistant. After a flood the riffles are often altered so drastically as to bear no resemblance to their pre-flood state, and a new stage-discharge relation must be defined. Minor stream rises usually move and sort the materials composing the riffle, and from the standpoint of the rating curve the greatest effect is usually a change in the gauge height of effective zero flow, *e*. The shift curve ideally should be defined by current meter discharge measurements. However, if only one or two measurements are available for the purpose they are examined and the gauge height shift that they indicate is applied to the section control segment of the original rating curve. If the shift rating is plotted on arithmetic paper it will tend to be parallel to the original rating. The extreme low water end of the curve can be extrapolated to the actual gauge height of zero flow, as determined in the field when low water discharge measurements are made. If the shift rating is plotted on logarithmic graph paper, it will be a curve that is either concave upward or downward, depending on whether the shift is to the left (increase in *e*) or the right (decrease in *e*). The shift curve will tend to be asymptotic to the linear rating at the higher stages of section control, but its precise slope in the range of stage where channel control is beginning to exert an effect will depend on whether or not a shift has occurred in the channel control segment of the rating curve.

Vegetal growth in the approach channel of the control or on the control itself will affect the stage-discharge relation. Aquatic vegetation in the approach channel will affect the velocity of approach, and if the channel growth encroaches on the control it may reduce the effective length of the control. Aquatic growth on the control itself will reduce the discharge corresponding to any given stage by reducing the effective head on the weir and increasing the resistance of flow and/or by reducing the effective length of the control. The shifts associated with vegetal growth are cyclic and therefore change with time. The growth increases as the growing season progresses and declines during the dormant season, but shifts may terminate abruptly if the vegetation is washed out by a stream rise.

In temperate climates, accumulations of water-logged fallen leaves on section controls each autumn clog the interstices and raise the effective elevation of all section controls. The effect of an increase in the gauge height of effective zero flow, *e*, has already been explained. The build up of water-logged leaves is progressive starting with the first killing frost (usually in October in the northern hemisphere) and reaching a maximum when the trees are bare of leaves. The first ensuing stream rise of any significance usually clears the control of fallen leaves.

Two other causes of backwater (increased gauge height for a given discharge), unassociated with hydrological events, also warrant discussion. Holiday-makers in the summer often use the gauge pool for swimming and they will often pile rocks on the control to create a deeper pool. This change in the height of the control manifests itself in the record of stage as an abrupt increase in gauge height, usually during a rainless period, without any corresponding decline in stage that would be associated with the passage of a stream rise. The abrupt rise in stage fixes the time when the shift in the rating occurred. The magnitude of the change in stage is a measure of the change in the value of *e*. In some regions of the world another cause of backwater is the construction of dams by beavers. These dams are built of boughs, logs, stones and mud to create a pool that is part of the beaver's habitat. Again, the time of occurrence and the effect on the stage of the stream can be detected in the gauge height record which will show a gradual rise, usually over a period of a few days as the dam is being built, without the corresponding decline in stage that would be associated with a stream rise. The beaver dams usually remain in place until washed out by a high discharge.

A recent technique for documenting changes of the control is the use of inexpensive digital cameras. A digital camera can be mounted in a protective enclosure with a clear glass or lexan window, and focused on the control section of the stream. The camera is programmed to automatically take a photograph periodically, for example every hour. These photos are very helpful in evaluating shift application, particularly during periods of leaf shifts, debris accumulation, ice effects or when someone piles rocks on the control. Web cams are also being used in some places to allow the hydrographer an opportunity to remotely review conditions of the control on a daily or more frequent basis.

1.12.5 **Rating shifts for channel control**

As mentioned earlier, most natural streams have compound controls, that is, a section control for low stages and channel control for high stages. The shifts in section control that were described on the preceding pages are commonly accompanied by shifts in channel control.

The most common cause of a shifting channel control, in a relatively stable channel, is scour or fill of the stream-bed caused by high velocity flow. The scour usually occurs during a stream rise and fill usually occurs on the recession, but that statement is an over simplification of the highly complex process of sediment transport. The degree of scour in a reach is dependent not only on the magnitude of the discharge and velocity, but also on the sediment load coming into the reach. On some streams it has been found that when scour is occurring in a pool at a meander bend there is simultaneous filling on the bar or riffle at the crossover, or point of inflection between successive meander bends. On other streams scour has been found to take place simultaneously through relatively long reaches of channel, both on pools and over bars. A further complication is the fact that the length of channel that is effective as a control is not constant, but increases with discharge.

From the preceding discussion it should be apparent that there is no really satisfactory substitute for discharge measurements in defining shifts in the channel control segment of the rating. Of particular importance are measurements made at or near the peak stage that occurs during periods of shifting control. However, in the usual situation a few (or possibly only one or two) measurements made at medium stages are the only ones available for analyzing channel control shifts, and the shifts must be extrapolated to peak stages. However, if it is known that the peak stage results in significant over bank flows, it is probable that a break in the rating slope will occur and that therefore the extrapolation is more likely to be in error – hence increasing the importance of measuring the peak discharge. The assumptions usually made in the station analysis are those discussed below. The results are accepted unless they are shown to be invalid by a determination of peak discharge as described in Volume I, Chapter 9, or are shown to be invalid by use of one or more of the methods of rating curve extrapolation described in previous sections of this Volume.

If a single predominantly large stream rise occurred shortly before the first measurement that indicated a shift, the shifts are assumed to have been caused solely by that rise. If more than one large stream rise occurred shortly before the first shift measurement, the shift curve may be prorated

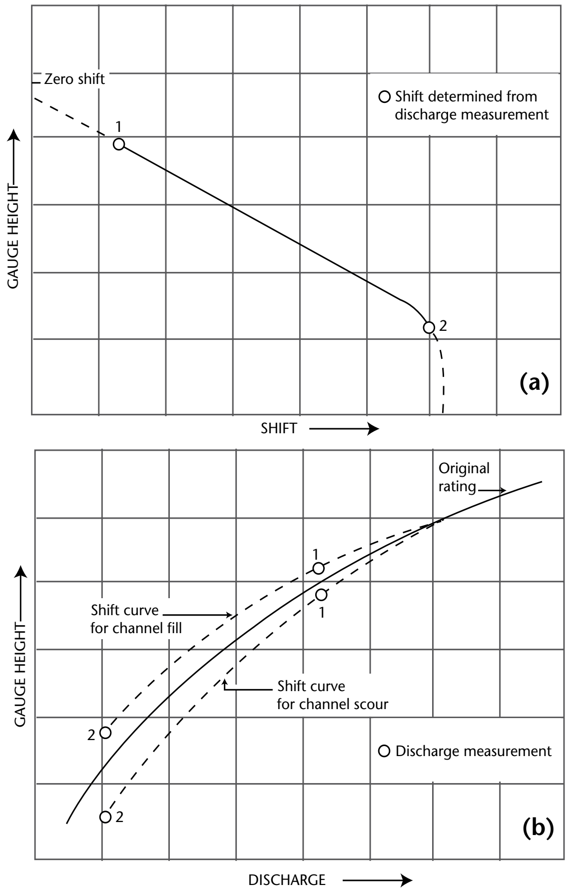


Figure II.1.16. First example of a stage-shift relation and the corresponding stage-discharge relation caused by scour or fill in the control channel

between rises. For example, if two rises of almost equal magnitude occurred just before the first shift measurement, and if the shift curve indicates a shift of 0.090 m at a given stage, the shift to be used during the period between the two rises would be 0.045 m at the given stage. It is often helpful to plot the shifts indicated by the discharge measurements against the observed stage of those measurements to obtain the trend of the shifts.

The pattern of scour and fill in the control channel determines whether the shift will increase with stage, decrease with stage, or be relatively constant at all stages. Figure II.1.16, graph (a), illustrates a common situation where the shifts, either plus in the case of scour or minus in the case of fill, increase as stage decreases. The highest value of the shift is assumed to be only slightly greater than the maximum value observed in order to avoid overcorrecting the original rating. Graph (b) of Figure II.1.16 shows the shifted ratings corresponding to measurements No. 1 and 2. The ratings have been plotted on arithmetic coordinate graph paper because the shifts are more easily visualized, at least by the inexperienced hydrologist, on that type of plotting paper. The stage shift curve is usually plotted on arithmetic coordinate paper, but the rating curves are usually plotted on logarithmic graph paper. On logarithmic paper the shift curves in this example would converge more rapidly toward the original rating curve at high stages. The shift curves at low stages would be shaped to merge smoothly with the shift curve for section control. The period for applying the shifts would be terminated on the stream rise following the last shift measurement; the original rating would be used on the recession from that rise.

In analyzing shifts there is no substitute for experience with a given stream because the shift pattern can often be interpreted logically in more than one way. For example, refer to the shift curve for channel fill in graph (b) of Figure II.1.16. Assume that measurements No. 1 and 2 were made on a stream recession and that measurement No. 1 was made a few days before measurement No. 2. Measurement No. 2 shows the effect of greater fill than measurements No. 1. Fill usually occurs on a recession, therefore it is possible that the shifts should have been made to vary with time or to vary with time and stage, rather than with stage alone as shown in Figure II.1.16. In the absence of additional knowledge the simplest interpretation is generally made, as was done here. Given more discharge measurements or a better knowledge of the behaviour of the particular stream, a more accurate analysis can be made.

Figure II.1.17, graph (a), illustrates a less common situation where the shifts increase as stage increases. Again the highest value of shift is assumed to be only slightly greater than the maximum value observed in order to prevent overcorrecting the original rating. Graph (b) of Figure II.1.17 shows the shift ratings corresponding to measurements No. 1 and 2. The period for applying shifting control corrections would be terminated on the stream rise following the last shift measurement; the original rating would be used on the rising limb of that rise. As in the case of Figure II.1.16, in the absence of additional knowledge more than one interpretation can be given to shifts shown by measurements No. 1 and 2, depending on the relative times when the measurements were made and the fact that scour generally occurs on stream rises and fill generally occurs on stream recessions.

If there had been an additional major rise, one that occurred between the pairs of measurements shown

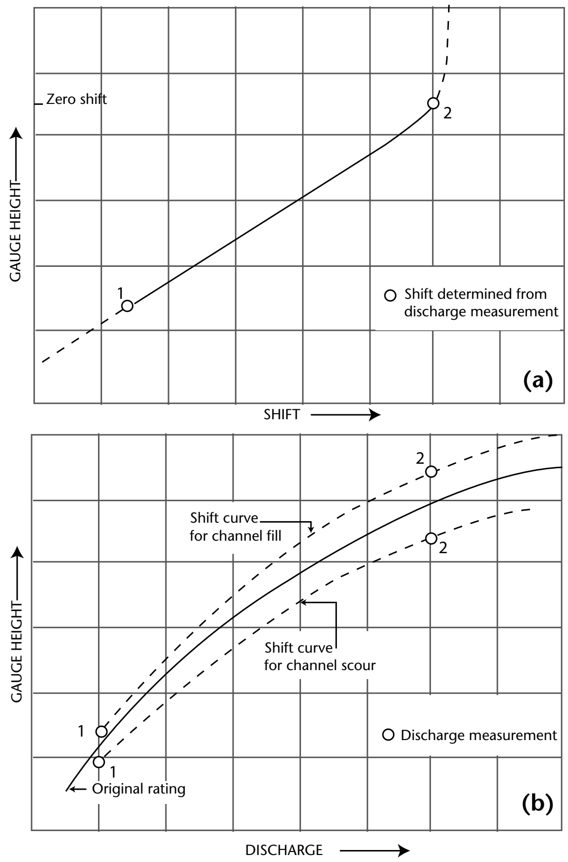


Figure II.1.17. Second example of a stage-shift relation and the corresponding stage-discharges relation caused by scour or fill in the control channel

in Figures II.1.16 and II.1.17, other courses of action would be available. If the analyst had no additional data on which to base a judgment, he or she could assume that two separate shift events occurred, each attributable to the rise that preceded a discharge measurement. For each shift period he or she could use a constant shift, equal to that shown by the discharge measurement made during that shift period. If, however, the analyst has had experience in the past with shifting control at the station caused by scour and fill in the control channel and if that experience had shown that shifts tend to vary with stage, another course of action would suggest itself. For each of the stage periods the analyst could use a stage shift relation of average shape that passed through the shift value shown by the appropriate discharge measurement. The above discussion would also apply to the situation of a single shift period and the availability of only a single discharge measurement made during that period. It is assumed that the single discharge measurement would include a check measurement to verify its accuracy, as discussed previously.

If during a period of shifting control several measurements had been made, but few of them could be fitted with a smooth shift curve, it would then be necessary to prorate the shifts with both time and stage, or possibly with time alone, based on the average shape of a stage shift relation.

As mentioned earlier, scour in the control channel causes a plus shift because depth, and therefore discharge, is increased for a given gauge height. Deposition or fill in the approach channel causes a minus shift, because depth, and therefore discharge, is decreased for a given gauge height. Thus the effect on the discharge of scour or fill in a channel control is opposite to that of scour and fill in a weir pool, which affects only the velocity of approach. Therefore, where a permanent weir is part of a compound control, scour in both the weir pool and in the channel control will cause a minus shift in the rating for section control and a plus shift in the rating for channel control. The converse is true when fill occurs in both the weir pool and the channel control. That situation is compatible with the stage shift relation shown in Figure II.1.17, where a further decrease in stage would change the sign of the shifts. Where the section control is a natural riffle, that riffle is likely to scour when the channel scours and fill when the channel fills, a situation that is compatible with the stage shift reaction shown in Figure II.1.16. In any event, the shift curves for low stages of channel control should be shaped to join smoothly with the shift curves for high ages of section control, where a compound control exists.

Up to now the discussion of channel control shifts has been confined to shifts caused by stream-bed scour and deposition. Shifts may also be caused by changes in the width of the channel. Even in a relatively stable channel the width of the channel may be increased during intense floods by widespread bank cutting, and in some areas (for example, north coastal California, United States) channel widths may be constricted by widespread landslides that occur when steep stream-banks are undercut. In meandering streams changes in channel width occur as point bars are built up by deposition and later eroded by flood flows. The effect of a change in channel width on the stage discharge relation, unaccompanied by a change in stream-bed elevation, is to change the discharge, for a given gauge height, by a fixed percentage. When the original rating curve for channel control is plotted linearly on logarithmic graph paper, in accordance with equation 1.4 the value of C increases with an increase in width and decreases with a decrease in width. The shift curve for a change in width alone will therefore plot on logarithmic paper as a straight line that is parallel to the original linear rating curve. Under those conditions a single discharge measurement is sufficient for constructing a shift curve for channel control.

When a change in channel width occurs concurrently with a change in stream-bed elevation the effects of the two changes are compounded. The resulting shift curve is complex and requires at least several discharge measurements for its definition. The growth of vegetation in a stream channel will affect the stage discharge relation by reducing the discharge for a given gauge height. The shift rating will therefore plot to the left (minus shift) of the original rating. The vegetation will increase the roughness coefficient of the channel and will tend to constrict the effective or unobstructed width of the channel. Both of those factors reduce the value of C in equation 1.4, and if the changes in roughness coefficient and effective width are unvarying with stage, the shift curve will be parallel to, and to the left of, the original rating curve that has been plotted linearly on logarithmic graph paper. However, the changes are not usually independent of stage. If the growth consists of aquatic weeds, the weeds will be overtopped and bent over by high water. If the growth consists of alders and willows, the backwater effect will be greater at higher stages when the tree crowns as well as when the tree trunks are submerged. The rating shift caused by channel vegetation is, of course, variable with time as the aquatic vegetation spreads and increases in size.

1.13 EFFECT OF ICE FORMATION ON DISCHARGE RATINGS

1.13.1 General

The formation of ice in stream channels or on section controls affects the stage- discharge relation by causing backwater that varies in effect with the quantity and nature of the ice, as well as with the discharge. Because of the variability of the backwater effect, discharge measurements should be made as frequently as is feasible when the stream is under ice cover, particularly during freeze up and break-up periods when flow is highly variable. Procedures for making measurements under ice cover are described in Volume I, Chapter 5.

In midwinter the frequency of measurements will depend on climate, accessibility, size of stream, winter runoff characteristics and required accuracy of the discharge record. As a general rule, two measurements per month is the recommended frequency. At stations below power plants that carry a variable load, it may be necessary to make two measurements during each winter visit, one at the high stage of the regulated flow and the other at the low stage. The backwater effects may be markedly different at the two stages. In very cold climates where winter ice cover persists and winter discharge shows a relatively smooth recession, fewer winter measurements are needed than in a climate that promotes the alternate freezing and thawing of river ice.

Knowledge of the three types of ice formation – frazil, anchor and surface ice – and their possible effects, is helpful in analyzing streamflow records for ice- affected periods. With regard to the type of stage gauge that is preferred for use at ice-affected stations, the graphic recorder (Volume I, Chapter 4) is by far the best, because the recorder graph generally provides dependable evidence of the presence and type of ice formation. However, many stations now use Electronic Data Loggers (EDL) or transmitters that do not directly produce a graph of the gauge height record. It is highly advisable, therefore, to use these records to plot a graph for periods when ice is probable, and to examine these graphs for evidence of ice formations.

1.13.2 **Frazil**

Frazil is ice in the form of fine elongated needles, thin sheets, or cubical crystals, formed at the surface of turbulent water, as at riffles. The turbulence prevents the ice crystals from coalescing to form sheet ice. The crystals may form in sufficient numbers to give the water a milky appearance. When the crystals float into slower water they come together to coalesce into masses of floating slush. When the current carries slush ice under a sheet of downstream surface ice, the slush may become attached to the underside of the surface ice, thereby increasing the effective depth of the surface ice. Most of the slush that adheres to the surface ice does so near the upstream end of the ice sheet. Frazil or floating slush has no effect on the stage-discharge relation but may interfere with the operation of a current meter. It is particularly troublesome to operators of hydroelectric plants. By adhering and building up on trash racks the ice may effectively reduce the flow to the turbines.

1.13.3 **Anchor ice**

Anchor ice is an accumulation of spongy ice or slush adhering to the rocks of a stream-bed. In former years the theory was held that the ice resulted from loss of heat by long-wave radiation from streambed to outer space, because anchor ice generally formed on clear cold nights on the streambeds of open reaches of river. This theory has been shown to be invalid because all of the long-wave radiation that can be lost from the bed of a stream at 0°C would be absorbed in less than 1 cm of water. Anchor ice is now commonly believed to be either:

(a) Frazil that turbulent currents have carried to the streambed where the ice adhered to the rocks; or
(b) The result of super-cooled water finding nucleating agents on the streambed on which to form as ice.

The ice first formed on the rocks acts as a nucleating agent for the continued growth of the ice mass.

Regardless of how anchor ice forms it cannot form or exist when the rocks are warmed by short-wave radiation from the sun which penetrates the water. When the morning sun strikes anchor ice that had formed the night before, and the streambed is warmed by the incoming solar radiation, the anchor ice is released and floats to the surface, often carrying small stones that it has picked up from the bed. For the next few hours the stream will be full of floating slush released in a similar manner upstream.

Anchor ice on the streambed or on the section control may build up the bed and/or control to the extent that a higher than normal stage results from a given discharge. The solid line graph in Figure II.1.18 shows a typical effect of anchor ice on a water stage recorder graph. The rise starts in late evening or early morning, many hours after

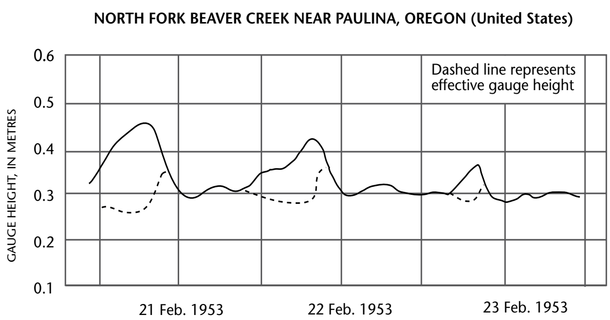


Figure II.1.18. Typical anchor-ice rises

the sun has set, when ice begins to adhere to the rocks and raise the water level. By 10 a.m. the sun has warmed the streambed sufficiently to release the ice and the stage starts to fall. The distinguishing feature of the anchor ice hump is that the rise is slow compared to the fall, whereas an actual increase in streamflow would occur in the opposite sequence, or at least the rise would be as rapid as the fall. The small rises in actual discharge in the late afternoon, shown by the short dashed lines in Figure II.1.18 probably result from water being released from channel storage when anchor ice upstream goes out. There may also be some runoff from the melting of snow and ice during the warmer part of the day.

1.13.4 Surface ice

As the name implies, surface ice forms on the surface, first as a fringe of shore ice, which then, if the stream is not too turbulent, spreads to form a continuous ice cover spanning the stream from bank to bank. A description of the formation of surface ice follows.

Formation of ice cover

With the onset of cold weather the water in a stream is gradually cooled. Along the banks where the water is quiescent, temperature stratification occurs, as in a lake. Because depths near the bank are usually very shallow, temperatures reach the freezing point more quickly there. Ice crystals form and adhere to the banks, twigs and projecting rocks, and a thin ice sheet forms. In the open part of the channel temperature stratification is generally absent because of turbulent mixing and the entire water body must reach 0°C before any freezing will occur. In the absence of nuclei or foreign material on which the ice crystals may form, there may be slight super-cooling of the surface layer before any ice crystals are produced.

The ice sheet builds out from the shore as super-cooled water, or water carrying ice crystals, impinges on the already formed shore ice, and the transported or newly formed ice crystals adhere to the sheet. In the centre of the stream, turbulence prevents coalescence of the ice crystals (frazil) that form. In the less turbulent areas, groups of crystals coalesce to form small pans of floating slush. These pans and/or individual ice crystals are carried by the currents until they too impinge and adhere to existing ice sheets. In this manner an ice sheet finally forms across the entire stream. The ensuing increase in thickness of the ice sheet occurs almost entirely at the interface of ice and water.

On a fairly wide stream there is no great build up of pressure as a result of the ice cover because the ice is, to a large degree, in floatation. Ice is weak in tension. If the stage rises, or if the ice thickens considerably, the increased upward force of the water causes tension cracks to appear at the banks. The ice floats up to a position in equilibrium with the water, and water fills the tension cracks and freezes. The result is again a solid sheet in equilibrium with the river. If the stage drops, the unsupported weight of the ice again causes tension cracks, especially at the banks, and the ice drops to an equilibrium position with respect to the water. Water again fills the tension cracks, freezes, and again a solid sheet of ice results.

On narrow streams the ice may be in floatation, bridged or under pressure. If the stream is so narrow or the ice so thick that the ice can resist the tensile stress placed on it by changes in stage, the ice will not change position regardless of change in stage. At high stages the stream, in effect, will be flowing in a pressure conduit; at low stages the ice sheet will be bridged so that it makes no contact with the water. This is particularly true when there are large boulders in the stream to which the ice is frozen, thereby reducing the length of the unsupported free span.

Effect of surface ice on stream hydraulics

Surface ice when in contact with the stream may, in effect, change streamflow from open channel flow to closed conduit flow. Frictional resistance is increased because a water-ice interface replaces the water-air interface, hydraulic radius is decreased because of the additional wetter perimeter of the ice, and the cross-sectional area is decreased by the thickness of the ice. The stage will therefore increase for a given discharge. Figure II.1.19 shows the water stage recorder graph for a gauging station as the formation of surface ice begins to cause backwater effect.

In this example, daily mean discharge remained about the same as before the freeze-up, although the discharge undoubtedly fluctuated somewhat during each day. It can be seen from Figure II.1.19 that surface ice can cause much uncertainty regarding the discharge because the stage-discharge relation becomes indeterminate. It is evident in Figure II.1.19 that backwater effect exists and is increasing, because the rise looks very unnatural, but the amount of backwater effect cannot be determined directly from the recorder chart.

Surface ice can also cause siphon action when it forms on a section control, but that effect is not

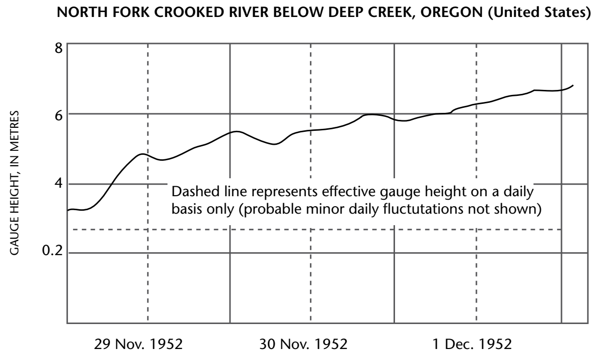


Figure II.1.19. Typical rise as complete ice cover forms (after Moore, 1957)

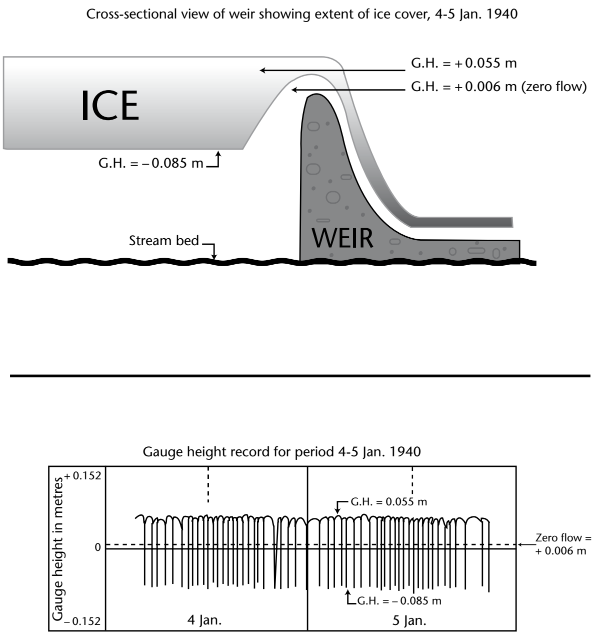


Figure II.1.20. Effect of siphon action at artificial control in Sugar Run at Pymatuning, Pennsylvania, United States, 4-5 January 1940

very common. In Figure II.1.20, when water filled the entire space between control and ice, siphon action began and water flowed over the control faster than it entered the gauge pool. The gauge pool was pulled down 0.100 m below the gauge height of zero flow when air entered the system and broke up the siphon action. Discharge ceased and then became a trickle while the inflow again filled the gauge pool. When the entire space between control and ice was filled once more, siphon action began again. Siphon action is easily recognizable from the rapid fluctuations of the stage record. If the gauging station is visited at that time, the discharge measurement should be made far enough upstream from the gauge pool to be beyond the effect of the fluctuating pool level.

If the section control is open and the gauge is not too far removed from the control, there will probably be no backwater effect even though the entire pool is ice covered. The only effect of the ice cover will be to slow up the velocity of approach and this effect will probably be minor. If the gauge, however, is a considerable distance upstream from the riffle, surface ice on the pool may cause backwater as the covered reach of pool becomes a partial channel control. Ice forming below an open section control may jam and raise the water level sufficiently to introduce backwater effect at the control.

1.13.5 **Computation of discharge during periods of backwater from anchor ice**

Discharge measurements are usually not made when anchor ice is present, for the following reasons. First, adjustment of the stage record for the effect of anchor ice can be made quickly and reliably. Second, a discharge measurement made at that time is of little help in the analysis because discharge is highly variable with time as a result of water entering or leaving channel storage.

Anchor ice rises are clearly recognizable on the recorder chart. In computing discharge for periods of anchor ice effect, adjustments to gauge height are made directly on the gauge height graph. In Figure II.1.18, the long dashed line connecting the low points of the anchor ice hump is the effective gauge height to use during the hours when the hump was recorded. Actually, the true effective gauge height is shown by the short dashed line. As the anchor ice builds up, the flow decreases faster than the normal recession shown by the long dashed line, because some of the flow is going into a storage as a result of the increased stage.

When the anchor ice goes out at about 9 or 10 a.m., a slug of water is released from storage and the true effective gauge height rises. It can be seen, however, that the areas formed by the short dashed lines above and below the long dashed line balance and we should get nearly identical results from use of either of the dashed lines. The rule then for obtaining effective gauge height during anchor ice periods is to cut off the hump with a straight line connecting the low points.

1.13.6 Computation of discharge during periods of backwater from surface ice

Figure II.1.21 is an example of how discharge measurements (Nos. 5, 37 and 38), made during periods of ice effect, plot on a rating curve. Figure II.1.19 is an example of a gauge height graph as complete ice cover forms. It is apparent from Figure II.1.19 that the backwater effect from surface ice cannot be determined directly from the recorder chart or plot of the gauge readings. The recorder chart or gauge height graph is very helpful, however, in determining which periods during the winter that are ice affected.

Complete notes describing ice conditions at the times the station was visited are also very valuable. Most important of all are discharge measurements made during ice-affected periods. A discharge measurement gives a definite point on a hydrograph plot of daily mean discharge versus date Figure II.1.22), through which the graph of estimated true daily discharge must pass. If little change in stage occurred during the day the discharge measurement was made, the measured discharge is considered to be the daily mean discharge. If a significant change in stage occurred that day, the daily mean discharge, Q, is computed from the formula:

$$Q = Q_a \frac{Q_m}{Q_r} \tag{1.14}$$

where Q_a is the discharge from the open water (ice free) rating curve corresponding to the daily mean gauge height; Q_m is the measured discharge, and Q_r is the discharge from the open water rating curve corresponding to the gauge height of the discharge measurement.

Several methods of correcting open water discharge for ice effect are in use. The term open water discharge, as used in this section of the Manual, refers to the discharge for ice-free conditions, obtained by applying the gauge height record to the rating or shift curve that was in use immediately before the start of the ice-affected period. The methods are:
(a) Discharge ratio method (sometimes known in the United States as the Lithuanian method);
(b) Shifting control method or Stout method;
(c) Direct discharge interpolation method;
(d) Hydrographic and climatic comparison method.

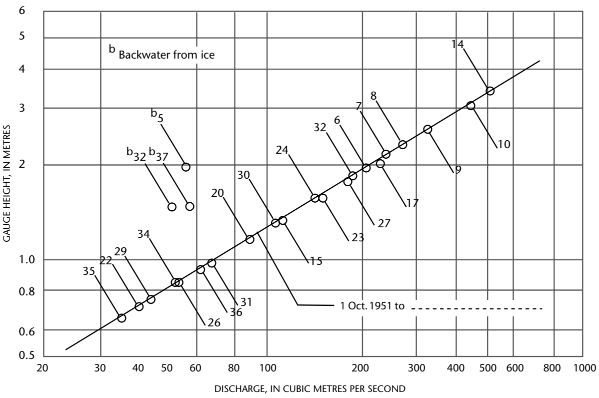


Figure II.1.21. Rating curve for Menominee River near Pembine, Wisconsin, United States

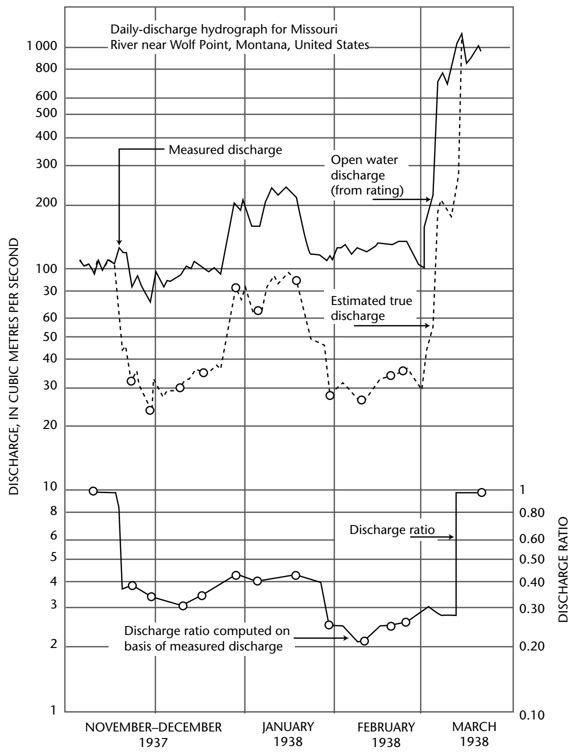


Figure II.1.22. Example of discharge-ratio method for correcting discharge record for ice effect

Some of the methods are somewhat similar, and vary mainly in how they are applied. The reliability of each of the methods varies almost directly with the number of discharge measurements that were made during the ice-affected period that is being studied. Regardless of the method used, the corrected hydrograph of daily discharge, if possible, should be checked for consistency with other records. Or in some cases, two or more of the above methods may be used for comparison purposes. The method that is most consistent with information from other sources would then be used. If the station being studied is on a stream that carries natural flow (flow not significantly affected by man-made development) its corrected record is compared with those for nearby streams that likewise carry natural flow. Particularly useful for that purpose are the hydrographs of streams that are unaffected by ice. If the station being studied is on a regulated stream, its corrected hydrograph is compared with the record of upstream reservoir releases or upstream hydroelectric generation, expressed either in units of discharge or in units of power output.

Discharge ratio method

In the discharge ratio method which is used in many European countries, the open water daily mean discharge is multiplied by a variable factor K to give the corrected discharge during periods of ice cover. A value of K is computed for each discharge measurement as the ratio of measured discharge (Q_m) to the open water discharge (Q_r). Because K varies during the winter with time, as changes occur in the ice cover, the value of K for use on any given day is obtained by interpolation, on the basis of time, between K values computed for consecutive discharge measurements. Meteorological data are generally used to modify the simple interpolation between K values for consecutive discharge measurements; for example, during a period of extremely low temperatures the values of K indicated by simple interpolation would be reduced because the discharge usually decreases sharply at such times. The dates on which ice effect begins and ends are based on the observed or deduced beginning and end of ice cover.

An example of the discharge ratio method is shown in Figure II.1.22. Note that discharge is plotted on a logarithmic scale. The upper daily hydrograph shows open water discharges and the solid circles are discharge measurements. The lower graph shows the K values obtained from discharge measurements (open circles) and the interpolation between those values; the middle graph is the hydrograph of estimated true daily discharges, obtained by multiplying concurrent values from the upper and lower graphs. The nonlinear interpolations for K values during the periods 9-23 November, 18 January to 19 February, and 24 February to 20 March, were based on the observer's notes concerning ice conditions and on temperature and precipitation records (not shown in Figure II.1.22).

Shifting control method

The shifting control method, at one time the standard method used in the United States, is seldom used there now but is still used in other countries. In the shifting control method, recorded gauge heights are reduced by a variable backwater value to obtain the effective daily gauge heights (sometimes called the equivalent gauge height). The effective gauge heights are then applied to the open water rating to obtain estimated true daily discharges. The backwater correction on days when discharge measurements are made is computed as the difference between the actual gauge height and the effective gauge height, effective gauge height being the gauge height from the open water rating that corresponds to the measured discharge. The backwater correction for use on any given day is obtained by interpolation, on the basis of time, between the backwater corrections computed for consecutive discharge measurements. As in the discharge ratio method, the interpolation is subject to modification on the basis of meteorological records and the dates on which ice effect begins and ends are based on the observed or deduced beginning and end of ice cover.

An example of the shifting control method is shown in Figure II.1.23 for the same gauging station used in the example in Figure II.1.22. Note that an arithmetic (not logarithmic) scale is used in Figure II.1.23. The upper daily hydrograph in Figure II.1.23 shows recorded gauge heights and the solid circles are the effective gauge heights for discharge measurements. The lower graph shows the backwater corrections obtained from discharge measurements (open circles) and the interpolation between those values. The middle graph is the hydrograph of effective gauge height obtained by subtracting values on the lower graph from concurrent values on the upper graph. The nonlinear interpolations for backwater corrections during various periods were based on the observer's notes concerning ice conditions and on temperature and precipitation records (not shown in Figure II.1.23). As mentioned in the preceding paragraph the effective gauge heights (middle graph) are applied to the rating curve to obtain estimated true daily discharges.

Direct discharge interpolation method

Daily mean discharges can be determined by direct interpolation between discharge measurements, usually linear or log-linear, during periods when it

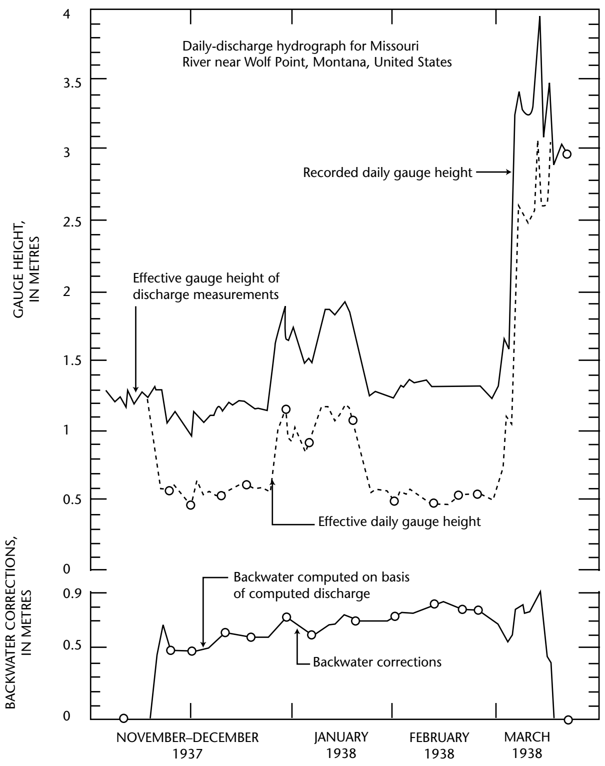


Figure II.1.23. Example of shifting-control method for adjusting stage record for ice effect

is believed that discharge changes are fairly uniform. The interpolation procedure can be modified at times when the air temperature and stage records indicate significant changes.

Hydrographic and climatic comparison method

The method of hydrographic and climatic comparison has been favored in the United States for many years. The mechanics of the method differ from those of the discharge ratio method, but both methods basically correct the daily open water discharge by a variable percentage.

The first step is to compute the station discharge record for the entire year as though there were no ice effect at any time. The daily hydrograph of open water discharge and the discharge measurements are then plotted, using a logarithmic discharge scale, and notes concerning ice conditions are entered on the graph. At this point the hydrograph sheet resembles the upper graph in Figure II.1.22. Where a measurement of ice affected discharge is not representative of the daily mean discharge, because of changing stage during the day, the daily mean discharge, as computed by equation 1.15, is also plotted. All is then in readiness for estimating the true daily discharge directly on the hydrograph sheet and that is done on the basis of three comparisons:

(a) Comparison with records for nearby gauging stations;
(b) Comparison with weather records;
(c) Comparison with base flow recession curve.

Comparison with records for nearby gauging stations

Comparison with other discharge records is the most important basis for determining the probable discharge for periods between discharge measurements. Even though the record used for comparison may also have been corrected for ice effect, its use provides an additional independent set of basic data, that is, another stage record and another set of current meter measurements. Without a nearby record that compares well with the record being studied, the accuracy of the daily discharges estimated between the dates of discharge measurements may be greatly reduced. However, hydrographic comparisons are not infallible because the relation between the flow of two streams may vary significantly during the year; hence the importance of making many discharge measurements during ice-affected periods.

In making the hydrographic comparison, the nearby station with the most reliable winter streamflow record is selected for use as a reference station. The reliability of the reference station may have been established by the fact that its discharge is unaffected by ice or is affected by ice for only a relatively short period, or by the fact that many winter measurements have been made at the station and the true discharge between the dates of measurement can be estimated from weather records. (See discussion below on use of weather records.) A hydrograph of daily discharge, corrected for ice effect if necessary, is prepared for the reference station on a separate sheet of graph paper, similar to that used for plotting the daily hydrograph for the station being studied.

A light table is used in comparing the two hydrographs or they are graphically compared by a computer plotting program; for example, the United States Geological Survey (USGS) uses a graphic program that allows one daily discharge hydrograph to be moved on the screen to overlay another hydrograph. Whether using a light table or computer program, the reference station hydrograph is used. The hydrograph for the study station is superposed on that of the reference station and positioned laterally so that the date lines of the two hydrographs coincide. The period preceding the first measurement (No. 1) that showed ice effect at the study station is the period first selected for consideration. The hydrograph for the study station is positioned vertically so that hydrographs for the two stations roughly coincide for the period immediately preceding the day or days when the start of ice effect is suspected. A comparison of the hydrographs and an inspection of the weather records should fix the date when ice effect started. That date will be preceded by a period of subfreezing weather, and on that date, usually a rainless day, the hydrograph for the study station will start a gradual rise not shown by the hydrograph for the reference station. For an appreciable period thereafter the hydrograph for the study station will remain above that of the reference station.

After the starting date, *A*, of ice effect at the study station has been selected, the vertical position of the hydrograph for the study station is changed slightly, if necessary, to make the two hydrographs coincide on that date. If that positioning causes measurement No. 1 to fall directly on the hydrograph for the reference station, the hydrograph for the reference station between date *A* and measurement No. 1 is traced with dashed lines on the hydrograph sheet for the study station. The daily discharges indicated by the dashed lines are the estimated true discharges at the study station during the period between date *A* and measurement No. 1.

However, it is a rare situation where measurement No. 1 coincides with the reference hydrograph when discharges at the two stations are made to coincide on date *A*. Measurement No. 1 will usually lie above or below the hydrograph for the reference station. In that situation, as discharges from the reference hydrograph are being transferred to the sheet bearing the study hydrograph, the study sheet will in effect be moved up or down, as the case may be, so that when the transfer of discharge points reaches measurement No 1, measurement No. 1 will coincide exactly with the reference hydrograph. If the temperature record shows no great fluctuation from day to day during the period between date *A* and measurement No. 1, the vertical displacement of the sheet bearing the study hydrograph will be made uniformly during the transfer process. If the temperature record does fluctuate from day to day during the period, the vertical displacement will be made at a variable rate to reflect the fact that the ratio of true discharge to open water discharge usually decreases during sharp drops in temperature; the ratio increases during sharp rises in temperature. In other words, the vertical distance between open water discharge and true discharge will increase on the study hydrograph sheet during sharp drops in temperature. The vertical distance decreases during sharp rises in temperature. Observer's notes concerning major changes in the ice cover, particularly where complete cover is intermittent during the winter, are also very helpful in estimating the degree of ice effect.

After correcting the discharge between date *A* and measurement No. 1, the process is repeated for the period between discharge measurement No. 1 and the next successive discharge measurement (No. 2). The two hydrographs are made to coincide at measurement No. 1 and the transfer of discharge points to the study hydrograph proceeds to measurement No. 2. In that manner the open water discharge for the study station is corrected until the date is reached when ice effect ceases.

Comparison with weather records

Records of air temperature and precipitation are a most valuable aid in making corrections for ice effect. The temperature record helps the hydrologist decide whether the precipitation is rain or snow. Snow will have no immediate effect on the runoff. The temperature record also helps the hydrologist decide whether ice cover is forming, increasing or dissipating. For stations for which there are no nearby discharge records for comparison and for which the recorder chart does not provide dependable clues to the fluctuation of discharge, it may be necessary to correct open water discharges for ice effect almost solely on the basis of weather records and available measurements of discharge. Discharge usually follows closely the ups-and-downs of the air temperature record and the discharge measurements help fix, within reasonable limits, the estimated rises and falls of the true discharge hydrograph. An exception to that statement is found in regions of extreme cold, such as the Arctic, that become blanketed with a heavy snow cover. The snow acts as an insulator for the underlying ground and it then requires a prolonged change in temperature to significantly change the slow uniform recession of streamflow during the winter.

It should be mentioned here that a water temperature recorder is a helpful adjunct to a gauging station. When the water temperature is above the freezing level there is little likelihood of ice effect.

Comparison with base flow recession curves

During periods of sub-freezing weather virtually all the flow in a stream is base flow; that is, water that comes out of ground-water storage to sustain the flow of the stream during periods when there is no surface runoff. It will often be found that during cold ice affected periods, the flow of the stream will be declining at a rate similar to the rate of recession shown by that stream during ice free periods. For example, consider a situation where a discharge of 0.56 $m^3 s^{-1}$ is measured on a specific day during the ice affected period, and an estimate of daily discharge is required for the next ten days, all of which were free of rain or snowmelt. An ice free period elsewhere in the record is then selected for the study station when there was no surface runoff, and one day during that period had a discharge of 0.56 $m^3 s^{-1}$. The receding values of discharge for the following ten days of that ice free period are then used for the ten days to be estimated. The ice free period that is used for an index should preferably be in the non-growing season because the use of water by vegetation affects the rate of base flow recession.

It is possible that daily discharges estimated from the base flow recession may be somewhat high because extremely cold weather reduces the rate at which water percolates through the ground and because some of the water that does reach the stream may go into storage behind ice dams. Nevertheless a standard base flow recession curve provides a valuable guide to the probable flow during recession periods when the stream is ice covered. Because the discharge during periods of base flow originates as groundwater, a record of the fluctuations of groundwater levels of wells in the area can be useful as an index for estimating the true discharge during those periods.

Example of hydrographic and climatic comparison method.

An example of the application of the hydrographic and climatic comparison methods is illustrated in Figures II.1.24 and II.1.25. Figure II.1.24 shows a portion of a plotted hydrograph of daily mean discharge for the gauging station on North Fork John Day River at Monument, Oregon, United States. The solid line represents open water discharge obtained by applying recorded gauge heights to the rating curve, and the open water discharge on 26 January corresponds to the gauge height of discharge measurement *C* made on that date. Note that the open water discharge is almost ten times as great as the measured discharge on 26 January. The dashed line on Figure II.1.24 represents the estimated true daily discharge obtained by comparison with the hydrograph of daily mean discharge for John Day River at Service

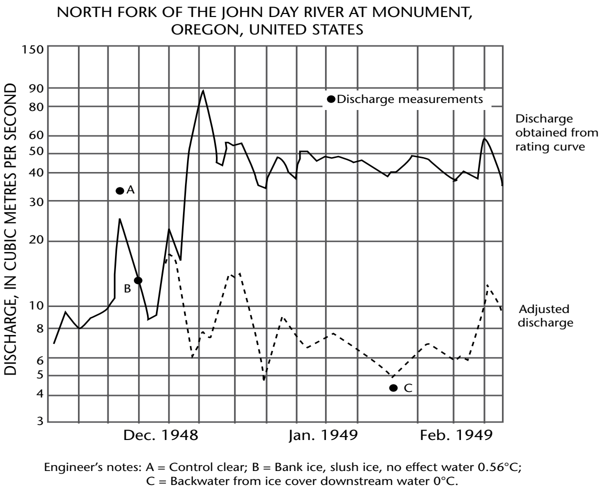


Figure II.1.24. Daily hydrographs for open-water discharge and for discharge corrected for ice effect (after Moore, 1957)

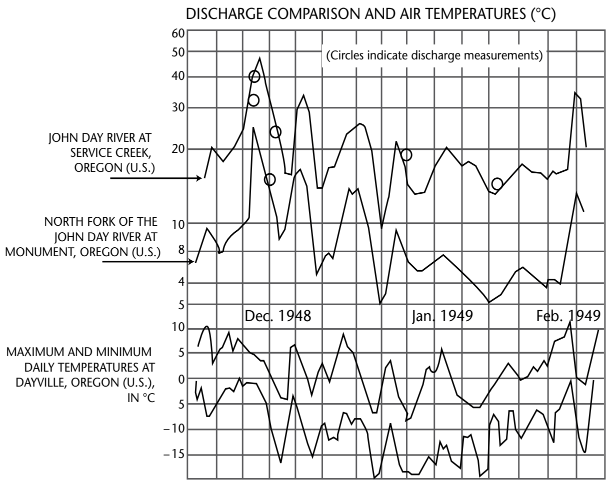


Figure II.1.25. Comparison of daily winter discharge at two gauging stations showing their response to air-temperature fluctuations (after Moore, 1957)

Creek, Oregon, and by comparison with the record of daily maximum and minimum temperature at Dayville, Oregon, United States. The reference hydrograph and temperature record used for the comparison are shown in Figure II.1.25. Actually the precipitation record at Dayville was also considered, but because all precipitation during the study period occurred as snow and therefore had no immediate effect on the runoff, the precipitation record is not shown in Figure II.1.25.

The corrected hydrograph for the study station on North Fork John Day River at Monument is also shown on Figure II.1.25. The hydrograph of open water discharge at that station has been omitted to reduce clutter in the illustration. The discharge for the reference station on John Day River at Service Creek was unaffected by ice. The shapes of the two hydrographs are not identical, but a useful comparison between the hydrographs for two stations does not require that their shapes be identical, as long as their discharge trends are similar. It can be seen on Figure II.1.25 that both hydrographs respond to the effect of air temperature fluctuations during the winter period.

In applying the method of hydrographic and climatic comparison, the hydrograph of true daily discharge, plotted on a logarithmic scale, was displaced from the open water hydrograph by a variable vertical distance. That means, in effect, that discharge ratios, variable with time, were applied to the open water discharges, and therefore a basic similarity exists between the hydrographic comparison method and the discharge ratio method. Application of the hydrographic comparison method would be greatly facilitated if the hydrograph of open water discharge for the study station were first adjusted by the discharge ratio method because application of that method is relatively simple. The adjusted hydrograph would then be refined by using it, rather than the open water hydrograph, in the hydrographic comparison method. It is much simpler to apply the hydrographic comparison method for refining discharge estimates than it is to apply that method for making original discharge estimates.

Use of Acoustic Doppler Velocity Meters for ice-affected periods

In making the hydrographic comparisons during ice-affected periods, the nearby station with the most reliable winter streamflow record is selected for use as a reference station. Discharge records generated using Acoustic Doppler Velocity Meters (ADVM) have the potential to produce reliable records of discharge during ice periods, even when the surface is totally ice covered (Morlock and others, 2002). The implications from this potential are: (a) installation of an ADVM at a station where installation is feasible (ADVM installation considerations are discussed in Volume I, Chapter 5) may provide an improved estimate of discharge during ice periods; and (b) A station equipped with an ADVM may provide an ideal station for hydrographic comparisons for stations not equipped with an ADVM but requiring ice estimates. Use of ADVMs to produce discharge records are discussed in more detail in Chapter 2 of this Volume.

1.14 SAND CHANNEL STREAMS

In fixed channels well-defined stage discharge relations can usually be developed that show only minor shifting at low flow. In sand channel streams, however, stage-discharge relations are continually changing with time, because of scour and fill, and because of changes in the configuration of the channel bed. These changes cause the shape and position of the stage-discharge relation to vary from time to time and flood to flood, and it becomes very difficult to explain the apparent haphazard scatter of discharge measurements available to define the rating. Familiarity with the results of research studies as reported by Colby (1960), Dawdy (1961), Simons and Richardson (1962), Beckman and Furness (1962) and Culbertson and Dawdy (1964) will greatly assist the analyst in defining the discharge rating.

1.14.1 Bed configuration

On the basis of laboratory investigation, Simons and Richardson (1962) described the bed configuration of sand channel streams as ripples, dunes, plane bed, standing waves and antidunes. This sequence of bed configurations occurs with increasing discharge. When the dunes wash out, and the sand is rearranged to form a plane bed, there is a marked decrease in resistance to flow which may result in an abrupt discontinuity in the stage-discharge relation. The forms of bed roughness, as shown in Figure II.1.26 and described in Table II.1.2, are grouped according to the two separate conditions of depth-discharge relationship that are evident in a given channel. The sequence of configurations described in Table II.1.2 is developed by continually increasing discharge. The lower regime occurs with lower discharges and the upper regime with higher discharges. An unstable discontinuity in the depth discharge relationship appears between these two more stable regimes.

The presence of fine sediment in the flow influences the configurations of the sand bed, and thus the resistance to flow. It has been found by Simons and Richardson (1962) that with concentrations on the order of 40 000 milligrams per litre of fine material, resistance to flow in the dune range is reduced as much as 40 per cent. The effect is less pronounced in the upper regime but fine sediment may change a standing wave condition into a breaking antidune which will increase the resistance to flow. Thus the stage-discharge relationship for a stream may vary with sediment concentration if the flow is heavily laden with fine sediment.

Table II.1.2. Surface and bed descriptions for the various flow regimes

Type of configuration	*Description*	
	Bed	*Flow*
Lower regime flow:		
Plane bed	Plane; no sediment movement	Plane surface; little turbulence
Ripples	Small uniform waves; no sediment movement	Plane surface; little turbulence
Dunes	Large, irregular, saw-toothed waves formed by sediment moving downstream; waves move slowly downstream	Very turbulent; large boils
Uper regime of flow:		
Plane bed	Dunes smoothed out to plane bed	Plane surface; little turbulence
Standing waves	Smooth sinusoidal waves in fixed position	Standing sinusoidal waves in phase with bed waves; termed "sand waves"
Antidunes	Symmetrical sinusoidal waves progressing upstream and increasing in amplitude; suddenly collapse into suspension then gradually reform	Symmetrical sand waves progressing upstream in phase with bed waves; amplitude increases until wave breaks, whole system collapses then gradually reforms

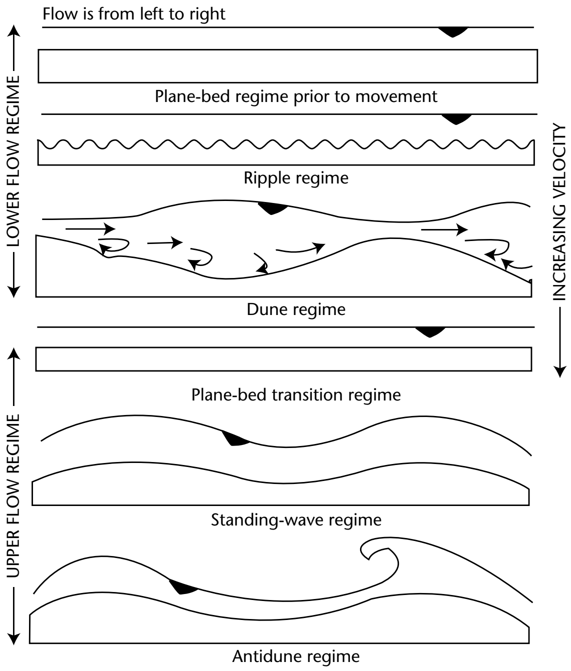


Figure II.1.26. Idealized diagram of bed and water surface configuration of a fluvial streams with various regimes of flow

Changes in temperature can also alter the form of bed roughness, and, hence, the resistance to flow. Lowering the temperature increases the viscosity of the water and increases the mobility of the sand. If, for example, the form of bed roughness is near to or in transition, and there is a reduction in the temperature of the water, the increased mobility of the sand may cause the dunes to wash out, and the bed to become plane. This phenomenon is reversible.

Changes in bed forms do not occur instantaneously with increasing or decreasing discharge. The time lag between change in bed form and change in discharge may result in loop rating curves. For example, if the bed configuration is initially dunes, the dunes will persist on rising stages to a discharge that is greater than the discharge at which the dunes will reform on falling stages. Thus at a given stage, the discharge may be greater when the stage is falling. Because the form of each loop curve depends on the initial condition of bed configuration and the rate of change of discharge, an infinite number of different loop curves, and even multiple-loop curves, may occur for a given reach of channel across the transition from dunes to plane bed. The stage-discharge relation within the transition band may be indeterminate. An example of a loop curve, typical of some sand channels, is shown in Figure II.1.27.

1.14.2 Relation of mean depth to discharge

A plot of stage against discharge in sand channel streams often obscures any underlying hydraulic relationship because neither the bottom nor sides of these streams are fixed. Figure II.1.28 shows as an extreme example of the stage-discharge plot for Huerfano River near Undercliffe, Colorado, United States for 1941 and 1942. The relation between stage and discharge is indeterminate. However, the

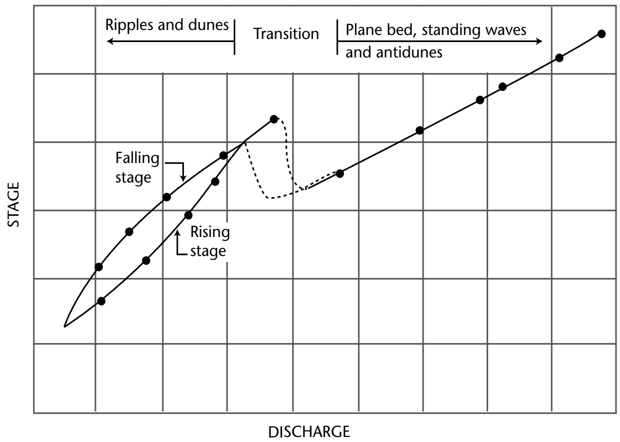


Figure II.1.27. Typical loop curve of stage versus discharge for a single flood-event in a sand channel (after Stepanich, Simons and Richardson, 1964)

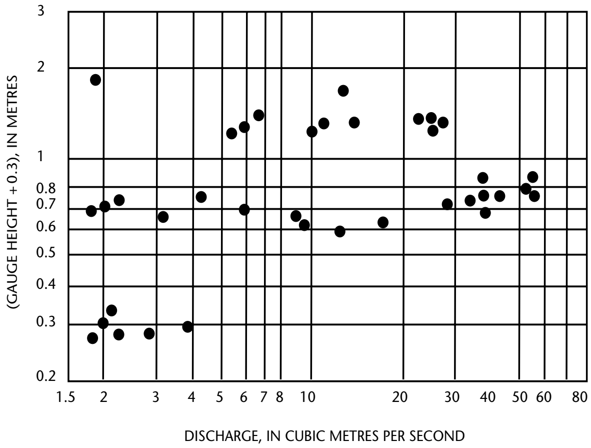


Figure II.1.28. Stage-discharge relation for Huerfano River near Undercliffe, Colorado, United States. From Dawdy (1961)

underlying hydraulic relation may be revealed by a change in variables. The effect of variation in bottom elevation is eliminated by replacing stage by mean depth or hydraulic radius. The effect of variation in width is eliminated by using mean velocity. Figure II.1.29 shows most of the same measurements for Huerfano River as were plotted in Figure II.1.28, replotted on the basis of velocity and hydraulic radius. Measurements for this stream with a hydraulic radius greater than 0.3 metres define a single curve with bed forms corresponding to the upper regime. Measurements in the transition range from dunes to plane bed scatter wildly as would be expected from the previous discussion.

The discontinuity in the depth discharge relation is further illustrated in Figure II.1.30 which shows a plot of hydraulic radius against velocity for Rio Grande near Bernalillo, New Mexico, United States. The measurements plotted on the left represent bed configurations of ripples and dunes and the curve on the right represents bed configurations of plane bed, standing waves or antidunes.

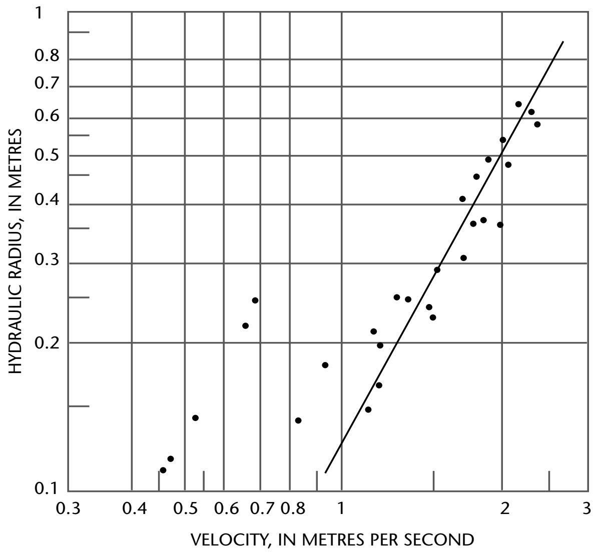


Figure II.1.29. Relation of velocity to hydraulic radius for Huerfano River near Undercliffe, Colorado, United States. From Dawdy (1961)

According to Dawdy (1961) the curve representing the upper regime in a true sand bed stream usually fits the following relation:

$$V = kR^{1/2} \tag{1.15}$$

where V is the mean velocity; k is a constant and R is the hydraulic radius.

He found this relation to be applicable for 26 of the 27 streams used in his study. More recent study has shown that the exponent of R ranges from 2/3 as in the Manning equation, to 1/2, the larger exponents being associated with the coarser grain sizes.

1.14.3 **Development of discharge rating**

Plots of mean depth or hydraulic radius against mean velocity or discharge per foot of width are valuable in the analysis of stage-discharge relations. These plots clearly identify the regimes of bed configuration and assist in the identification of

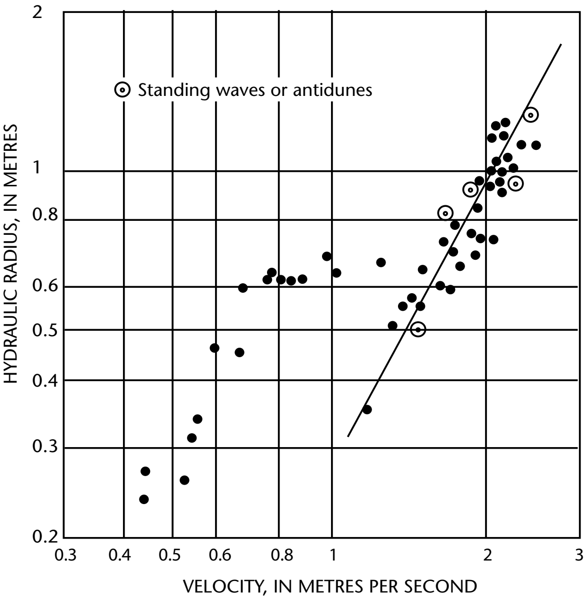


Figure II.1.30. Relation of velocity to hydraulic radius for Rio Grande near Bernalillo, New Mexico, United States. From Dawdy (1961)

the conditions represented by individual discharge measurements. For example, only those measurements identified with the upper regime should be used to define the position and slope of the upper portion of the stage-discharge curve. Similarly, only those measurements identified with the lower regime should be used to define the lower portion of the stage-discharge curve. Measurements made in the transition zone may be expected to scatter widely but do not necessarily represent shifts in more stable portions of the rating.

Plots of stage against mean depth and stage against width are also helpful in developing a mean stage-discharge relation and in analyzing the cause of shifts from the mean relation. In the upper regime the use of these plots in conjunction with the plot of velocity versus mean depth or hydraulic radius raised to the 1.5 to 2/3 power, (depending on grain size) may be useful in establishing a reasonable slope to the upper portion of the stage-discharge relation.

The stage-discharge relation developed by Colby (1960) for Pigeon Roost Creek, Mississippi, United States, is shown in Figure II.1.31. This stream is about 22.86 m wide, the banks are relatively stable and the median size of the bed material is 0.4 mm. The mean elevation of the channel bed does not change appreciably with time or discharge. The discontinuity in the stage-discharge relation is very abrupt. Discharges from 25 to 50 $m^3 s^{-1}$ may occur at a stage of 1.6 m.

According to Colby (1960), stage-discharge relations may be expected to have a discontinuity provided the reach has all of the following characteristics:

(a) A bed of uniform and readily shifting sediment which does not form distinct pools and riffles;
(b) At some flows almost all of the stream-bed is covered with loose sand dunes;
(c) At higher flows the bed of the stream is mostly plane or has antidunes;
(d) The depth of flow at the point of discontinuity must be great enough so that changes in the stage-discharge relation at the discontinuity can be distinguished from changes caused by small local shifts of the channel bottom;
(e) The lateral distribution of depths and velocities must be sufficiently uniform for the bed configuration to change across most of the stream-bed in a relatively short time.

The above conditions are very restrictive. Many streams with sand beds have well-developed pools and riffles at the stage where the discontinuity might otherwise occur. Streams do not generally have uniform sediment sizes; many have large sorting coefficients. A few streams having suitable bed material may never show the discontinuity because dunes exist even at the highest flow rates. Others may have such high slopes that the lower regime cannot be defined by discharge measurements because of the shallow depths at which the discontinuity occurs. Winding streams seldom have uniform lateral distribution of

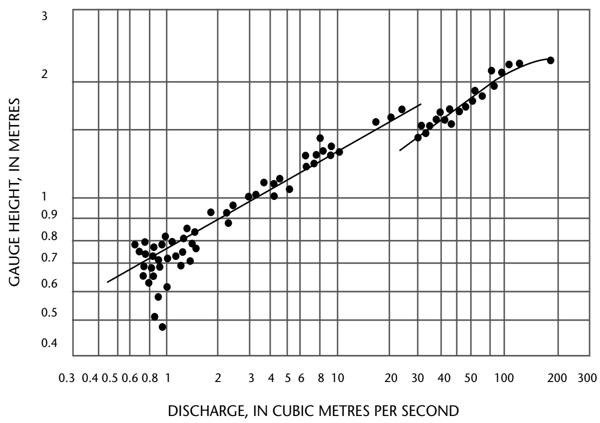


Figure II.1.31. Stage-discharge relation for station 34 on Pigeon Roost Creek, Mississippi, United States. After Colby (1960)

velocity and depths. Some streams have such gradual or inconsistent transitions between dunes and plane bed that the discontinuity may be difficult if not impossible to define. Dunes may exist near the banks at the same time that a plane bed exists near the centre of the stream. The transition in this case may occur so gradually with increasing stage that the discontinuity in rating is eliminated. However, at any station where dunes exist at low flows and a plane bed exists at higher flows, there is a major change in bed roughness. Knowledge of the bed forms that exist at each stage or discharge can be very helpful in developing the discharge rating.

1.14.4 Evidence of bed forms

Evidence of the bed forms that exist at a given time at a particular station can be obtained in several ways, a listing of which follows:

(a) Visual observation of the water surface will reveal one of several conditions: large boils or eddies, which indicate dunes; a very smooth water surface, which indicates a plane bed; standing waves, which indicate smooth bed waves in phase with the surface waves; or breaking waves, which indicate antidunes. Visual observations of the water surface should be recorded on each discharge measurement;
(b) Noting whether the sand in the bed is soft or firm. A soft bed often indicates lower regime conditions. The stream-bed during upper regime flow will usually be firm;
(c) Measurements of bed elevations in a cross section will usually indicate the type of bed forms. A large variation in depths indicates dunes and a small variation in depths a plane bed. The small variation in depths for a plane-bed (upper-regime) configuration should not be confused with small variations caused by ripples or by small dunes, both of which are definitely lower-regime configurations. A large variation in bed elevation at a particular point in the cross-section during a series of discharge measurements indicates the movement of dunes;
(d) The amount of surge on a recorder chart may also indicate the configuration of the channel bed. Medium surge may indicate dunes, little or no surge may indicate a plane bed, and violent surge may indicate standing waves or anti-dunes. The transition from plane bed to dunes during a recession of discharges may cause a secondary hump on the gauge height trace if the transition occurs over a short time period;
(e) Relationships which define the occurrence of bed forms as a function of hydraulic radius, R, in metres, slope, S, mean velocity, V, in metres per second and median grain size, in millimeters, are useful in developing discharge ratings. A relationship of this type presented by Simons and Richardson (1962) is shown in Figure II.1.32.

Recent studies suggest that the lower regime of bed forms will occur when the ratio:

$$\frac{V^4}{g^2 D^{1/2} d_{50}^{3/2}}$$

is less than 1 x 10^3, that the upper regime of bed forms will occur when the ratio is greater than 4 x 10^3 and that the bed will be in transition if the ratio is between these values. In the ratio, V is the mean velocity in metres per second, g is the acceleration of gravity in metres per second, D is the mean depth in metres and d_{50} is the median grain size in metres.

1.14.5 Shifting controls

The upper part of the stage-discharge relation is relatively stable if it represents the upper regime of bed forms. Rating shifts that occur in upper regime flow can be analyzed in accordance with the methods or principles discussed in previous sections on rating curve shifts. However, the shift ratings after minor stream rises will generally have a strong tendency to parallel the base rating when plotted on arithmetic coordinate graph paper. That is, the equation for each shift curve will differ from that of the base rating by a change in the value of e in equation 1.4. The shifts will change on stream rises and will often vary with time between rises. Major stream rises may also change the value of C in equation 1.4.

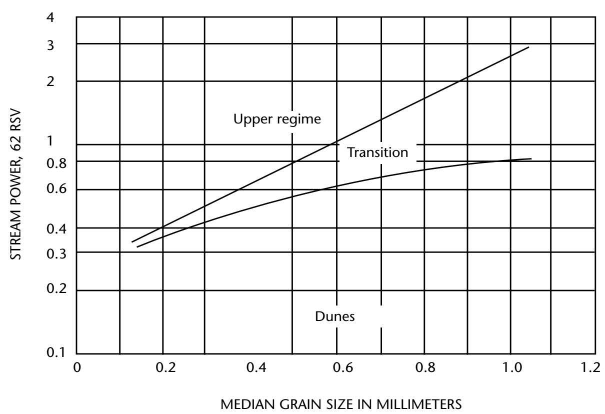


Figure II.1.32. Relation of stream power and median grain size to form of bed roughness

The lower part of the rating is usually in the dune regime and the stage-discharge relation varies almost randomly with time. Frequent discharge measurements are necessary to define the stage-discharge relation and for some streams they are necessary to determine the variation of discharge with time in the absence of any usable relation between stage and discharge. A frequency of three discharge measurements per week is often recommended, but for some streams even daily measurements barely suffice.

A mean curve for the lower regime is frequently used with shifts as defined by discharge measurements. In some instances the shift defined by a single discharge measurement represents only the temporary position of a dune moving over a partial section control. A series of discharge measurements made at short time intervals over the period of a day may define a pattern of shifts caused by dune movement. When discharge is constant but the stage fluctuates, the changing gauge height trace generally reflects dune movement.

Continuous definition of the stage-discharge relation in a sand channel stream at low flow is a very difficult problem. The installation of a control structure should be considered if at all feasible.

1.14.6 **Artificial controls for sand channels**

When conventional weirs are installed in sand channels they are seldom satisfactory, even when designed to be self-cleaning. The principle difficulty is that for such weirs in a sand channel, discharge is dependent not only on water surface elevation, but also on the bed elevation and flow regime upstream from the structure. A satisfactory weir is one whose stage-discharge relation is unaffected by bed configuration. A few successful low water controls have been designed for use in sand channels. One example is the weir designed for the gauging station on the Rio Grande conveyance channel near Bernardo, New Mexico, United States (Richardson and Harris, 1962). That structure will not be described here because generalizations concerning weir shape are meaningless. Each control structure must be individually designed for compatibility with channel and flow conditions that exist at the proposed site for the control. A laboratory model study involving a reach of channel is therefore needed for each site investigated. Efforts continue to design low water controls that are both relatively cheap and that have satisfactory operating characteristics when installed in sand channels (Stepanich et al., 1964).

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CHAPTER 2

DISCHARGE RATINGS USING THE VELOCITY INDEX METHOD

2.1 INTRODUCTION

Discharge ratings using the velocity-index method have been used for many years on streams where variable backwater hinders or prohibits the use of a simple stage-discharge rating to compute stream discharge. The first applications were used with a deflection (vane) gauge, or in some cases a mechanical current meter, to determine a continuous record of a velocity index at a point in the stream. Later applications used electronic devices such as an ultrasonic point velocity meter or electromagnetic point velocity meter. Each of these methods of obtaining a continuous record of an index of mean stream velocity at a point are described in Chapter 8 of Volume I and will not be repeated in this chapter.

In recent years, the velocity-index method of creating ratings and computing records of stream discharge has become more prevalent, due to the increase in use of hydroacoustic current meters installed at gauging stations. While the method is discussed mainly in the context of acoustic meters, the method could in theory be applied to any instrumentation that measures water velocities, including deflection vanes, impellor-type fixed meters and electromagnetic meters. This chapter will illustrate the use of hydroacoustic current meters where the velocity index for a segment of the stream is recorded. The development of the velocity-index rating is basically the same for all methods, and that is the primary purpose of this chapter. In addition, this chapter will provide an illustrative example of the method and present some considerations for using the method, such as sources of error.

2.2 BASICS OF THE VELOCITY INDEX METHOD

In the velocity-index method, the water velocity measured by a hydroacoustic current meter in a portion of a river can be used as an index for the mean-channel velocity. Mean-channel velocities computed using velocity-index methods together with the cross-sectional area of the channel can be used to generate records of river streamflow.

Hydroacoustic current meters commonly used for velocity-index applications include Acoustic velocity meters (AVMs), Acoustic Doppler Velocity Meters (ADVM) and Acoustic Doppler Profilers (ADP). Chapter 6 in Volume I contains more detailed descriptions of these instruments and the theory of their operation. The velocity that is measured by the instrument and used as an independent parameter in the velocity-index rating is called the velocity-index. This chapter will illustrate the method by using the ADVM as the instrument for measuring the velocity index.

The velocity-index method can be summarized as follows:

(a) A hydroacoustic current meter is installed in a river where it measures water velocity on a continuous basis for a portion of the channel:
   (i) AVMs measure velocity that can be the line velocity from one acoustic path or from multiple acoustic paths. For example, if the horizontal angle of flow in the channel is constant, the AVM line velocity from one acoustic path may be sufficient to accurately index mean velocity. If the horizontal angle of flow in the channel changes over time, using the average of AVM line velocities from both acoustic paths of a cross-path configuration may be a more accurate index of mean velocity than the velocity from a single path;
   (ii) ADVMs measure velocities in a sample volume. The ADVM-measured velocity used to index mean velocity is usually the sample volume downstream component of velocity. If an ADVM is equipped with multiple sample volumes, the average downstream component of velocity from more than one sample volume can be used to index mean velocity;
   (iii) Profilers measure velocities in uniformly-sized cells or bins along the acoustic beams. By measuring velocities in a number of bins across a channel or vertically through the water column, these instruments produce horizontal or vertical water velocity profiles, hence the designation profiler. The profiler-measured velocity used to index mean velocity can be the velocity measured in one or multiple bins.

(b) Velocity and various current meter performance and quality-assurance data are recorded by the current meter in internal memory or are logged by an Electronic Data Logger (EDL). Some hydroacoustic current meters can measure stage acoustically, or stage can be measured by a separate stage sensor;

(c) A cross-section, called a standard cross-section is surveyed near the current meter installation. The standard cross-section is used to develop a relation between stage and channel area, called a stage-area rating. The channel area computed from the stage-area rating will be used, with mean-channel velocity, to compute discharge;

(d) Discharge measurements are made while stage and velocity-index are measured and recorded. The average stage and average index-velocity are computed for each discharge measurement. The stage-area rating is used to compute channel area (A) from the average stage (S). The measured discharge divided by the channel area computed from the stage-area rating yields average mean velocity (V) for the discharge measurement. The channel area is always computed for the standard cross-section, using the stage-area rating;

(e) Each discharge measurement produces a mean channel velocity (V) and velocity-index (V_i). After multiple measurements have been made, a relation between V and V_i is developed; the relation is called a velocity-index rating. Velocity-index ratings are commonly developed by first creating scatter plots with V as the ordinate (y axis) variable and V_i as the abscissa (x axis) variable. A line or curve is fitted to the plotted points. The line or curve is the velocity-index rating. Many velocity-index ratings can be represented as a mathematical formula (the equation of the plotted line or curve). Single-parameter ratings have one independent variable (V_i) used to compute V. Velocity-index ratings can also have more than one independent variable (such as V_i and S) and are called multi-parameter or complex ratings;

(f) Discharge (Q) is computed from the equation $Q = VA$. V is computed from application of the velocity-index rating to V_i and A is computed from application of the stage-area rating to S.

Some countries have been using a similar method, in which one of the two verticals (one on each side of the cross-section) where, based on previous measurements, the average velocity in the vertical is known to be approximately equivalent to the average velocity in the whole section, is identified. Once the average velocity in this vertical is calculated, by simply multiplying its value by the area of the cross section, a good estimate of the discharge passing through the cross-section can be obtained.

2.3 STAGE-AREA RATING DEVELOPMENT

A stage-area rating is one of the two ratings needed for computing discharge records using the velocity-index method. The stage-area rating is used to obtain a rated area for a given stage at the stream gauging station. It should be developed based on data collected at a fixed cross-section in the stream (also referred to as the standard cross-section). The standard cross-section should be located near the velocity-index gauge, if possible, and it should be a reasonably stable cross section. Periodic surveys of the standard cross section should be made to check for changes in channel geometry and therefore the stage-area rating.

Several different instruments may be used for surveying the standard cross-section, including:

(a) Level and stadia rod;
(b) Tagline;
(c) Depth soundings with a sounding rod or sounding weight;
(d) Echo sounder with surveying software;
(e) Acoustic Doppler Current Profiler (ADCP).

ADCPs are convenient for surveying channel depths, however they were not designed as echo sounders and can be difficult to calibrate for precise depth soundings. Use a tagline if possible, to provide transects that are approximately perpendicular to the flow. Make the ADCP transect straight across the channel. Calibrated echo sounders used in conjunction with differential GPS and hydrographic surveying software are useful for obtaining accurate channel geometry.

Important considerations include the following:

(a) The cross-section must be referenced to stage at gage datum;
(b) Make sure that the survey extends above the maximum expected stage;
(c) Periodically check for channel changes – even upstream or downstream of the standard cross section. When computing mean channel velocity (V) for a discharge measurement for development or checking of a velocity-index rating, always compute V using the area from the stage-area rating.

Stage-area ratings can also be developed by plotting stage versus area and then fitting a curve to the stage-area points. The curve can be fitted by multiple or curvilinear regression techniques or drawn by eye. The resulting stage-area rating curve can be represented in a discharge processing program as a table or an equation of the curve.

2.4 VELOCITY INDEX RATING DEVELOPMENT

A velocity-index rating represents the relation between the velocity-index and concurrent mean velocity of discharge in the standard cross section. It is important to note that the mean velocity of the discharge measurement is computed using the cross-section area from the standard cross section, and not the cross-section area of the discharge measurement. In the following paragraphs this will be referred to as mean velocity. Also note that in the following paragraphs the ADVM is referred to as the instrument for measuring the velocity-index. This is just for convenience. The velocity-index can be obtained from other instruments such as the AVM or velocity profiler.

A number of discharge measurements throughout the expected range in stage are required to develop a velocity-index rating. This rating provides the method for computing mean velocities from the index velocities recorded at a station.

Various methods can be used to develop a velocity-index rating. For example, a velocity-index rating could be a single coefficient to relate velocity-index to mean velocity, provided the range in stage at the station is not large. Other ratings could be more complex, particularly at stations with bidirectional flow or a large range of stage. For stations with a large range of stage, stage may be a factor in the computation of mean velocities from index velocities. The velocity-index rating must be developed individually for each station based on measured data.

Common practice in developing velocity-index ratings is to plot the mean velocity and ADVM velocity-index from a series of measurements on an x-y plot, where the y-axis represents mean velocity and the x-axis represents ADVM velocity-index. This plot is the start of the analysis of the relation between mean velocity and ADVM velocity-index. With this plot and knowledge of the hydraulics at the station, a velocity-index rating can be developed. For some stations, the relation may be linear. For others, the relation may best be described as curvilinear or as a compound curve. The relation between ADVM velocity-index and mean stream velocity can be used simply as a graphical rating or as a tabular rating.

If the relation between mean velocity and ADVM velocity-index is linear, it can be defined by a linear equation as shown in equation 2.1:

$$V = C_1 V_i + C_2 \tag{2.1}$$

where V is the mean velocity at the standard cross section; V_i is the velocity-index measured by the ADVM, and C_1 and C_2 are constants.

If the relation between mean velocity and ADVM velocity-index is curvilinear, a second order equation of the following form may be used:

$$V = C_3 V_i + C_4 V_i^2 + C_5 \tag{2.2}$$

where C_3, C_4, and C_5 are constants.

If enough data are available equation 2.2 can be defined by a least squares solution. In some cases, stage may also be a factor in the relation between ADVM velocity-index and mean velocity. If this is the case, a multiple linear regression can be used to define the relation. However, a considerable amount of data will be required throughout the full range of stage and velocity to define a significant equation. The equation for a multiple linear regression equation where velocity index (V_i) and stage (S) are independent variables will take the form shown in equation 2.3:

$$V = C_6 V_i + C_7 S V_i + C_8 \tag{2.3}$$

where C_6, C_7, and C_8 are constants.

The basic process of developing the rating can be summarized as follows:

(a) Create a table summarizing the measurement data, including measurement number, date, mean gage height, discharge, the rated area (A) from the stage-area rating, mean velocity (V), and velocity index (V_i) for each measurement and any remarks. A computer spreadsheet program is a convenient tool for listing, plotting and analyzing the data;

(b) Plot the data. Create plots of mean channel velocity (V) and the velocity index (V_i) for evaluating the velocity-index relation for linearity. Plot V on the ordinate and V_i on the abscissa. If the only independent variable required to accurately compute mean velocity is velocity-index, a single parameter equation such as equation 2.1 may be used. If the velocity-index relation is not linear then a graphical or tabular method may be used to compute mean velocity;

(c) If the plot indicates that stage (*S*) or other variable might be a significant variable, then least-squares *multiple* linear regression can be used to define the equation, similar to equation 2.3 (Sloat and Gain, 1995). Additionally, the residuals (unexplained error) from the resulting regression equation can be evaluated to determine if a significant relation exists between the response variable (mean velocity) and independent variable(s) and if the response variable is adequately estimated. Least-squares multiple linear regression and the analysis of residuals are described, for instance, by Draper and Smith (1982);

(d) Assess the linear regressions defined in steps 2 or 3 using the following:
   (i) *Correlation coefficient (R-square)*. The R-square should be about 0.95 or greater. If R-square is approximately 0.90 or less, determine whether a multi-parameter velocity-index rating with V_i and stage (*S*) improves the velocity-index rating;
   (ii) *Standard error (SE)*. The SE is an overall indicator of the error of the velocity-index rating. The lower the SE, the more accurate the rating;
   (iii) *Residual plots*. Plots of the regression residuals should be approximately random, indicating no patterns or trends. Outliers on these plots should be checked carefully for measurement errors or irregularities in velocity-index data.

(e) At the stage (water surface elevation) where flow goes over bank, the rating will likely change. In this case a compound rating may be necessary (that is one equation for flows within the channel and a separate equation for over bank flow). It will likely be necessary to input such a rating in tabular form in discharge processing software.

Experience has shown that the single or multiple linear relations are often applicable for velocity-index ratings. Velocity-index ratings documented in Sloat and Gain (1995) and Morlock and others (2002) were all linear. Ruhl and Simpson (2005) stated that a wide range of relations have been developed in the San Francisco Bay and Delta region most of which are linear (Figure II.2.1(a)). However, Ruhl and Simpson (2005) found that more complex ratings also are possible. They documented several higher-order polynomial ratings (Figure II.2.1(b)); loop ratings that are indicative of ebb-flood asymmetries in the current structures at the measurement location causing a different relation between the flood-to-ebb transition versus the ebb-to-flood transition (Figure II.2.1(c)) and occasionally a bimodal relation (Figure II.2.1(d)).

While a single parameter linear velocity-index rating would be easy to input as an equation in a discharge processing program, multiple parameter linear ratings or complex ratings such as those described by Ruhl and Simpson (2005) may present a greater challenge. In these instances ratings might be input by using tables, approximated using simpler equations or processed outside of the processing program using spreadsheets or mathematical processing programs. The output of these programs, mean velocity, could then be input into the discharge processing software so that discharge can be computed.

2.5 DISCHARGE COMPUTATION

2.5.1 General

Discharge (*Q*) is computed from the equation $Q = VA$ in the following manner:

(a) Recorded values of stage are quality assured and appropriate corrections are made as documented in Section 6.5 of this Manual. The computed stages are then input to the stage-area rating to compute area, *A*;

(b) Recorded values of velocity-index are quality assured, appropriate corrections are made, and the values are then input to the velocity-index rating (stage may also be a input parameter for multiple parameter ratings). The output from the velocity-index rating is mean velocity, *V*;

(c) For each recorded (unit) value of stage and corresponding velocity-index, a computed unit value discharge, *Q*, is produced by multiplying the area, *A*, times the mean velocity, *V*. Daily-mean discharges are computed from the mean of the unit value discharges in a one day period.

The records of stage and velocity data are verified and edited as described in Section 6.5 of this Manual. Unit value plots are very useful for analyzing the velocity data, as are various instrument quality assurance parameters such as ADVM signal strength. The instrument quality assurance parameters vary from instrument to instrument; the instrument manuals should be consulted to determine the availability and use of quality assurance and diagnostic parameters.

Common experience has found that it is usually difficult or impossible to correct velocity data from

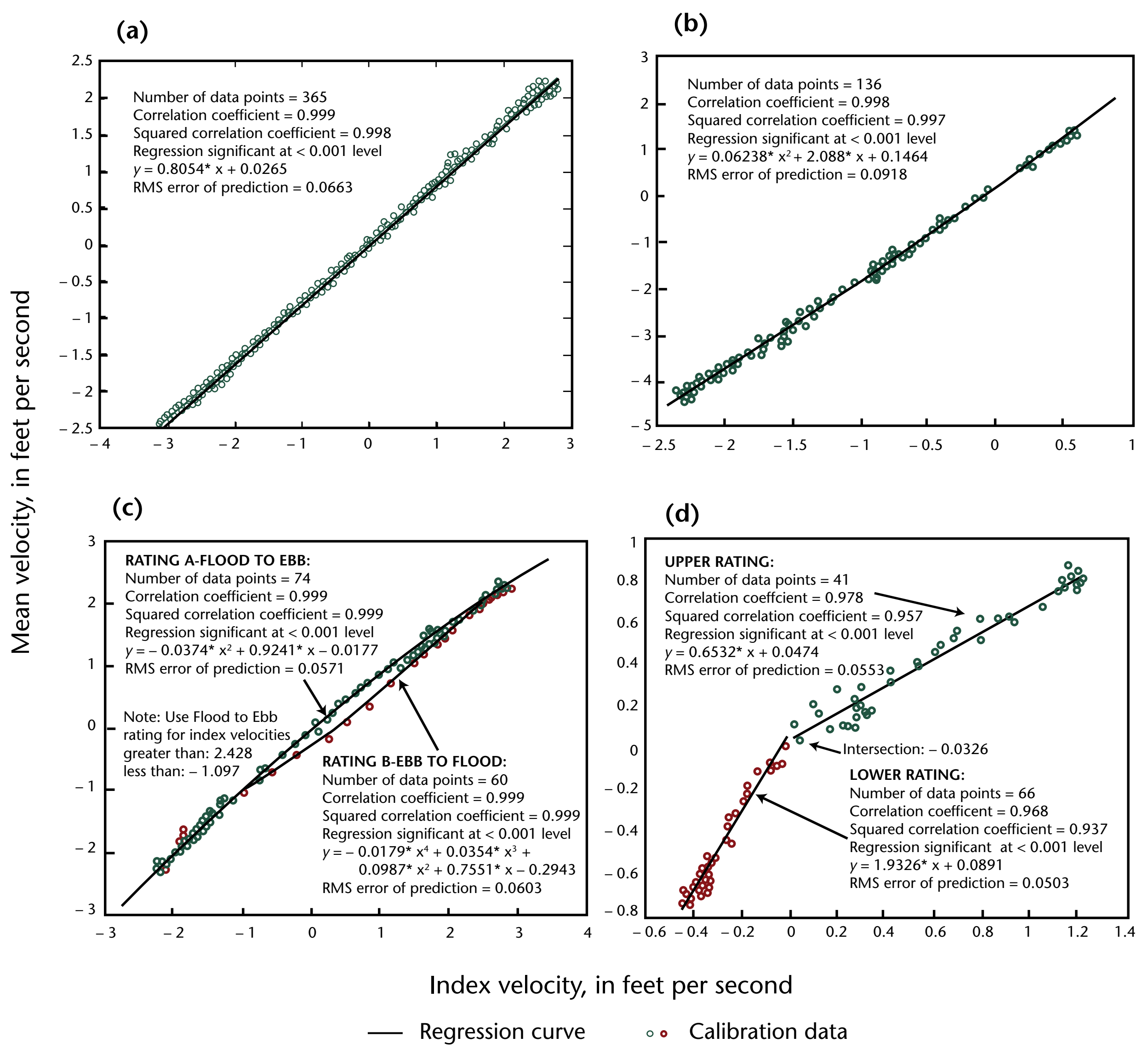

Figure II.2.1. Examples of velocity-index versus mean velocity ratings: (a) simple linear rating; (b) quadratic rating; (c) loop rating and (d) bi-modal rating

ADVMs. A common example is a low-velocity bias caused by a fixed object in the ADVM sample volume. The resultant velocity error is not usually a consistent bias error that could be fixed with the application of a simple velocity correction or shift, but instead the error shows a random pattern. Shift applications are however possible for the velocity-index method. For example, a change in channel geometry can change the stage-area and velocity-index ratings. If this condition is temporary, it can be handled by a shift adjustment. It is common practice to represent the adjustment as a velocity shift (rather than a stage shift that would cause a shifted area computation). The shift used in this example should only be used until the channel is resurveyed at the standard cross section. At that time, a new stage area rating and velocity-index rating should be developed. Section 6.10 discusses the application of shifts for velocity-index methods.

2.5.2 Mean discharge at tidally affected sites

Velocity-index rating methods are often used at coastal gages where flow is affected by tidal fluctuations. Individual unit values of discharge are computed just as described in the preceding paragraphs. However, tidal affects need to be taken into account when computing mean daily discharges. Ruhl and Simpson (2005) discuss these considerations for several stations in the San Francisco Bay and

Delta, United States of America where the velocity-index method was used to compute discharges at tidally affected gauges.

Calculating daily discharge in a tidally influenced environment cannot be accomplished simply by averaging all of the values collected during a 24-hour period. Simple averaging causes cyclical variations, or aliasing, in the data that are spurious and are a function of the averaging scheme, not the data. Therefore, a low-pass filter is used to remove frequencies that have periods less than 30 hours. The most energetic variations removed in this process are the astronomical tides (typically with periods at or around 12 and 24 hours); however, other variations (meteorological, hydrologic or operational) that have periods less than 30 hours also are removed. A number of filters are available including the Godin filter (Godin, 1972), a Fourier transform filter (Walters and Heston, 1982; Burau and others, 1993) or a Butterworth filter (Roberts and Roberts, 1978). All of these filters have been used in the San Francisco Bay and Delta Program for a variety of purposes, however, published daily discharge values, as shown by the red lines in Figure II.2.2 are calculated using a Butterworth filter with a 30-hour stop period and a 40-hour pass period. Note that tidal variations with periods greater than 30 hours, such as spring/neap cycle effects, will remain in the resulting tidally averaged data (Roberts and Roberts, 1978). In addition, approximately 2 days of filtered data at the beginning and end of the time-series or adjacent to any gap in the time-series are erroneous due to filter ringing and are not used. The daily discharge is calculated as the 24-hour daily average of the tidally filtered data, as shown by the black line with open circles (Plot B) in Figure II.2.2.

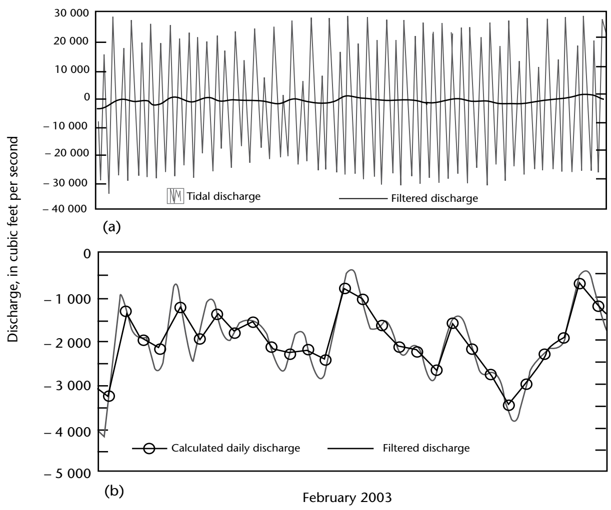

Figure II.2.2. Example of tidal discharge record. Plot (a) is tidal discharge (blue) and filtered discharge (red). Plot (b) is daily average discharge (black) and filtered discharge (red)

Note: These plots are from Ruhl and Simpson (2005) and are in English units.

2.5.3 Discharge records during ice-affected periods

During ice-affected periods, a simple stage-discharge rating often is not reliable because the ice will cause stages to rise without a corresponding rise in discharge. During such periods, the discharge record is usually estimated using methods described in Chapter 1. The estimation is aided if one or more discharge measurements were made during the period. Otherwise, comparisons with the hydrographs of other stations in conjunction with weather data must be used to estimate the record. Estimated records during ice periods are subjective and usually are rated poor.

ADVMs may allow for less-subjective estimates of discharge during ice-affected periods. The following example was taken from Morlock and others (2002). An ADVM discharge was computed during a discharge measurement made at the Iroquois River gauging station when the river was totally ice covered. The measurement yielded a discharge of 1.119 $m^3 s^{-1}$. The ADVM computed discharge during the measurement period was 1.444 $m^3 s^{-1}$, a difference of 29 per cent. Measurement notes indicated the average ice thickness was 0.15 m, and when holes were chopped in the ice to make the measurement, the water level came to the top of the ice. This result means that the river was under pressure and that the channel area used to compute the ADVM discharge was based on a stage that was about 0.15 m high. When the channel area was recomputed manually for a 0.15 m lower stage, the ADVM discharge for the measurement period was recomputed at 1.206 $m^3 s^{-1}$, 7.8 per cent higher than the measurement discharge. Because of the rough underside of the ice, the velocity profile usually is different than when the surface is free of ice. Rantz and others (1982) recommend that for velocities sampled at 60 per cent of the distance from the bottom of ice to streambed, a coefficient of 0.92 should be applied to estimate the mean velocity. By adjusting the ADVM measured velocity by multiplication of 0.92 a discharge of 1.110 $m^3 s^{-1}$ was computed, which is within 1 per cent of the measured discharge.

A more rigorous approach to estimates of discharge during ice-affected periods is described by Wang (2000). Flow-distribution models for deriving station-specific equations are used to compute discharges from AVM velocities during periods of channel ice cover. In this approach, bed-roughness and ice-roughness parameters are estimated from velocity profiles collected at a station. A hydraulic parameter is determined from cross-section area and locations of the AVM transducers. A beta coefficient is computed from the roughness and hydraulic parameters. The AVM velocity is multiplied by the beta coefficient which yields a discharge for a particular stage. The beta coefficient can be expressed as a function of stage through a regression analysis. Thus, discharge becomes a function of stage and AVM velocity. Discharges computed from this method compared closely to discharge measurements made at AVM stations in Canada during ice periods (Wang, 2000). Discharges computed from Wang's methods, although developed for AVM stations, can be applied directly to ADVMs as well. Whether using a simple coefficient or Wang's approach, the stage of the bottom of the ice needs to be known. One way to estimate the stage of the bottom of the ice is to use an ADVM equipped with an upward looking stage transducer.

2.6 EXAMPLE ADVM VELOCITY-INDEX SITE

This example of an ADVM velocity-index station is based on a report by Morlock and others (2002) for the United States gauging station Iroquois River near Foresman, Indiana. Tables and illustrations were taken directly from Morlock's report which was published in English units of measurement, which are retained for this Manual. An Argonaut-SL ADVM was installed at the gauging station in September 1999. The range of expected flows would be contained by the main channel where the ADVM samples velocities. Figure II.2.3 shows a photograph of the gauging site and a sketch of the instrument layout. Selected river characteristics are given in Table II.2.1.

A custom mount for the ADVM was constructed and attached to a downstream highway bridge pier for protection from debris. The ADVM was mounted at a depth of about 1 metre (at median flow) on a galvanized steel and aluminum frame designed for strength and weather resistance. The mount was designed so that the ADVM could be pulled up for maintenance and was connected by cable to an EDL in the station instrument shelter. The cable allowed the ADVM to be interfaced for programming and allowed the EDL and ADVM to communicate, using

View, looking upstream

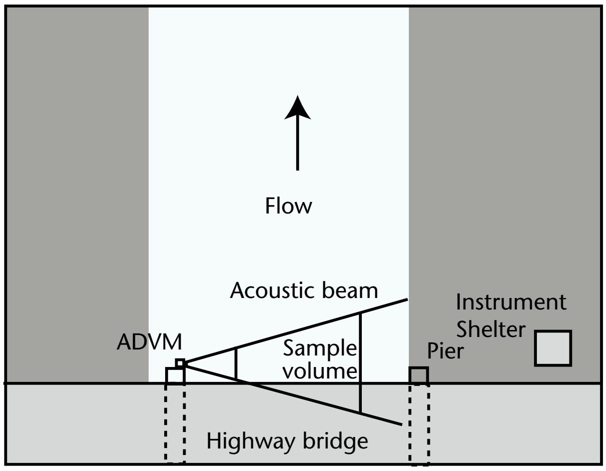

Figure II.2.3. Photograph and site sketch of Iroquois River near Foresman, Indiana, United States, ADVM velocity-index station

Table II.2.1. Selected river characteristics for the Iroquois River near Foresman, Indiana, United States

Mean discharge, $m^3 s^{-1}$	11.61
Median discharge, $m^3 s^{-1}$	5.61
Maximum discharge, $m^3 s^{-1}$	167.9
Discharge range, $m^3 s^{-1}$	167.8
Peak stage, m	6.4
Stage range, m	6.4
Channel width, m	23.8
Mean channel depth, m	1.829

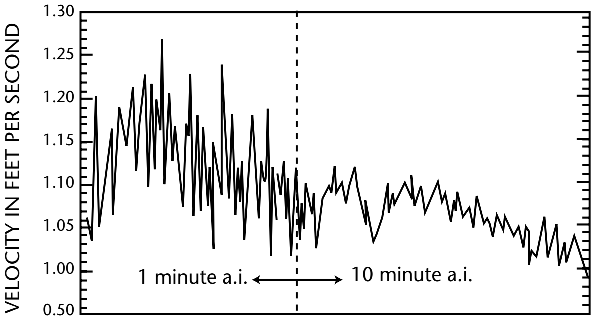

Figure II.2.4. ADVM velocity unit values showing the effect of increasing the ADVM averaging interval from 1 to 10 minutes

the SDI-12 communications protocol. The EDL also logged stage data from a separate stage sensor.

Following installation, the ADVM software was used to examine signal strengths for spikes so the sample volume end could be programmed. The sample volume or cell size was programmed so that no known obstacles were in the sample volume or cell and so that the end of the sample volume was positioned in such a way that signal strength was at least five counts above the instrument-noise level. The start and end distances for the sample volume, as measured from the ADVM transducer, was 1 and 8 metres, respectively. The start of the sample volume was beyond the estimated extent of the bridge-pier wake-turbulence zone.

After installation, the velocity-averaging interval was programmed at 1 minute. The EDL was programmed with a sampling interval of 15 minutes; that is, every 15 minutes the EDL would command the ADVM to sample. Upon completion of the ADVM-averaging interval, the EDL was programmed to log ADVM parameters such as velocities, beam amplitudes and quality indicators. Thus, an ADVM velocity was the 1-minute average velocity measured by the ADVM logged every 15 minutes.

The velocity-averaging interval was later increased from 1 to 10 minutes. This interval increased the time that velocities were being sampled from 7 to 67 per cent. Increasing the sampling time lowered random ADVM velocity variations from sample to sample by as much as 100 per cent, as shown in Figure II.2.4. Because velocity unit-value variations were reduced and velocities were being sampled a greater percentage of the time, velocity and discharge uncertainties from short-time scale fluctuations were reduced.

2.6.1 Development of stage-area rating

The stage-area rating was developed by first surveying the cross section at the downstream side of the bridge where the ADVM was attached. The cross section was a trapezoid in which all flows were contained. The cross-section survey was completed using a steel measurement tape referenced to the low chord of the bridge to measure elevations and a steel tape for distance measurement. The resulting surveyed cross section had an irregular bottom and sloping sides; from this surveyed cross section, the standard cross section was developed as shown in Figure II.2.5. A tabular rating was developed for a stage range of 2 to 24 ft at shown in Table II.2.2. Linear interpolation was used between the recorded values.

Table II.2.2. Stages and computed channel areas for the ADVM station, Iroquois River near Foresman, Indiana, United States

Stage m	Channel Area, m^2	Stage m	Channel Area, m^2	Stage m	Channel Area, m^2
0.610	3.99	3.048	54.53	5.486	138.4
0.914	8.45	3.353	63.17	5.791	151.2
1.219	13.47	3.658	72.37	6.096	164.6
1.524	19.04	3.962	82.12	6.401	178.5
1.829	25.08	4.267	92.34	6.716	192.9
2.134	31.68	4.572	103.0	7.010	207.8
2.438	38.74	4.877	114.4	7.315	223.2
2.743	46.36	5.182	126.2	–	–

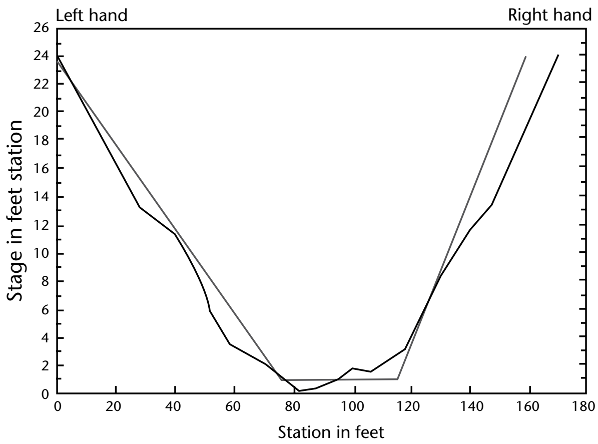

Figure II.2.5. Surveyed and standard cross sections for Iroquois River near Foresman, Indiana, United States

2.6.2 Development of velocity-index rating

Eighteen discharge measurements, 521 to 544, were made at the Iroquois River evaluation station. Fourteen of the measurements shown in Table II.2.3, were used to construct an index velocity rating. Measurement 523 was not used to construct the rating because the river was ice covered when the measurement was made; measurement 527 was not used because pertinent measurement data were not available at the time the rating was created. For the 14 measurements, stages ranged from 4.23 to 12.24 feet (1.289 to 3.731 metres) and measurement discharges ranged from 25.3 to 988 $ft^3\ s^{-1}$ (0.716 to 27.98 $m^3\ s^{-1}$). Mean channel velocities computed from the measurements ranged from 0.27 to 1.48 $ft\ s^{-1}$ (0.082 to 0.451 $m\ s^{-1}$). The Argonaut-SL x- component of velocity was used for development of the index velocity rating.

Plotting ADVM and mean velocities indicated that a well-defined linear relation between ADVM-measured velocities and mean velocities is present within the range of stages and discharges represented by measurements 521 to 536 (Figure II.2.6). As a result, the relation between ADVM-measured velocities and mean velocities could be represented by fitting a straight line to the data.

A linear regression was performed in which the 14 mean channel velocities, V, computed from the 14 discharge measurements were regressed against the corresponding ADVM index velocities, V_i. All measurements were weighted equally in the analysis. The equation of the line is:

$$V = 0.97V_i - 0.10 \tag{2.4}$$

The r^2 for the regression was 0.98, and the standard error was 0.05 $ft\ s^{-1}$, which is 5.9 per cent of the

Table II.2.3. Summary of discharge measurements for the Iroquois River near Foresman, Indiana, United States

Measurements Number	Date	Rated	Stage m	Discharge $m^3\ s^{-1}$	Mean Velocity $m\ s^{-1}$
521	15/10/1999	Fair	1.289	0.716	0.082
522	16/12/1999	Poor	1.676	2.688	0.155
523	2/2/2000	Fair	1.475	1.119	–
524	1/3/2000	Good	2.499	9.006	0.268
525	6/4/2000	Fair	1.753	3.172	0.192
526	24/4/1999	Good	3.264	16.029	0.268
527	9/5/2000	–	2.121	5.749	–
528	10/5/2000	Fair	3.280	20.305	0.399
529	10/5/2000	Fair	3.328	21.013	0.381
530	11/5/2000	Fair	3.359	19.767	0.351
531	11/5/2000	Fair	3.353	20.390	0.347
532	11/5/2000	Fair	3.331	18.266	0.335
533	11/5/2000	Fair	3.313	18.238	0.332
534	7/6/2000	Good	2.530	9.884	0.287
535	21/6/2000	Fair	3.511	27.980	0.451
536	26/6/2000	Fair	3.731	21.891	0.335

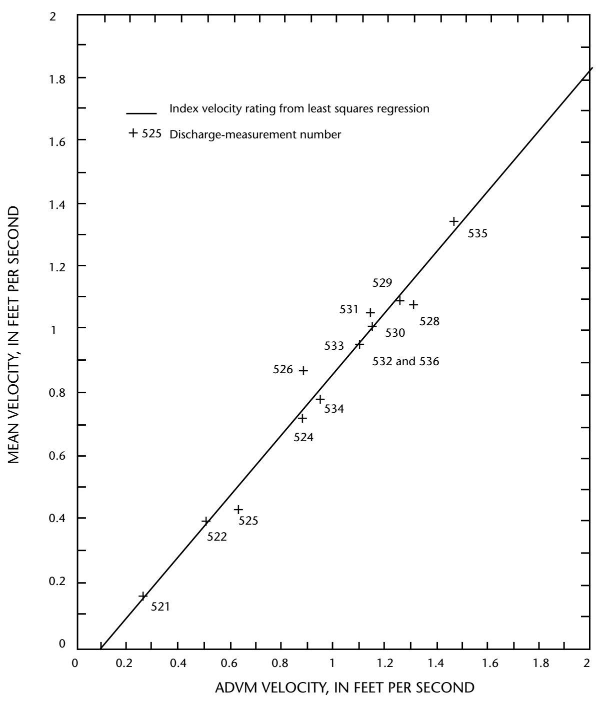

Figure II.2.6. Index velocity rating for an ADVM at gauging station for the Iroquois River near Foresman, Indiana, United States

average of the mean velocities (0.85 ft s^{-1}) computed from the 14 measurements used in the regression analysis. The r^2 of 0.98 indicated that 98 per cent of the variation in mean velocities was accounted for in variations of the ADVM velocities. Therefore it was assumed that, within the range of discharge measurements used to produce equation 2.4, stage was not a factor. The residual plot from the regression analysis did not indicate nonlinearity, non-constant variances, or large outliers. Equation 2.4 was found to be an adequate fit of the measurement data and became the velocity-index rating.

Computed discharges and measured discharges for the 14 discharge measurements used to develop the ratings are compared in Table II.2.4. Also, measured discharges and ADVM discharges for four discharge measurements (537 to 540) made after the ratings were developed are compared in Table II.2.4. Except for measurements 525 and 526, all 18 discharge measurements were within 10 per cent of the ADVM discharges.

2.7 VELOCITY INDEX ERROR SOURCES

As in any method of discharge computation, velocity-index methods have associated error sources that subsequently affect the quality of the discharges computed with the methods. Sloat and Gain, 1995 analyzed error for velocity-index methods and found that:

(a) Uncertainty in estimates of instantaneous and mean daily discharge is produced by random and systematic errors. Three principal sources of error in the estimated discharge can be identified:
   (i) Instrumental errors associated with measurement of area and velocity-index;
   (ii) Biases in the representation of mean daily stage and velocity due to natural variability in these over time and space;
   (iii) Errors in cross-sectional area and mean-velocity ratings based on stage and velocity-index;
(b) In practice, instrumental errors in stage and velocity measurements tend to be small and appear to be randomly distributed;
(c) Errors in sample representation tend to be periodic and may induce bias in discharge computations over short periods of time, but increasing the number of observations and the length of the computational period tend to improve representation;
(d) The errors in cross-sectional area ratings generally are relatively small because stage and cross-sectional area are relatively easy to measure and verify on a consistent basis;
(e) The largest single source of error remaining in discharge computations is uncertainty in the velocity-index rating.

Ruhl and Simpson (2005) discuss the importance of and potential errors associated with the collection of field data for velocity-index rating development. It is particularly critical in tidal systems where the tidally averaged flows are desired. Often tidally averaged flows are several orders of magnitude smaller than instantaneous tidal flows; therefore, a relatively small bias in the tidal flows can become a substantial error in the tidally averaged data. Clock synchronization, channel-bottom movement, boat positioning, discharge measurement duration (too fast or too slow), configuration file settings and equipment positioning all can affect the resulting data. Attention to detail is critical in minimizing problems during data collection.

Duncker and others (2006) applied a first-order error analysis to Acoustic Velocity Meter (AVM) data, stage-area, and velocity-index ratings at each gauging station. The error analysis results indicate that the uncertainty is sensitive to the value of uncertainty associated with Acoustic Doppler Current Profiler (ADCP) discharge measurement data. ADCPs are often used for velocity-index rating

Table II.2.4. Comparison of discharges computed from ADVM to measured discharges for the Iroquois River near Foresman, Indiana, United States

Discharge Measurement				ADVM Computed	Percent
Number	Date	Rated	Discharge $m^3 s^{-1}$	Discharge $m^3 s^{-1}$	Difference
521	15/10/1999	Fair	0.716	0.651	– 9.1
522	16/12/1999	Poor	2.688	2.625	–2.3
523	02/02/2000	Fair	1.119	–	–
524	01/03/2000	Good	9.006	9.176	1.9
525	06/04/2000	Fair	3.172	3.653	15.2
526	24/04/1999	Good	16.029	13.990	– 12.7
527	09/05/2000	–	5.749	–	–
528	10/05/2000	Fair	20.305	21.750	7.1
529	10/05/2000	Fair	21.013	21.212	0.9
530	11/05/2000	Fair	19.767	19.654	– 0.6
531	11/05/2000	Fair	20.390	19.371	– 5.0
532	11/05/2000	Fair	18.266	18.351	0.5
533	05/11/2000	Fair	18.238	18.380	0.8
534	07/06/2000	Good	9.884	10.110	2.3
535	21/06/2000	Fair	27.980	27.527	– 1.6
536	26/06/2000	Fair	21.891	21.920	0.1
537	12/07/2000	Fair	36.816	36.533	– 0.8
538	13/10/2000	Fair	1.020	1.048	2.8
539	15/11/2000	Fair	3.427	3.172	– 7.4
540	12/02/2001	Fair	59.189	59.189	0.0

development and verification because of the unsteady flow condition at many velocity-index sites. Duncker and others' findings emphasize the importance of following recommended procedures such as those documented in Volume I when making ADCP measurements.

Morlock and others (2002) provides a detailed discussion of stage-area and velocity-index ratings for three ADVM-equipped gauging stations, one of which is used in the preceding example. Ruhl and Simpson (2005) provide an excellent summary of all aspects of velocity-index rating development, through examples of real stations and data. Duncker and others (2006), Sloat and Gain (1995) and Ruhl and Simpson (2005) provide more detailed discussions of error analysis including equations for estimating errors.

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CHAPTER 3

DISCHARGE RATINGS USING SLOPE AS A PARAMETER

3.1 GENERAL CONSIDERATIONS

If variable backwater or highly unsteady flow exists at a gauging station, the energy slope is variable at a given stage and the discharge rating cannot be defined by stage alone. Variable backwater is most commonly caused by variable stage at a downstream confluence for a given discharge upstream, or by the manipulation of gates at a downstream dam. The discharge under those conditions is a function of both stage and the slope of the energy gradient. Where the rate of change of stage is sufficiently great, the acceleration head must also be considered, but this chapter deals only with situations where the acceleration head is insignificant and can be neglected.

The unsteady-flow situation treated in this chapter is that of a natural flood wave, in which the flow maintains a stable wave profile as it moves down the channel. That type of wave is known as a uniformly progressive wave and it often produces a loop rating at the gauging station. That is, for a given stage the discharge is greater when the stream is rising than it is when the stream is falling. The difference between the two discharges is significant only when the flow is highly unsteady. The term highly unsteady, when associated only with the property of producing loop ratings, is a relative term, because channel slope is of equal importance in determining whether or not loop ratings will occur. A flood wave in a steep mountain channel will have a simple stage-discharge relation. That same flood wave in a flat valley channel may have a loop rating. The sections of this chapter that deal with unsteady flow are concerned only with loop ratings whose definition requires the use of slope, as well as stage, in a relation with discharge.

When a new gauging station is established, the need for a slope parameter in the rating can often be anticipated from the rating procedures used for existing stations nearby in a similar hydrologic and hydraulic environment. At other times the need for a slope parameter is not as evident. However, a plot of a series of discharge measurements made at medium and high stages will indicate the type of rating required for the station and will dictate whether or not an auxiliary gauge is necessary to continuously measure water-surfaced slope.

If a pair of gauges is needed, the locations of the base and auxiliary gauges are based on the characteristics of the slope reach. The length of the reach should be such that ordinary errors that occur in the determination of gauge heights at stage stations will cause no more than minor error in computing the fall in the reach. A fall of about 0.15 m is desirable but satisfactory records can often be obtained in reaches where the minimum fall is considerably less than 0.15 m. Channel slope in the reach should be as uniform as possible. The reach should be as far upstream from the source of backwater as is practicable and inflow between the two gauges should be negligible. If possible, reaches with overbank flow should be avoided, as should reaches with sharp bends or unstable channel conditions. If the reach includes a natural control for low stages, the upstream (base) gauge should be located just upstream from that control so that a simple stage-discharge relation will apply at low stages. Rarely will a slope reach be found that has all of the above attributes, but they should be considered in making a selection from the reaches that are available for stage sites.

3.2 THEORETICAL CONSIDERATIONS

Variable slopes that affect flow in open channels are caused by variable backwater, changing discharge or variable backwater in conjunction with changing discharge. The pair of differential equations given below provides a general solution to both gradually varied and unsteady flow:

$$\frac{Q^2}{K^2} = -\frac{\partial H}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \tag{3.1}$$

$$\frac{Q}{x} = -B\frac{\partial h}{\partial t} \tag{3.2}$$

In the above equations, Q is the discharge; K is the conveyance of the cross section; H is the total energy head; x is the distance along the channel; g is the acceleration of gravity; V is the mean velocity; t is the time; B is the top width of the channel, and h is the water-surface elevation.

A solution to these equations in uniform channels may be obtained by approximate step methods after the conveyance term has been evaluated by discharge measurements.

In those practical problems of determining flow in open channels that require application of equation 3.1, the increment of slope due to the acceleration head:

$$\frac{1}{g}\frac{\partial V}{\partial t}$$

is, in general, so small with respect to the other two terms that its effect may be neglected. Thus, in equation 3.1, the terms that remain in addition to discharge Q, are conveyance K which is a function of stage, and energy gradient H/x which is related to water-surface slope. At those sites where tidal action or variation in power production cause the acceleration head to be large, an approximate method of integration of equations 3.1 and 3.2 is used in computer models of unsteady flow, such as those that are discussed in Chapter 4.

The discussion of stage-fall-discharge ratings presented in the following sections draws heavily on previously published reports. The three primary references used are Corbett and others (1945), Eisenlohr (1964), Mitchell (1954), Rantz (1982), Kennedy (1984) and ISO 9123 (2001).

3.3 VARIABLE SLOPE CAUSED BY VARIABLE BACKWATER

The stage at a gauging station for a given discharge, under the usual sub-critical flow conditions, is influenced by downstream control elements. The stage at the station attributable to the control elements is known to the hydrographer as backwater. As long as the control elements are stable, the backwater for a given discharge is unvarying, and the discharge is a function of stage only. The slope of the water surface at that stage is also unvarying. If some of the control elements are variable, such as movable gates at a downstream dam or the varying stage at a downstream stream confluence, then for any given discharge the stage at the station and the slope are likewise variable. In the preceding section it was demonstrated that for the above variable conditions, discharge can be related to slope and stage. Because the slope between two fixed points is measured by the fall between those points, it is more convenient to express discharge as a function of stage and fall.

Stage-fall-discharge ratings are usually determined empirically from observations of (a) discharge, (b) stage at the base gauge, which is usually the upstream gauge and (c) the fall of the water surface between the base gauge and an auxiliary gauge. The general procedure used in developing the ratings is as follows:

(a) A base relation between stage and discharge for uniform flow or a fixed backwater condition is developed from the observations. The discharge from that relation is defined as the rating discharge, Q_r;

(b) The corresponding relation between stage and fall for conditions of uniform flow or fixed backwater is developed. The fall defined by this relation is defined as rating fall, F_r. Figure II.3.1 shows schematically three forms the stage fall relation may have;

(c) The ratios of discharge, Q_m, measured under conditions of variable backwater, to Q_r, are correlated with the ratios of the measured fall F_m to the rating fall F_r. Thus:

$$\frac{Q_m}{Q_r} = f\left(\frac{F_m}{F_r}\right) \tag{3.3}$$

The form of the relation depends primarily on the channel features that control the stage-discharge relation. The relation commonly takes the form:

$$\frac{Q_m}{Q_r} = \left(\frac{F_m}{F_r}\right)^n \tag{3.4}$$

where n varies from 0.4 to 0.6, the theoretical value of n being 0.5. Generally speaking, the stage-fall-discharge rating can be extrapolated with more confidence when equation 3.4 is used with the theoretical exponent of 0.5.

The fall between the base and auxiliary gauge sites, as determined from recorded stages at the two gauges, may not provide a true representation of the slope of the water surface between the two sites. That situation may result from the channel and gauging conditions that are described below.

First, the water surface in any reach affected by backwater is not a plane surface between points in the reach, as sinuosity of the channel will produce variations in the height of the water surface, both across and along the reach; variations in channel cross-section and the effects of backwater also tend to produce curvature of the water surface. The slope determined from observed differences in stages is that of a chord connecting the water-surface elevations at points at the ends of a reach. It may not represent the slope of the water surface at either end of the reach, but may be parallel to a line that is tangent to the water surface at some point in the reach.

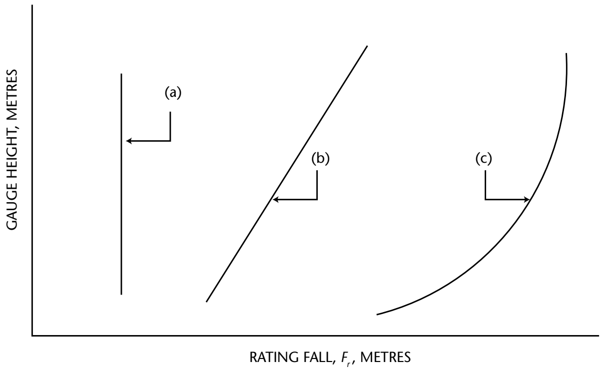

Figure II.3.1. Schematic representation of typical stage-fall relations. Curve (a), rating fall constant: curve (b), rating fall a linear function with stage: curve (c), rating fall a complex function of stage

Second, no reach of a natural stream selected for the determination of slope is completely uniform. The area of the cross section may vary considerably from point to point in the reach, but more important is the effect that shoals, riffles, rapids or bends in the stream channel within the reach may have on the slope of the water surface, as well as on the energy gradient.

Third, the positions of the gauges at the ends of the reach with respect to the physical features of the channel may have a material effect on the recorded gauge heights, and hence on the indicated slope. For example, if one gauge is on the inside of a rather sharp bend and the other on the outside of a similar bend, the slope computed from records of stages at those gauges may be widely different from the actual slope of the water surface. Also, if one gauge is attached to a bridge pier where drawdown around the pier affects the recorded stages in amounts that vary with the velocity past the pier, and the other gauge is on the bank or at a section of different velocity conditions, the records obtained from these gauges may not be true indices of the slope. There may be a drawdown effect on the intake pipe to a gauge well so that effects similar to those described above may be produced.

Fourth, both gauges may not be set to exactly the same datum, the difference in datum possibly being a large percentage of the total fall if the fall is very small. The slope determined from gauges not set to the same datum would not indicate the true water-surface slope but would include in it the quantity y/L, in which y is the difference in datum and L the length of the reach.

Because of those conditions, theoretical relations between stage, fall and discharge cannot be directly applied, and the relations must be empirically defined by discharge measurements made throughout the range of backwater conditions. Thus the best value of the exponent of F_m/F_r in equation 3.4 will often be in the range from 0.4 to 0.6, rather than having the theoretical value of 0.5. Sometimes it may even be necessary to depart from a pure exponential curve in order to fit the plotted points satisfactorily. At other times the substitution of a term $F + y$, for F values in equation 3.4 will improve the discharge relation. The use of a constant, y, whose best value is determined by trial computations, compensates in part for the inaccuracies in the value of F that were discussed above.

It is convenient to classify stage-fall-discharge ratings according to the type of relation that may be developed between stage and rating fall. The two types are:

(a) Constant-fall method:
This type of relation (curve *a* in Figure II.3.1) may be developed for channels which tend to be uniform in nature and for which the water-surface profile between gauges does not have appreciable curvature. A special case of the constant-fall method is the unit-fall method;

(b) Variable-fall method:
This type of relation (curves *b* and *c* in Figure II.3.1) may be developed where any of the following conditions exist:
   (i) Appreciable curvature occurs in the water-surface profile between gauges;
   (ii) The reach is non-uniform;
   (iii) A submerged section control exists in the reach between gauges, but the control does not become completely drowned by channel control even at high discharges;
   (iv) A combination of some of the conditions listed above.

It is not uncommon for variable backwater to be effective only part of the time. That follows from the two general principles that apply to backwater effect. First, for a given stage at the variable control element, backwater effect decreases at the base gauge as discharge increases. Second, for a given discharge, backwater effect decreases at the base gauge as stage decreases at the variable control element. Thus there are many possible combinations of stage at the variable control element and of discharge that will result in negligible backwater effect at the base gauge. For example, high flows may be free of variable backwater in a long gauging reach of relatively steep slope. On the other hand, low flows may be free of variable backwater if the upstream (base) gauge has a section control that is not submerged by low stages at the variable control element downstream.

Other basic principles and detailed procedures used in defining stage-fall-discharge ratings are discussed in the pages that follow. The discussions are arranged in accordance with the preceding classification of stage-fall-discharge relations. Knowledge of the hydraulic principles applicable to a given slope is essential as a guide to the empirical analyses of the data.

3.3.1 Constant-fall method

In uniform channels the water-surface profile is parallel to the bed. The water-surface slope, and thus, the fall between gauges, is the same for all discharges. The rating fall, F_r, for the condition of no variable backwater (open-water condition) would be the same at any stage. The stage-discharge relation with no backwater could be described by the Chezy equation:

$$Q_o = CA_o\sqrt{R_oS_o} \qquad (3.5)$$

where the subscripts denote uniform flow, or by:

$$Q_r = CA\sqrt{RF_r/L} \qquad (3.6)$$

where the subscripts denote the base rating conditions.

If variable backwater is imposed on the reach by a downstream tributary, the measured fall, F_m, and measured discharge, Q_m, would be less at a given stage than indicated by the open-water rating. If the slope or fall, as measured, truly represents the slope at the base gauge, those measurements would define, as shown in Figure II.3.2, a family of stage-discharge curves, each for a constant but different value of fall. The relation of each curve in the family to the curve for base rating conditions, according to equation 3.6, is expressed by:

$$\frac{Q}{Q_r} = \sqrt{\frac{F}{F_r}} \qquad (3.7)$$

The discharge under variable backwater conditions may be computed as the product of the discharge, Q_r, from the base rating and the square root of the ratio of the measured fall to the constant-value rating fall.

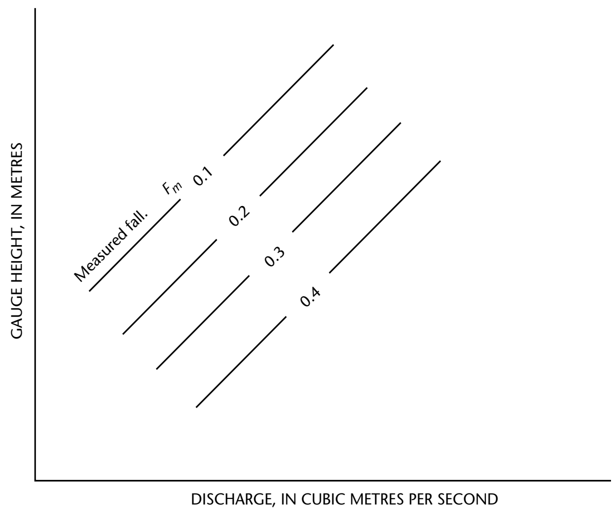

Figure II.3.2. Schematic representation of family of stage-discharge curves, each for a constant but different value of fall

A constant rating fall may also exist at sites where the base rating is controlled by a dam downstream from the reach in which fall is measured. If the curvature in the backwater profile is not significant and if the channel is uniform, the water-surface profile will be approximately parallel to the channel bed at all discharges. For example, the curve in Figure II.3.2 for a constant fall of 0.36 m may be taken to represent the base stage-discharge relation for a fixed or stable control element. The curves for lesser falls which might result from variable submergence of the dam are theoretically related to this base curve by the square root of the fall ratios as described above.

Quite commonly a constant value of 1.0 is used for F_r in equation 3.7. That special case of the constant-fall method is usually referred to as the unit-fall method, which simplifies the computations because equation 3.7 then reduces to:

$$Q_r = \frac{Q}{F^{1/2}} \qquad (3.8)$$

A constant rating fall is not the usual case encountered in natural streams. However, if discharge measurements cover the entire range of flow conditions and if such measurements conform to a constant rating fall, there is no need to use a more complicated technique. If profile curvature and velocity-head increments are truly negligible the relation between the discharge ratio and fall ratio should resolve into a single curve. Otherwise this relation may be a family of curves with stage as a third variable.

Procedure for establishing a constant-fall rating

The general procedure used in establishing a stage-fall-discharge rating with a constant rating fall is outlined as follows:

1. Plot all discharge measurements using stage at the base gauge as ordinate and discharges as abscissa, and note the measured fall, F_m, beside each plotted point. If the information on this plot indicates a family of curves, each corresponding to a constant value of fall (Figure II.3.2), the use of a constant rating fall should be investigated;
2. The most satisfactory type of constant-fall rating, from the standpoint of high-water extrapolation, is one whose discharge ratio-fall ratio relation is a pure parabolic relation, as in equation 3.7, with the exponent equal to, or nearly equal to 0.5. If such a relation fits the measured discharges, the results are unaffected by whatever value of constant fall, F_r is used. For convenience unit fall is used, as in equation 3.8;
3. For each discharge measurement, Q_m, compute Q_r by use of the equation:

   $$Q_r = Q_m / F_m^{0.5}$$
4. Plot values of gauge height versus Q_r for each discharge measurement and fit a curve to the plotted points to define the Q_r rating curve. Determine values of rating discharges from the Q_r rating curve;
5. Compute and tabulate the percentage departures of the plotted Q_r discharges from the Q_r rating curve;
6. Repeat steps 3-5, using exponents of F_m other than, but close to 0.5. Try exponents equal to 0.40, 0.45, 0.55 and 0.60;
7. Compare the five Q_r rating curves and select the curve that best fits the plotted points. In steps 8 and 9 that follow, the discharges from that best rating curve will be referred to as Q_{rd}, and the corresponding exponent of F_m will be referred to as d;
8. If the plotted discharges closely fit the Q_{rd} rating curve, that curve and the relation of Q_m/Q_{rd} to F_m are accepted for use;
9. If the plotted discharges do not closely fit the Q_{rd} rating curve repeat steps 3-5, using the exponent d but substituting the term $(F_m + y)$ for F_m. Several values of y, a small quantity that may be either positive or negative, are tried to obtain a Q_r rating curve that closely fits the plotted discharge;
10. Compare the various Q_r rating curves obtained from step 9 and select the curve that best fits the plotted points used to define it. If the plotted discharges closely fit that Q_r rating curve, that rating curve and the corresponding relation of (Q_m/Q_r) to $(F_m + y)$ are accepted for use. If the fit is not considered to be sufficiently close, the use of a pure parabolic relation, such as equation 3.8, is abandoned and the strictly empirical approach described in the following steps is used;
11. From the family of stage-discharge curves discussed in step 1, select one as the base Q_r curve and use the constant fall for this curve as F_r;
12. Compute the ratios Q_m/Q_r and F_m/F_r, plot the discharge ratios as ordinates and the fall ratios as abscissas and draw an average curve through the plotted points that passes through the point whose coordinates are (1.0 and 1.0);
13. Adjust each measured discharge by dividing it by the discharge ratio corresponding to the fall ratio on the above curve. Plot these computed values of Q_r against stage and draw an average curve (Q_r curve) through the points;
14. Repeat steps 11-13 using alternative constant values of F_r until the best relation between stage, fall and discharge is established;
15. If the best relation derived from the application of steps 11-14 is still unsatisfactory use the more flexible variable-fall method described subsequently.

Example of a constant-fall rating

A typical stage-fall-discharge rating is presented in Figure II.3.3 as an example of a rating with constant rating fall. The upper gauge is a water-stage recorder installed in a well attached to a pier of a highway bridge. The lower gauge is a water-stage recorder installed on the right bank 13 300 m below the upper gauge and 1 000 m above a dam.

The channel conditions in this reach are reasonably uniform. Variable backwater is caused by the operations at the dam.

A satisfactory relation between stage, fall and discharge could not be established for the upper (base) gauge by use of the procedure for a pure parabolic fall-ratio curve as described in steps 1-10 earlier. The empirical approach described in steps 11-14 was therefore used. The best rating was obtained by using a value of F_r equal to 0.457. The fall-ratio curve in Figure II.3.3 approximately fits equation 3.7 for all fall ratios no greater than 1.0. For fall ratios greater than 1.0 the curve is flatter than a parabola defined by equation 3.7.

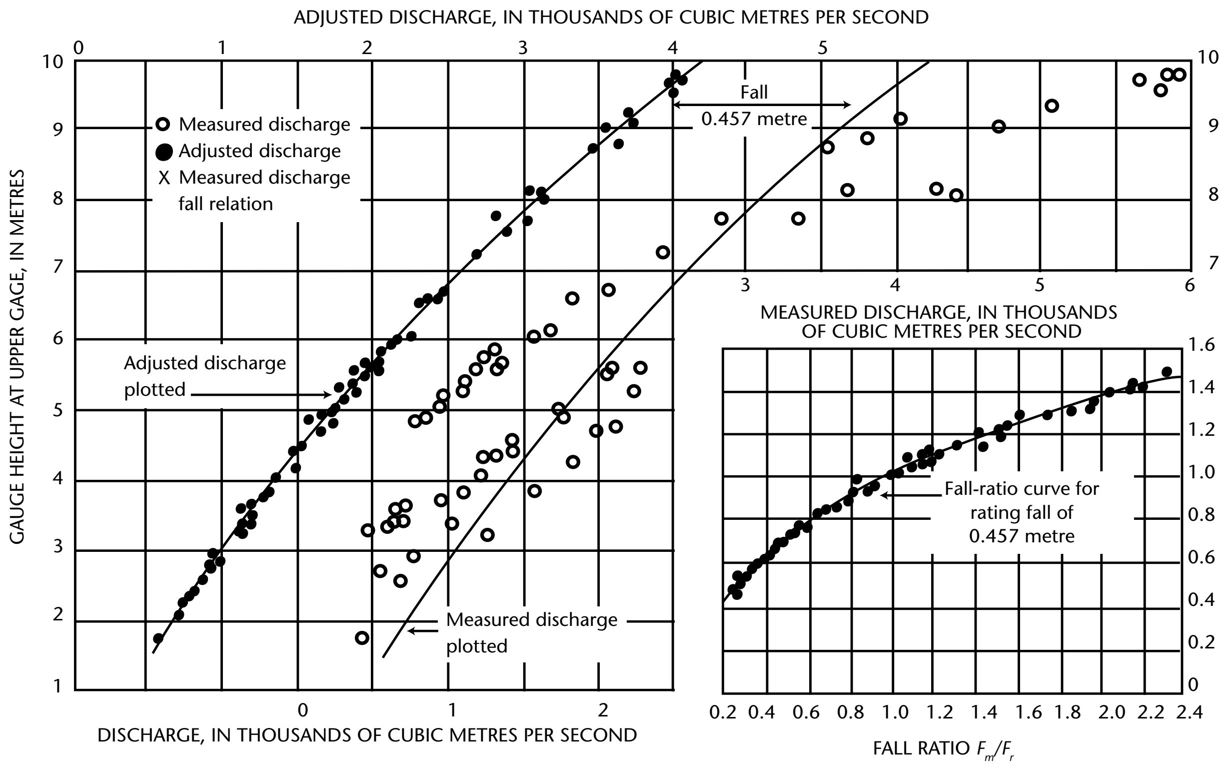

Figure II.3.3. Stage-fall-discharge relation for Tennessee River at Guntersville, Ala, United States

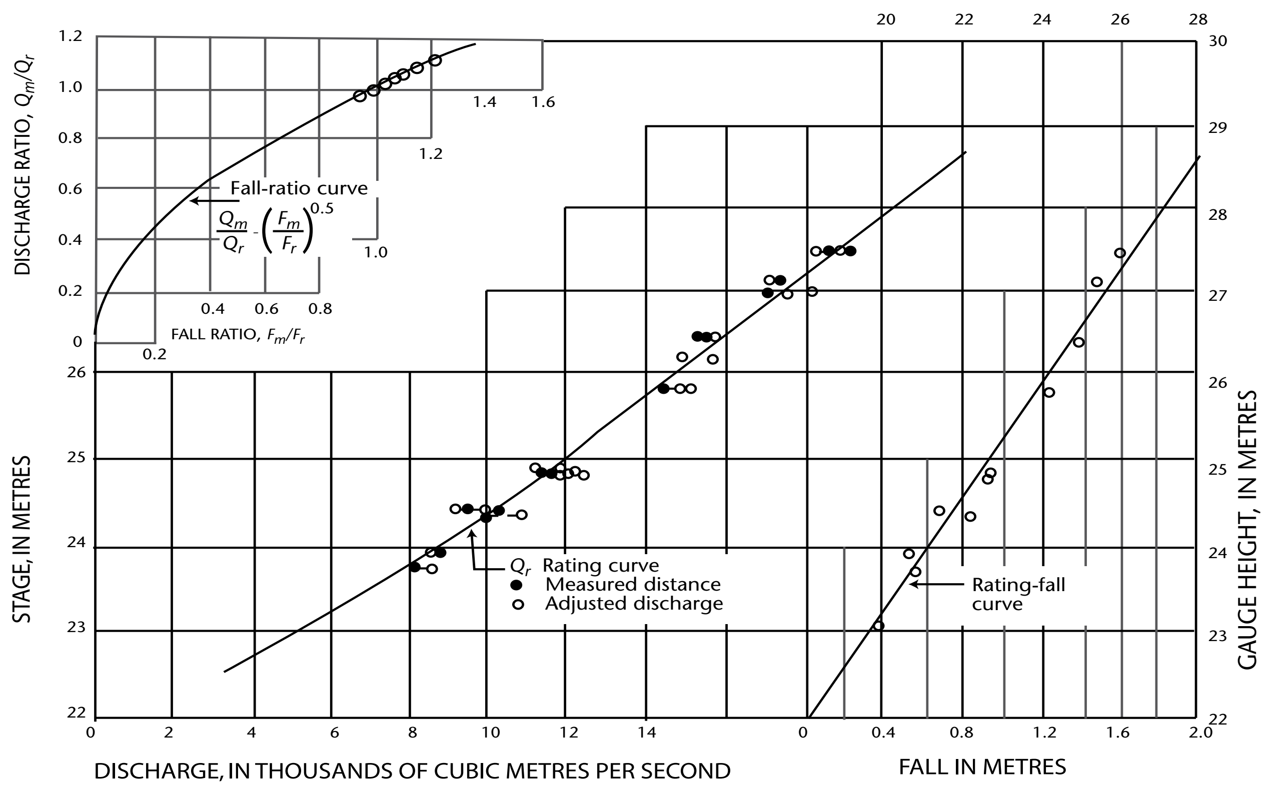

Figure II.3.4. Stage-fall-discharge relations for Columbia River at The Dalles, Oregon, United States

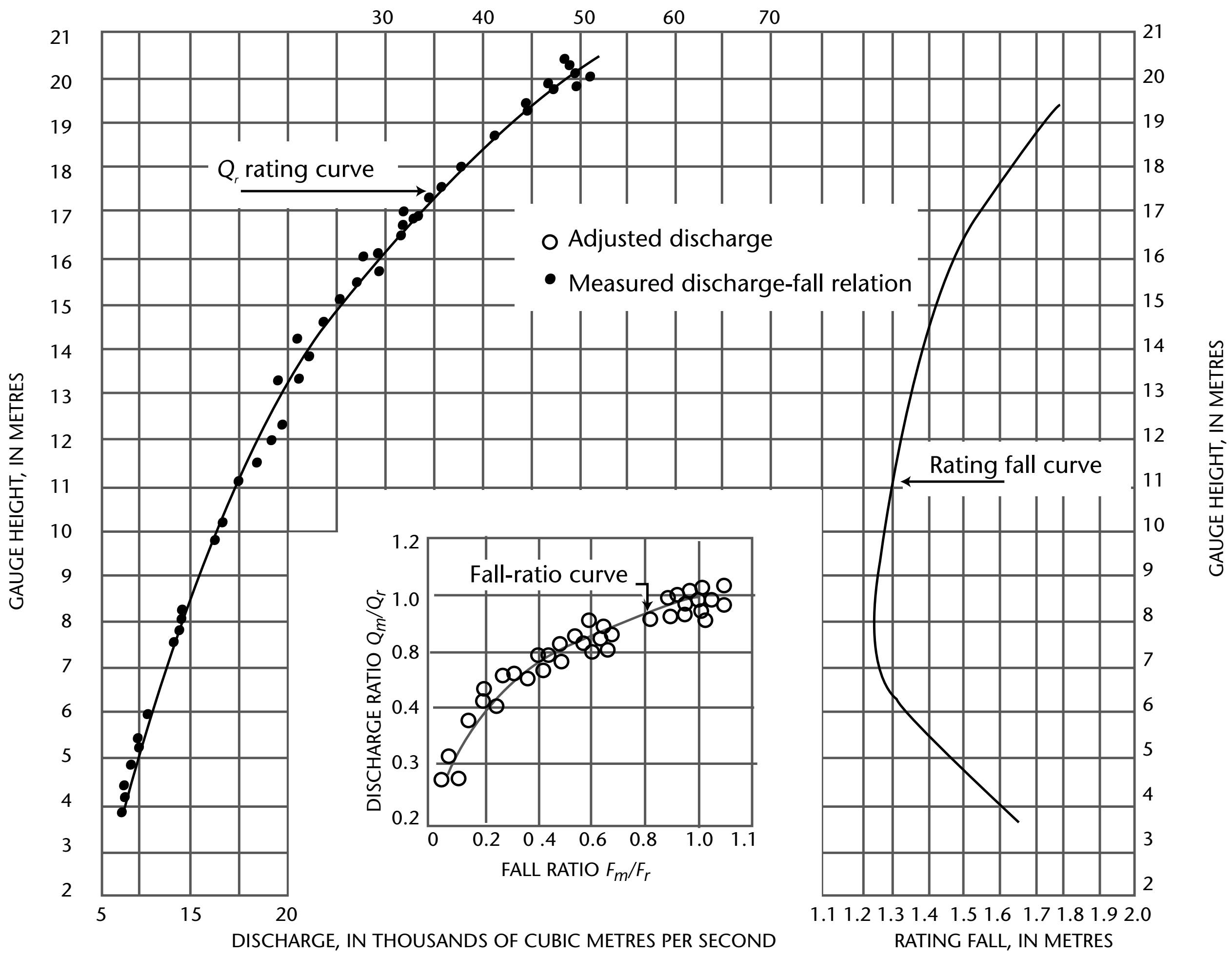

Figure II.3.5. Stage-fall-discharge relations for Ohio River at Metropolis, Illinois, United States (from Corbett et al.,1945)

To plot a rating discharge, Q_r, computed from a measured discharge, Q_m, and a measured fall, F_m, on the Q_r rating curve, the fall ratio, F_m/F_r or $F_m/0.457$ is first computed. The fall-ratio curve is then entered with the computed fall ratio, and the discharge ratio, Q_m/Q_r is read. Q_m is then divided by that value of the discharge ratio to give the value of Q_r to be plotted.

To obtain the discharge from Figure II.3.3 for a given gauge height and given fall F_m, the fall ratio, $F_m/0.457$, is first computed. The fall-ratio curve is then entered with the computed fall ratio and the discharge ratio, Q_m/Q_r, is read. From the rating curve, the value of Q_r corresponding to the given gauge height is read. The desired discharge, Q_m, is then obtained by multiplying Q_r by the discharge ratio.

3.3.2 **Variable fall method**

Where variable backwater is a factor in the discharge rating it will generally be found that fall is a function of stage. The average relation between fall and discharge may be linear or fall may be a complex function of stage. Rating principles are best discussed by reference to examples.

The right-hand graph in Figure II.3.4 is an example of a linear relation between stage and fall. The stage-discharge relation at the base gauge is affected by reservoir operations more than 130 km downstream. The auxiliary gauge is located 30 km downstream from the base gauge. Within the range of measured discharges, fall increases linearly with stage.

A much more complex stage-fall relation is shown in the right-hand graph in Figure II.3.5. At the downstream (auxiliary) gauge, the stage-discharge relation is affected only at the lower stages by a constriction, the backwater from which causes fall to decrease with stage in the slope reach. At the higher stages the constriction has little effect and fall increases with stage.

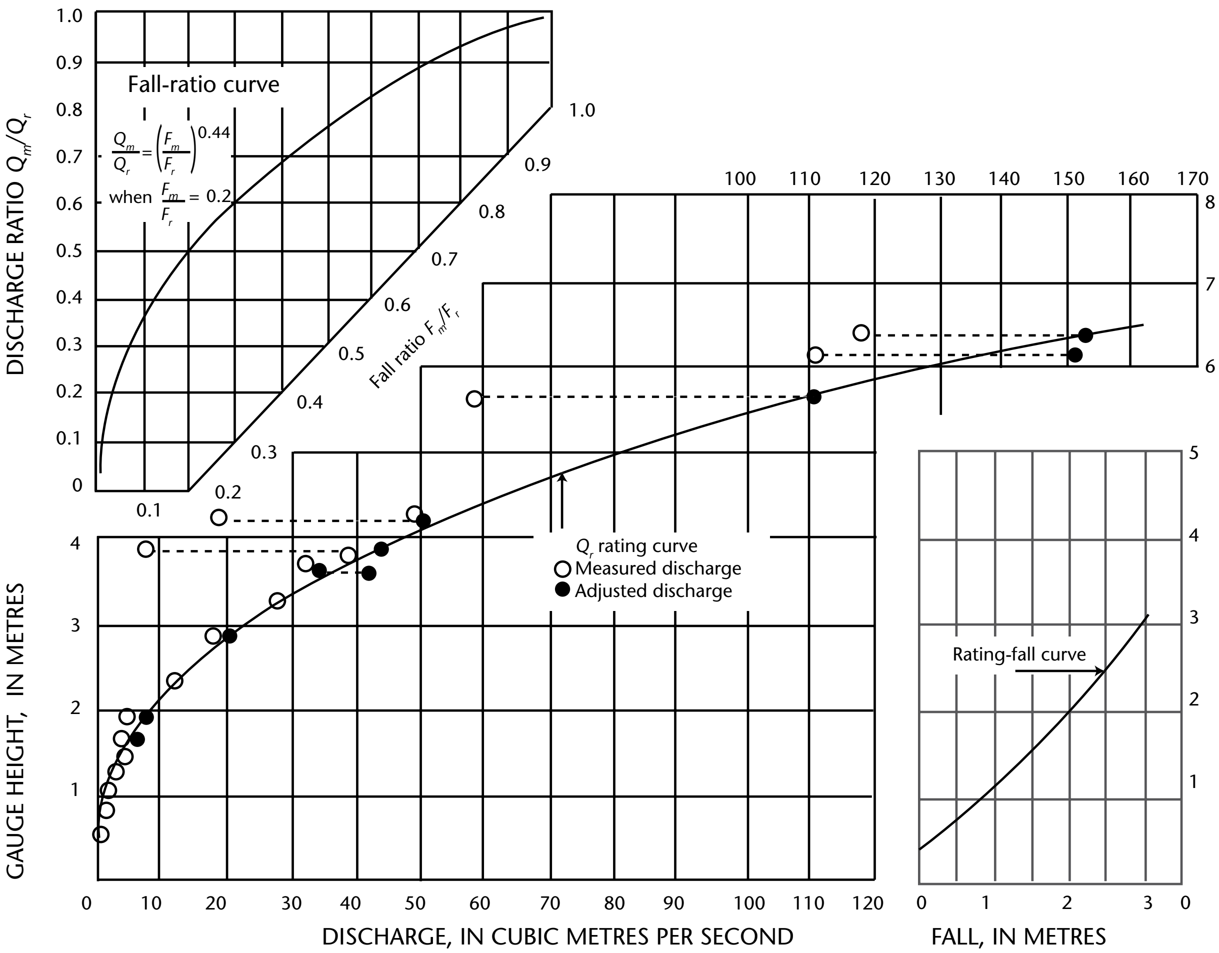

Figure II.3.6. Stage-fall-discharge relations for Kelly Bayou near Houston, Louisiana, United States

Another example of a complex stage-fall relation is shown in the right-hand graph in Figure II.3.6 for a tributary. The base gauge for this rating is about 4.3 km upstream from the mouth of the tributary. The auxiliary gauge is on the main river, 6.7 km downstream from the base gauge. At low stages, fall increases with stage; at medium and high stages the backwater effect from the main river is more pronounced and fall tends to assume a constant value.

Where a section control exists just downstream from the base gauge, it is necessary to identify those situations when backwater effect is absent at the base gauge. Obviously there will be no backwater when the tailwater at the section control is below the crest of the control. Most artificial controls are broad-crested and submergence is generally effective only when tailwater rises to a height above the crest that is equal to or greater than 0.7 times the head on the control. Looked at another way, submergence is effective only when the fall between the upstream and downstream stages is equal to or less than 0.3 times the head on the control. Thus a straight line of initial submergence may be drawn on the curve of stage versus fall. The line passes through the coordinates representing the elevation of the control crest and zero fall, with a slope of 1 m of stage per 0.3 m of fall. The precise position and slope of the line will depend on the location of the downstream gauge with respect to the section control. If the gauge is immediately downstream from the control, the line of initial submergence will have the position and slope stated above. If the gauge is far downstream from the control, the line on the stage-fall graph will intersect the elevation of the control crest at a value of fall greater than zero and the slope of the line will depend on the hydraulic features of the station. Field observation will be necessary to define the graph coordinates of the line of initial submergence. All observed or recorded values of fall that lie below the line of initial submergence indicate free-fall discharge (discharge unaffected by the tailwater elevation). All observed or recorded values of fall that lie above the

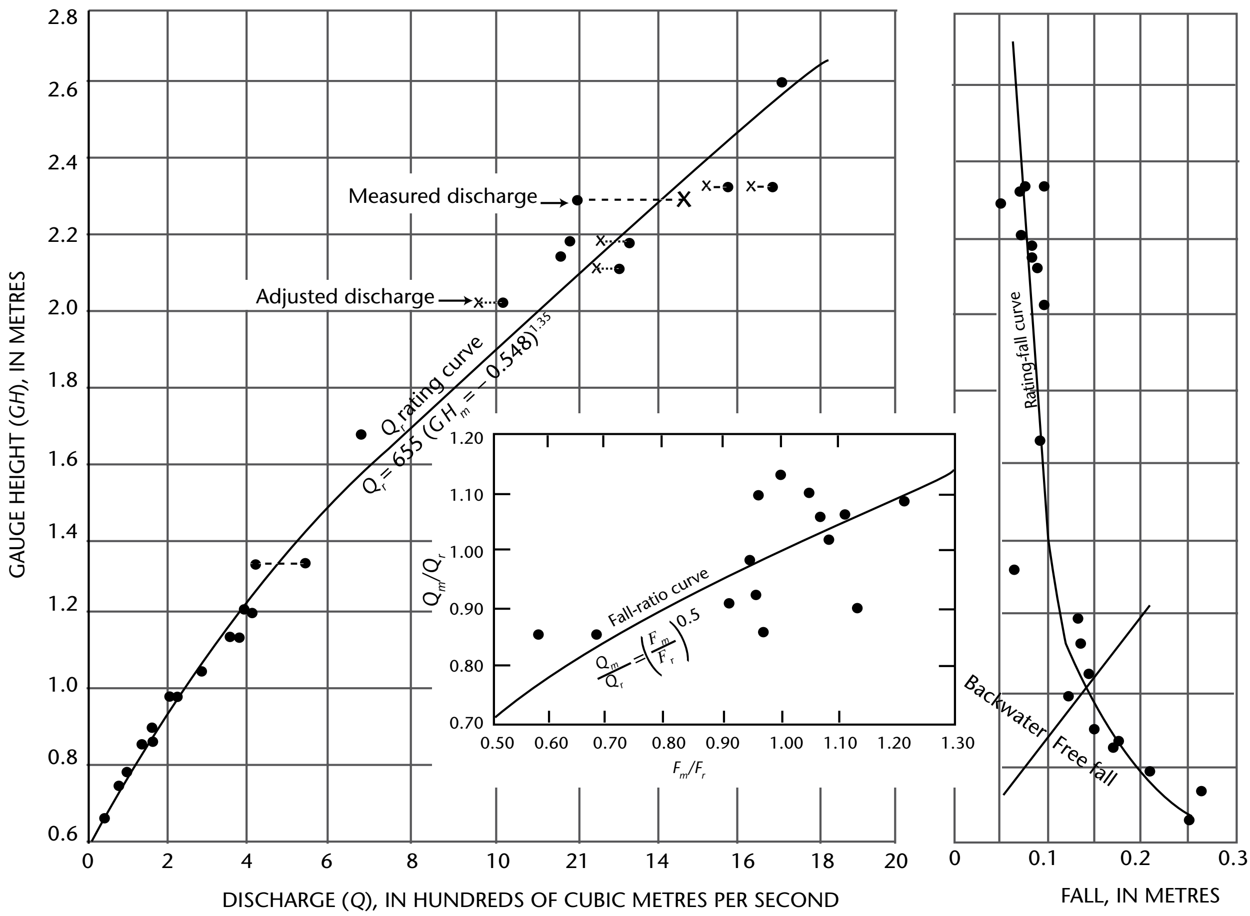

Figure II.3.7. Stage-fall-discharge relations for Colusa weir near Colusa, California, United States

line of initial submergence indicate discharge affected by variable backwater. Furthermore, if the tailwater gauge is close to the control, the fall-ratio curve for discharges affected by backwater should closely fit the theoretical equation 3.4. If the tailwater gauge is distant from the control, the fall-ratio curve will depart from the theoretical equation.

The right-hand graph in Figure II.3.7 shows a stage-fall relation where the base gauge for the station is a short distance from an ungated weir which acts as a section control, and the auxiliary gauge is a short distance downstream from the control. There is no pool immediately upstream from the weir, the streambed being at the elevation of the weir crest. There is a drop of about 0.6 m immediately downstream from the weir. The line of initial submergence shown crossing the lower part of the stage-fall relation has the theoretical position and slope discussed above. The weir is at the downstream end of a large natural detention basin along the left bank of river and water that passes over the weir immediately enters the river. Because the river stage rises faster than the stage of the detention pool, fall decreases with stage at the base gauge, as shown by the rating-fall curve.

The right-hand graph in Figure II.3.8 is a plot of stage versus fall for a Canadian river. The base gauge for this section is on the west arm of Kootenay Lake about 13 km upstream from Groham Narrows. Downstream from the narrows is the fore bay of the Corra Linn power plant, and in the fore bay is the auxiliary gauge, about 15 km downstream from the base gauge. Groham Narrows is the control for the base gauge, but operations of Corra Linn Dam cause variable submergence of the control when the stage of the fore bay is sufficiently high. The line of initial submergence, shown as the free-fall curve in Figure II.3.8, was determined from observation and discharge measurements. Discharge measurements whose values of fall plot below, or to the right of, the free-fall curve are unaffected by backwater and those discharges are therefore independent of fall. Discharge measurements whose values of fall plot above or to the left of, the free-fall curve are affected by variable backwater. For those measurements the graph shows no apparent relation between stage

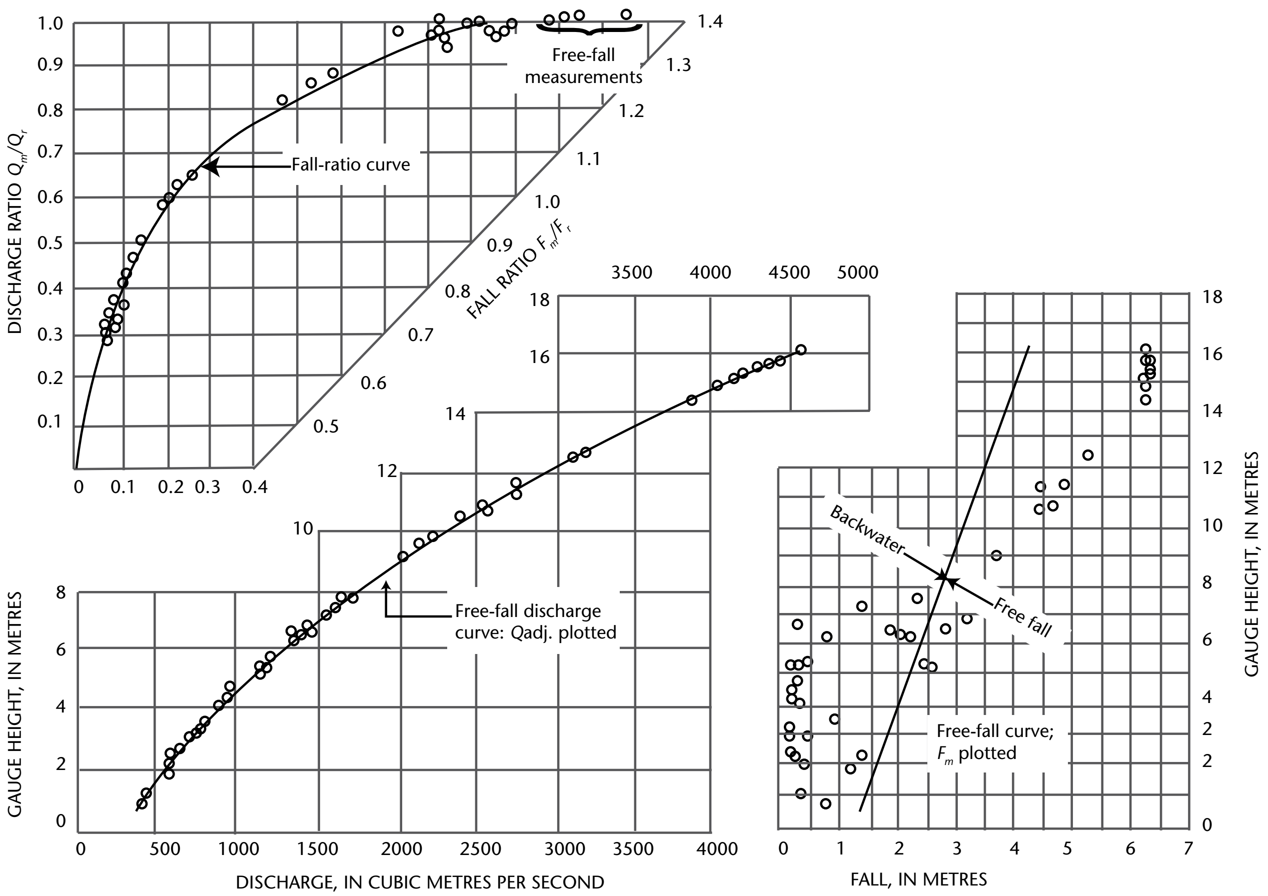

Figure II.3.8. Stage-fall-discharge relations for: Kootenay River, British Columbia, Canada. Measurements with falls less than 0.12 m not plotted, from Eisenlohr (1964)

and fall, and the free-fall curve (the line of initial submergence was used as the rating-fall curve for the measurements affected by variable backwater).

The rating for a gauging station whose base gauge has no section control is analyzed in a manner similar to that already described, the principal difference being that instead of using a constant value of rating fall, the rating fall for any stage is obtained from the rating-fall curve. The rating for a gauging station whose base gauge has a section control is analyzed in two separate steps. The free-fall part of the rating (no variable backwater) is analyzed as explained in Chapter 1, where simple stage-discharge relations are discussed. That part of the rating that is affected by variable backwater is analyzed as though no section control existed. It is not necessary to use the free-fall rating curve as the basis for establishing that part of the rating that is affected by variable backwater.

In view of the many different and complex situations that exist in natural channels, it is difficult to give general guidelines for establishing stage-fall-discharge relations. The analyst should make every effort to acquaint himself with the physical characteristics of the channel and the source of variable backwater. The best position of the relation curves that comprise the discharge rating must be determined by trial and error. The complexity of these relations determines to a large degree the number of discharge measurements necessary to define the discharge rating. Although the methods are empirical, experience has shown that there may be found a stage-discharge relation (the Q_r curve) which, taken in conjunction with its associated stage-fall relation (the rating-fall curve), will give close approximation to the true discharge under all possible combinations of stage and fall by the application of a single-curve relation, Q_m/Q_r vs F_m/F_r. It is desirable, but not always possible, to have that relation fit the theoretical equation 3.4.

Procedure for establishing the rating

The general procedure used in establishing a stage-fall-discharge rating with variable fall is outlined as follows:

1. Plot all discharge measurements using stages at the base gauge as ordinates and discharges, Q_m, as abscissa, and note the measured fall, F_m, beside each plotted point;
2. On another graph plot the measured fall, F_m, for each discharge measurement against stage at the base gauge, using stage as the ordinate;
3. If the base gauge has a section control, determine the position of the line of initial submergence on the plot of stage versus measured fall. Its position is based on discharge measurements known to have been made under conditions of free fall. Those measurements, plotted against stage on logarithmic graph paper, are fitted with a free-fall rating curve which is extrapolated in accordance with the principles discussed in Chapter 1. The remaining measurements are added to the logarithmic rating plot. Those measurements that plot to the left of the extrapolation are considered to be affected by backwater. That knowledge, along with knowledge of the probable degree of submergence required to cause backwater effect, enables the analyst to fix the position of the line of initial submergence. Only those measurements that plot above, or to the left of, the line of initial submergence are used in the analysis of the rating for variable backwater that is discussed in the steps that follow;
4. Fit a curve, (Q_r rating curve) to the stage-discharge plot in step 1, and another curve (F_r rating-fall curve) to the stage-fall plot in step 2;
5. From the curves in step 4 obtain values of Q_r and F_r corresponding to the stage of each discharge measurement;
6. Compute the ratios Q_m/Q_r and F_m/F_r for each discharge measurement;
7. Plot Q_m/Q_r as ordinate against F_m/F_r as abscissa, and on that graph draw the curve: $Q_m/Q_r = (F_m/F_r)^{0.5}$
8. On the basis of the scatter of the plotted points about the curve in step 7, adjust the Q_r and F_r curves (step 4) to obtain revised values of Q_r and F_r (step 5), such that the new ratios of Q_m/Q_r and F_m/F_r fit the curve in step 7 as closely as possible. The adjustments to the Q_r and F_r curve should not be so drastic that the adjusted curves are no longer smooth curves;
9. Repeat steps 4-8, using exponents of F_m/F_r other than, but close to 0.5. Try exponents equal to 0.40, 0.45, 0.55 and 0.60;
10. Compare the five plots of Q_m/Q_r versus F_m/F_r and select the one which shows the best fit between curve and plotted points. (The ratio of plotted values of Q_m/Q_r to curve values of Q_m/Q_r is identical with the ratio of measured discharge to discharge obtained from the stage-fall-discharge relations.) In steps 11 and 12 that follow, the exponent of that best fall-ratio curve will be referred to as n;
11. If the plotted ratios closely fit the curve $(Q_m/Q_r) = (F_m/F_r)^n$, that curve and the corresponding Q_r and F_r curves are accepted for use;
12. If the plotted ratios do not closely fit the curve $(Q_m/Q_r) = (F_m/F_r)^n$, repeat steps 4-8, using the exponent n but substituting the terms $(F_m + y)$ for F_m and $(F_r + y)$ for F_r. Several values of y, a small quantity that may be either positive or negative, are tried to obtain a close fit between plotted points and the curve defined by the equation:

$$\left(Q_m/Q_r\right) = \left[\left(F_m + y\right) / \left(F_r + y\right)\right]^n \tag{3.9}$$

13. Compare the various plots of the fall-ratio graph obtained from step 12 and select the one showing the best fit between curve and plotted points. If the fit is satisfactory, that curve and the corresponding Q_r and F_r curves are selected for use. If the fit is not considered to be sufficiently close, the use of a pure parabolic relation, such as the equation shown in step 11 or the equation shown in step 12, is abandoned and the strictly empirical approach described in the following steps is used;
14. Select one of the trial Q_r and F_r curves, such as were constructed in step 4, along with the corresponding values of Q_r, F_r, Q_m/Q_r and F_m/F_r, such as were obtained in steps 5 and 6;
15. Plot the discharge ratios as ordinates and the fall ratios as abscissas, and draw an average curve through the plotted points that passes through the point whose coordinates are (1.0, 1.0);
16. On the basis of the scatter of the plotted points about the curve in step 15, adjust the Q_r and F_r curves (step 14), as well as the fall-ratio curve. The adjusted curves must remain smooth curves;
17. Repeat steps 14-16, using other trial curves of Q_r, F_r and fall ratio versus discharge ratio, until the best relation is established between stage, fall and discharge. In other words, until a close fit is obtained between plotted points and the fall-ratio curve;
18. After having obtained acceptable Q_r, F_r, and fall-ratio curves, plot adjusted values of the discharge measurements on the Q_r rating curve. The adjusted values are computed as follows:
    (i) Given a measured discharge, Q_m, and a measured fall, F_m, enter the F_r curve (stage-fall relation) with the gauge height of the discharge measurement and read F_r;

(ii) Compute the fall ratio, F_m/F_r, and enter the fall-ratio curve to obtain the discharge ratio, Q_m/Q_r;
(iii) Obtain the value Q_r to be plotted by dividing Q_m by Q_m/Q_r. The method of obtaining the discharge corresponding to a given gauge height and a given fall, F_m, is explained later.

Examples of rating procedure

Figures II.3.4 through II.3.8 are examples of stage-fall-discharge relations for slope stations where fall is a function of stage.

Figure II.3.4 shows that excellent results were achieved in the range of discharge that was measured. The linear trend of fall increasing with stage is clearly evident, and the fall-ratio curve not only is represented by the theoretical equation 3.7, but is closely fitted by the plotted points. Where the rating-fall curve (stage versus fall) is so well defined, the first estimate of the Q_r curve is usually made by the use of equation 3.7, in which Q_m would represent the measured discharges. The computed Q_r values for the discharge measurements would then be plotted against stage, and a curve fitted to the plotted points would represent the first trial Q_r curve.

Figure II.3.5 is an extremely complex example, as can be seen from the shape of the rating-fall curve. It is not surprising that the fall-ratio curve could not be expressed by a simple parabolic equation, such as equations 3.4 or 3.9.

Figure II.3.6 shows an example of a station where there is relatively minor effect from variable backwater at low stages. At medium and high stages, the variable stage of the main river causes variable backwater at the base gauge. The rating-fall used during high-water periods has the constant value of 10.0. The fall-ratio curve, for values of F_m/F_r greater than 0.2, has the equation:

$$\frac{Q_m}{Q_r} = \left(F_m / F_r\right)^{0.44} \tag{3.10}$$

Because the exponent 0.44 does not differ greatly from its theoretical value of 0.5, the Q_r rating curve can be extrapolated with some confidence.

Figure II.3.7 is an example of the stage-fall-discharge relation for a station whose base gauge has a section control. There is no backwater at low flow, as shown by the six discharge measurements that plot below the line of initial submergence on the graph of stage versus fall. The remaining 16 discharge measurements show the effect of variable backwater. While the fit of adjusted measured discharges to the Q_r rating curve is not completely satisfactory, there is some satisfaction to be derived from the facts that the equation of the fall-ratio curve is theoretically correct and the fall-ratio curve balances the plotted points.

Figure II.3.8 for a station on the Kootenay River is an example of the stage-fall-discharge relation for a station whose base gauge has a control that is unsubmerged at high stages. Of the 59 discharge measurements shown, 23 were made under free fall conditions. Those 23 measurements plot below, or to the right of, the line of initial submergence on the graph of stage versus fall. The remaining 36 discharge measurements are affected by variable backwater and were used in the stage-fall-discharge analysis. Because the line of initial submergence was used as F_r in the analysis, the value of F_m for any measurement affected by backwater is less than F_r. Consequently the fall-ratio curve fitted empirically to the plotted points and is not expressed by a simple parabolic relation such as equations 3.7 or 3.9.

Determination of discharge from relation for variable backwater

After the three necessary relations are available (a) stage versus rating fall, F_r, (b) stage versus rating discharge Q_r and (c) Q_m/Q_r versus F_m/F_r, then the determination of discharge, Q_m, for a given stage and a given fall, F_m, proceeds as follows:
(a) From a stage-fall table determine the rating fall, F_r, for the known stage;
(b) Compute the ratio F_m/F_r;
(c) From a table of discharge ratios, Q_m/Q_r, and fall ratios F_m/F_r, determine the value of the ratio Q_m/Q_r;
(d) From a stage-discharge table, determine the rating discharge, Q_r, for the known stage;
(e) Compute Q_m by multiplying the ratio Q_m/Q_r by the value of Q_r.

Much emphasis has been placed on obtaining a purely parabolic function, such as equations 3.7 or 3.9, for the relation between fall ratio and discharge ratio. Such a relation not only permits the analyst to extrapolate the Q_r curve with more confidence, but it also expedites the computation of discharge. For example equation 3.4 may be transposed to:

$$Q_m = \left(\frac{Q_r}{F_r^n}\right)\left(F_m^n\right) \tag{3.11}$$

Two tables can be prepared, one giving the values of the quantity Q_r / F_r^n corresponding to stage, and the other giving values of F_m^n corresponding to values of F_m. The discharge is then computed as the product of the two values picked from the tables. Equation 3.9 may be transposed in a similar way.

3.4 VARIABLE SLOPE CAUSED BY CHANGING DISCHARGE

3.4.1 Theoretical considerations

Where channel control is effective, the effect of changing discharge on a graph of the stage-discharge relation is such as to produce a loop curve as shown in Figure II.3.9, on which the discharge for a given stage is greater when the stream is rising than it is when the stream is falling. In other words, given a simple stage-discharge relation for steady flow – that is, a rating that averages all discharge measurements – it will be found that the measurements made on a rising stage plot to the right of the curve and those made on a falling stage plot to the left. The discharge measurements for individual flood waves will commonly describe individual loops in the rating. The departure of measurements from the rating curve for steady flow is of significant magnitude only if the slope of the stream is relatively flat and the rate of change of discharge is rapid. For gauging stations where this scatter of discharge measurements does occur, the discharge rating must be developed by the application of adjustment factors that relate steady flow to unsteady flow. Unsteady flow refers to discharge at a site that changes appreciably with time, as in the passage of a flood wave.

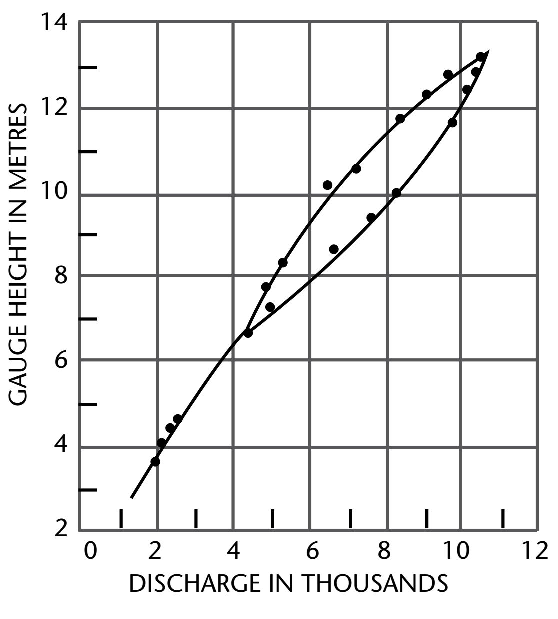

Figure II.3.9. Stage discharge loop of the Ohio River at Wheeling, West Virginia, United States, during the flood of 14 March 1905

The relation between the discharges for steady and unsteady conditions at the same stage can be derived from the general equations for unsteady flow (Rouse, 1950). A simplified equation shown below may also be derived by neglecting all terms due to change of velocity head or acceleration:

$$\frac{Q_m}{Q_c} = \sqrt{1 + \frac{1}{S_c v_w} \frac{dh}{dt}} \tag{3.12}$$

where Q_m is the discharge for unsteady flow; Q_c is the discharge for steady flow; S_c is the energy slope for steady flow at the same stage; v_w is the wave velocity, and *dh/dt* is the rate of change of stage with respect to time (*dh* is positive for rising stages).

Because equation 3.12 is basic to the methods commonly used for adjusting discharge ratings for the effect of changing discharge, it is appropriate to elaborate on its derivation. The ratio of the magnitudes of two discharges that occur at a given stage is equal to the ratio of the square roots of their energy slopes. That principle can be expressed in the following basic equation, which is similar to equation 3.12. that was used in preceding sections of the Manual:

$$\frac{Q_m}{Q_c} = \frac{\sqrt{S_m}}{\sqrt{S_c}} \tag{3.13}$$

where S_m is the energy slope for unsteady flow at the time of Q_m and the remaining terms are defined above for equation 3.12.

During changing discharge, the slope of the water surface increases or decreases by an increment of slope (ΔS), where:

$$\Delta S = \frac{1}{v_w} \frac{dh}{dt} \tag{3.14}$$

If it is assumed that the increment of slope by which the energy gradient changes is likewise equal to ΔS, then:

$$S_m = S_c + \Delta S = S_c + \frac{1}{v_w} \frac{dh}{dt} \tag{3.15}$$

By combining equations 3.13 and 3.15:

$$\frac{Q_m}{Q_c} = \left(\frac{S_c + \frac{1}{v_w}\frac{dh}{dt}}{S_c} \right)^{1/2} \quad (3.16)$$

or

$$\frac{Q_m}{Q_c} = \left(1 + \frac{1}{S_c v_w}\frac{dh}{dt} \right)^{1/2} \quad (3.17)$$

The wave velocity v_w in the above equations may be evaluated by the Sedden principle (Sedden, J.E., 1900):

$$v_w = \frac{1}{B}\frac{dQ}{dh} \quad (3.18)$$

where B is the width of the channel at the water surface, and dQ/dh is the slope of the stage-discharge curve for constant-flow conditions.

From examination of formulae for mean velocity in open channels the ratio of wave velocity to mean velocity may be shown to vary as follows:

Channel type	Ratio v_w/V_m	
	Manning	Chezy
Triangular	1.33	1.25
Wide rectangular	1.67	1.50
Wide parabolic	1.44	1.33

Experience seems to indicate that the most probable value of the ratio in natural channels is 1.3.

Equation 3.14 explains why the effect of changing discharge is significant only on flat streams during rapid changes in discharge because that combination is necessary to make ΔS significantly large. During rapid changes in discharge, absolute values (either plus or minus) of dh/dt are large. On flat streams wave velocity, v_w, is small. The combination of a large value of dh/dt and a small value of v_w results in a large value of ΔS.

3.4.2 Methods of rating adjustment for changing discharge

The four methods of adjusting discharge ratings for changing slope attributable to changing discharge are:

(a) Jones method;
(b) Boyer method;
(c) Lewis method;
(d) Wiggins method.

All four methods are based on equation 3.12, or on a modification of that equation. The Jones and Lewis methods are rarely used any more and will be only briefly described here. The Boyer and Wiggins methods are preferred for use and are described in detail.

The Jones method originally used water-surface slope rather than energy gradient for the term S_c in equation 3.12. Consequently an auxiliary stage gauge was required for measuring slope. A stage-slope relation for steady flow conditions was obtained from the recorded or observed stages at the base and auxiliary gauges. In later years a computed value of the energy gradient, S_c, was used in place of water-surface slope and an auxiliary stage gauge was no longer needed. Discharge measurements that had been made during periods of steady flow were used to evaluate S_c, using the Manning equation:

$$S_c = \left(\frac{Q_c}{K} \right)^2 \quad (3.19)$$

where K, the conveyance of the channel, is equal to:

$$\frac{1}{n} AR^{2/3}$$

Values of S_c were computed in that manner for several stages to provide the data needed for a relation of S_c to stage. The term v_w in equation 3.12 was computed for several stages by use of a complex empirical equation that included the term $1.3V_m$, to provide the data needed for a relation of v_w to stage. Equation 3.12 could then be solved for Q_c, which is the steady-flow discharge corresponding to the stage of Q_m, because Q_c then became the only unknown quantity in the equation. In a final step, the computed values of Q_c were used, with their corresponding stages, to construct the required steady-flow rating curve.

The Lewis method is a simplification of the Boyer method in which the term $1/S_c v_w$ in equation 3.12 is assumed to have a constant value J at all stages. That assumption reduces equation 3.12 to:

$$\frac{Q_m}{Q_c} = \left(1 + J\frac{dh}{dt} \right)^{1/2} \quad (3.20)$$

where Q_m and dh/dt are measured or observed quantities and, Q_c and J must be determined by trial-and-error computations. For that determination a trial Q_c rating curve is drawn on the basis of a plot of measured discharge, Q_m, versus stage, the positioning of the curve being influenced also by the value of dh/dt for each discharge measurement. The Q_c curve is then adjusted by trial and error until the values of J, computed by use of equation 3.20, approximate or scatter about a constant value. The Lewis method is satisfactory only for those gauging stations were the term $1/(S_c v_w)$ varies little with stage.

Boyer method

The Boyer method provides a solution of equation 3.12 without the necessity for individual evaluation of v_w and S_c. The method requires numerous discharge measurements made under the conditions of rising and falling stage. Measured discharge, Q_m, is plotted against stage in the usual manner, and beside each plotted point is noted the value of dh/dt for the measurement. For convenience dh/dt is expressed in feet or metres per hour and the algebraic sign of dh/dt is included in the notation – plus for rising stage and minus for falling stage. A trial Q_c rating curve, representing the steady-flow condition where dh/dt equals zero, is fitted to the plotted discharge measurements, its position being influenced by the values dh/dt noted for the plotted points. Values of Q_c from the curve corresponding to the stage of each discharge measurement, are used in equation 3.12, along with the measured discharge, Q_m, and observed change in stage, dh/dt, to compute corresponding values of the adjustment factor, $1/S_c v_w$. The computed values of $1/S_c v_w$ are then plotted against stage and a smooth curve is fitted to the plotted points. If the plotted values of $1/S_c v_w$ scatter widely about the curve, the Q_c curve is modified to produce some new values of $1/S_c v_w$ that can be better fitted by a smooth curve. The modifications of the curves of Q_c and $1/S_c v_w$ should not be so drastic that the modified curves are no longer smooth curves, nor should the modified shape of the Q_c rating curve violate the principles underlying rating curves, as discussed in Chapter 1. Construction of the two curves completes the rating analysis. Figure II.3.10 is an example of such an analysis.

To adjust the value of subsequent discharge measurements for plotting on the Q_c rating curve, the adjustment-factor curve is first entered with the stage of the measurement to obtain the appropriate value of the factor, $1/S_c v_w$. Next, the observed value of dh/dt is used with that factor to compute the term:

$$\left(1 + \frac{1}{S_c v_w}\frac{dh}{dt}\right)^{0.5}$$

That term is then divided into the measured discharge, Q_m, to obtain the required value of Q_c.

To determine true discharge Q_m, based on the Q_c rating curve and adjustment-factor curve, during a period when the stage and rate of change of stage are known, the procedure described above is used to obtain the value of the term noted above. That term is then multiplied by Q_c, which is obtained by entering the Q_c rating curve with the known stage. The product is the true discharge, Q_m.

Wiggins method

The Wiggins method is a modification of the Jones method in which the energy gradient, rather than the water-surface slope, is used. The method is convenient for adjusting measured discharge Q_m for the effect of changing discharge to obtain the corresponding steady-flow discharge Q_c. However, the reverse procedure of computing discharge Q_m for unsteady flow from the steady-flow discharge rating is rather complicated. Consequently the Wiggins method is used only for those stations where only occasional adjustment of measured discharge at high stages is required. If the discharge is affected by changing stage on numerous days each year the Boyer method of discharge adjustment should be used.

The discharge measurements adjusted by the Wiggins method are used to develop a steady-flow rating, and that rating is used directly with the gauge-height record to obtain daily values of discharge. This course of action is justifiable for those streams whose discharge is affected by changing discharge on only a few days each year. For that type of stream it will generally be found that the discharge adjustment is less than ten per cent. On the affected days, the discharge obtained from the steady flow rating will be underestimated by a small percentage when the stage is rising rapidly, and overestimated by a small percentage when the stage is falling rapidly. The discrepancies are compensating and if only a few days are involved, the streamflow record is not significantly impaired. The advantage of applying the adjustment to discharge measurements made under unsteady-flow conditions is that the scatter of discharge measurements on the rating curve is reduced and the rating curve can therefore be more precisely defined.

Application of the Wiggins method has been simplified by the preparation of diagrams that eliminate much of the computational labour. Figures II.3.11 to II.3.14 are used to determine the unsteady-flow value of the energy slope, S_m, at the time of the discharge measurement, Q_m, for combinations of values of mean velocity, V_m and

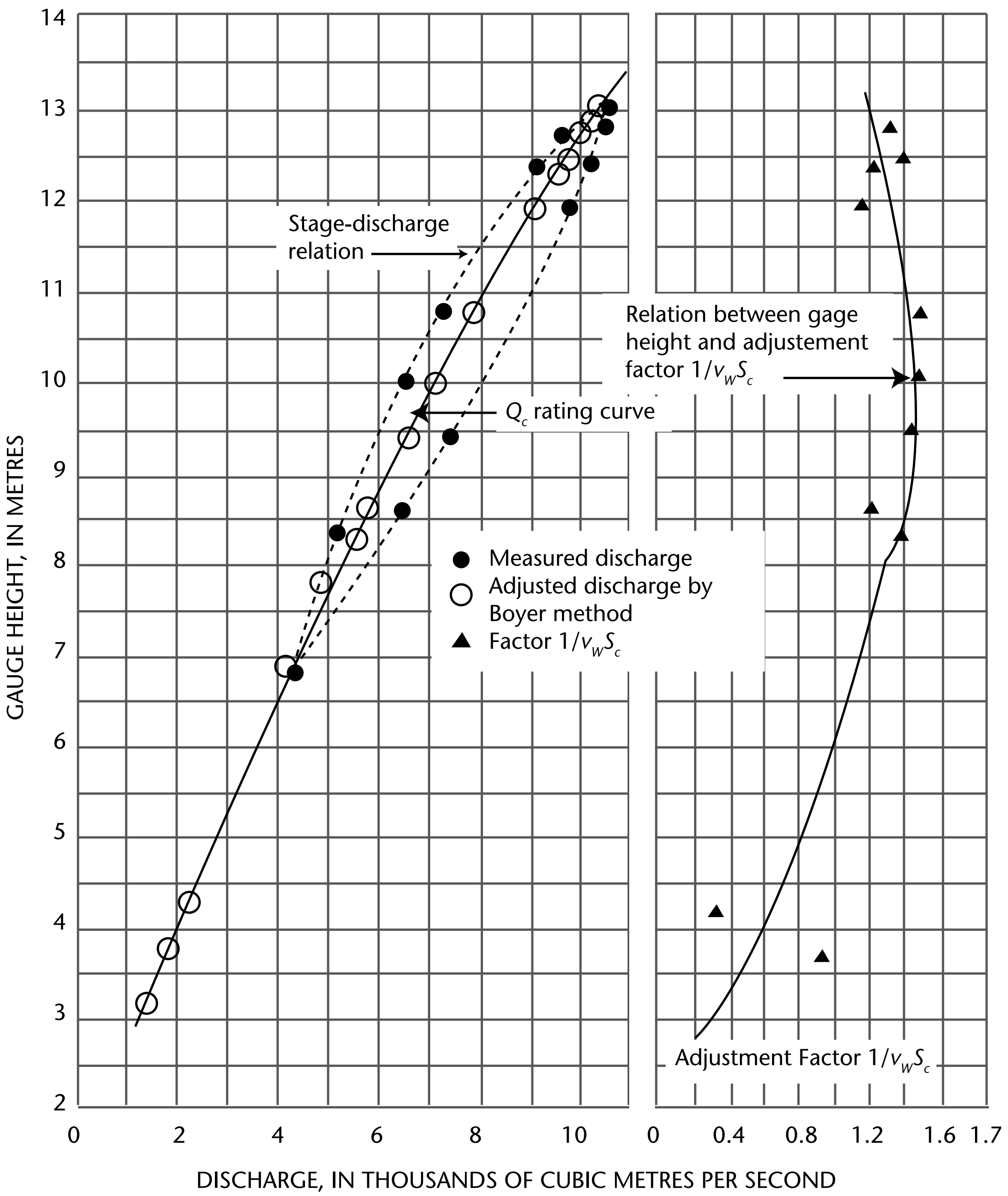

Figure II.3.10. Adjustment of discharge measurements for changing discharge, Ohio River at Wheeling, West Virginia, United States during the period 14-27 March 1905, from Corbett et al. (1945)

hydraulic radius, *R*. The Manning equation was used in preparing the graphs, and each of the four graphs is applicable for a particular value of Manning's *n*, as shown in the following tabulation:

- Figure II.3.11: $n = 0.025$ (smooth bed and banks)
- Figure II.3.12: $n = 0.035$ (fairly smooth)
- Figure II.3.13: $n = 0.050$ (rough)
- Figure II.3.14: $n = 0.080$ (very rough)

Figure II.3.15 is used to determine the increment of energy slope:

$$\frac{1}{v_w}\frac{dh}{dt}$$

attributable to changing discharge, for combinations of values of flood-wave velocity, v_w, and rate of change of stage *dh/dt*. Flood-wave velocity is assumed to equal $1.3V_m$.

Figures II.3.16 and II.3.17 are used to determine the factor to apply to the measured discharge, Q_m, to obtain the steady-flow discharge, Q_c. The factor, which is equal to:

$$\left[\frac{\left(S_m - \frac{1}{v_w}\frac{dh}{dt}\right)}{S_m}\right]^{0.5}$$

is given for combinations of values of S_m from Figures II.3.11 to II.3.14 and of:

$$\frac{1}{v_w}\frac{dh}{dt}$$

from Figure II.3.15. Note that the factor differs from that given in equation 3.16, because S_m is used here as the base slope, rather than S_c as in equation 3.16. Figure II.3.16 is used for rising stages and Figure II.3.17 is used for falling stages.

An example of the use of the Wiggins diagrams follows.

Given: A discharge measurement with the following data for a stream with a fairly smooth bed ($n = 0.035$):

Q_m = 6513 m^3 s^{-1}
Area = 5007 m^2
Width = 823 m
V_m = 1.30 m s^{-1}
Change in stage = 0.265 m in 1.5 hours (rising).

Compute adjusted discharge to be plotted on the rating curve.

First compute:

$$R = \frac{Area}{Width} = \frac{5007}{823} = 6 \text{ m}$$

$$v_w = 1.3V_m = 1.3 \times 1.30 = 1.69 \text{ m s}^{-1}$$

$$\frac{dh}{dt} \text{ change in stage per hour } = \frac{0.265}{1.5} = 0.177 \text{ m / hr}$$

Then:

(a) Enter Figure II.3.12 with V_m = 1.30 and R = 6.0 and read S_m = 0.00018;
(b) Enter Figure II.3.15 with *dh/dt* = 0.177 and V_w = 1.69 and read slope increment equal to 0.000029;
(c) Enter Figure II.3.16 (rising stage) with S_m = 0.00018 and slope increment 0.000029 and read factor = 0.915;
(d) Compute adjusted discharge = 0.915 × 6513 = 5959 m^3 s^{-1}.

Because the stage was rising the unadjusted discharges would plot to the right of the rating curve. The computed adjustment moves the measurement to the left.

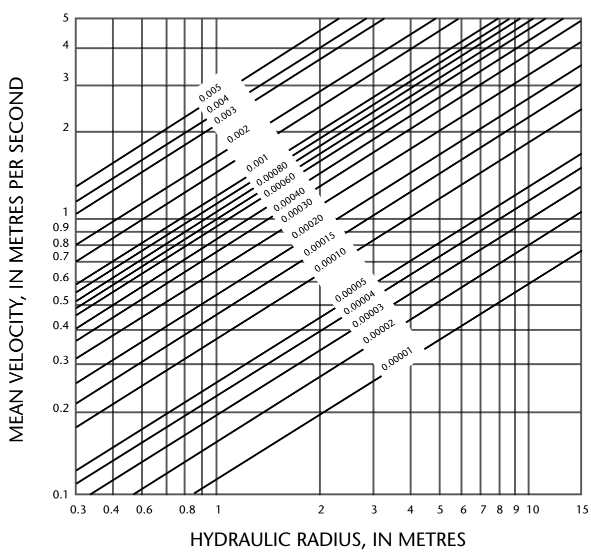

Figure II.3.11. Diagram for solution of the Manning formula to determine S_m. Smooth bed and banks, n = 0.025.

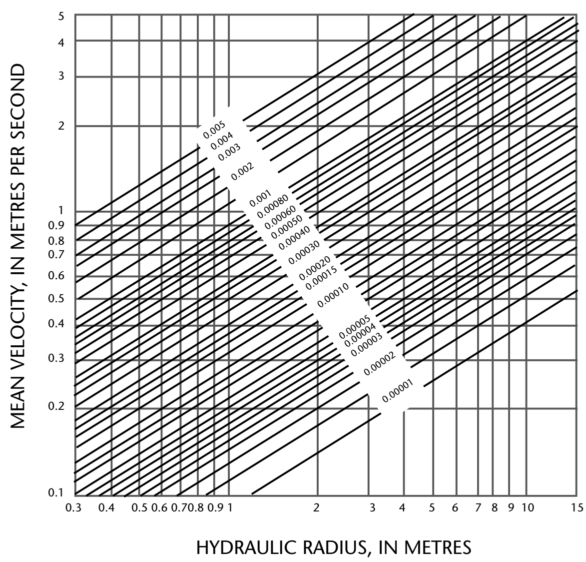

Figure II.3.12. Diagram for solution of the Manning formula to determine S_m. Fairly smooth bed, n = 0.035.

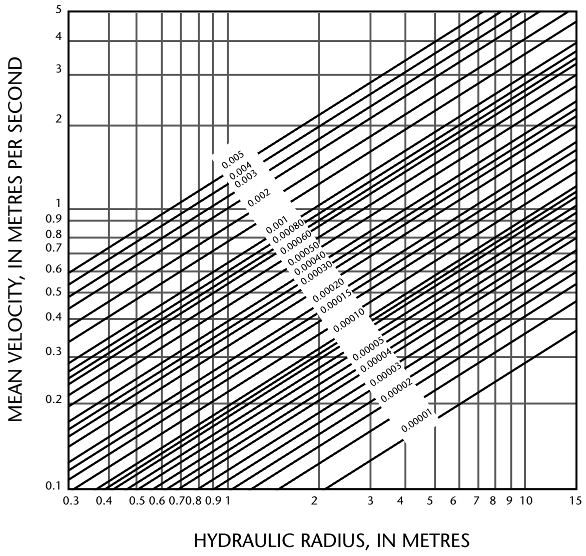

Figure II.3.13. Diagram for solution of the Manning formula to determine S_m. Rough bed, n = 0.050

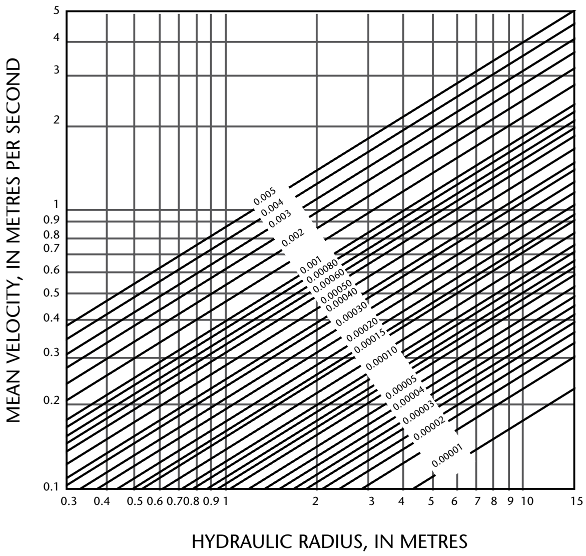

Figure II.3.14. Diagram for solution of the Manning formula to determine S_m. Very rough bed, n = 0.080

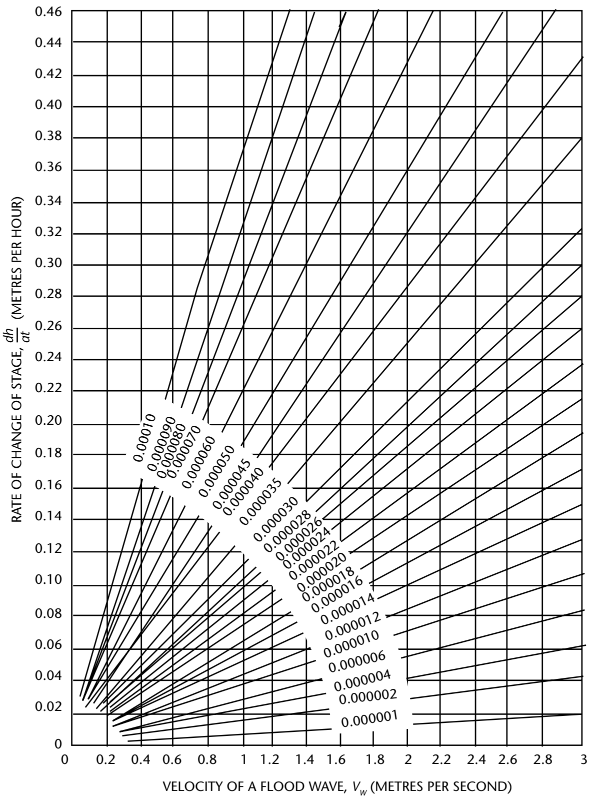

Figure II.3.15. Diagram for determining slope increment due to changing discharge

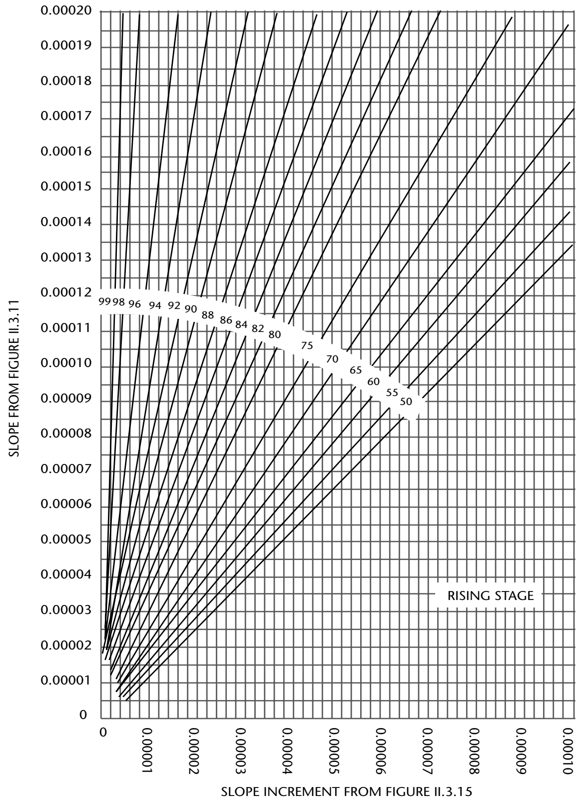

Figure II.3.16. Diagram for determining factor to apply to measured discharge during periods of rising stage

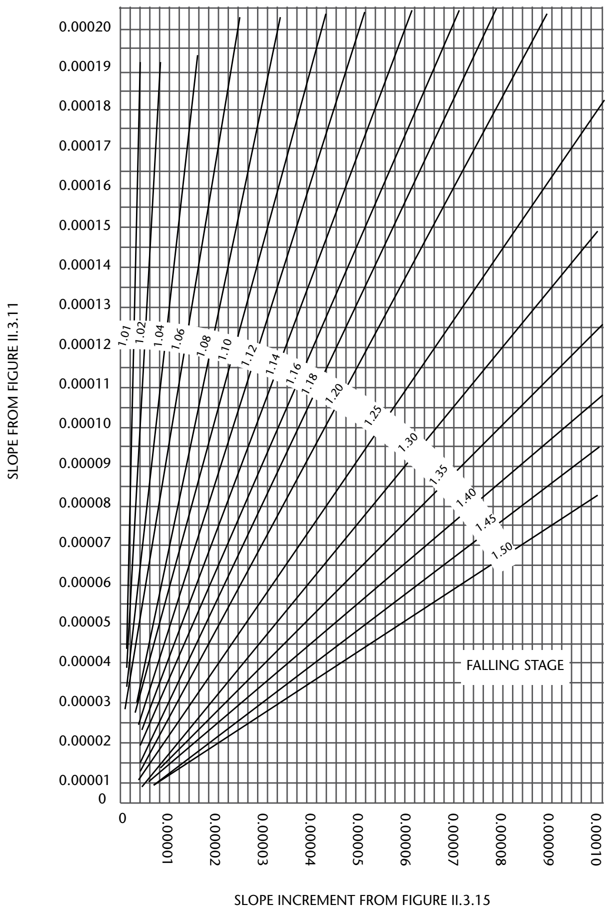

Figure II.3.17. Diagram for determining factor to apply to measured discharge during periods of falling stage

Both the measured discharge, Q_m, and adjusted discharge, Q_c, are entered in the list of discharge measurements and both are plotted on the rating curve. Suitable symbols are used, however, to differentiate between the measured and adjusted discharges.

3.5 VARIABLE SLOPE CAUSED BY A COMBINATION OF VARIABLE BACKWATER AND CHANGING DISCHARGE

Where the rating for a gauging station is affected by a combination of variable backwater and changing discharge, the rating should be analyzed as though it were affected by variable backwater only, using the fall-rating methods described in previous sections of this chapter. The basic equation for variable-backwater adjustments (equation 3.7) and that for changing-discharge adjustments (equation 3.13) are similar, but only the fall-rating methods are versatile enough to handle the combined effect of the two factors.

3.6 SHIFTS IN DISCHARGE RATINGS WHERE SLOPE IS A FACTOR

Changes in channel geometry, such as scour or fill; and/or changes in flow conditions, such as vegetal growth, will cause shifts in the discharge rating where slope is a factor, just as they cause shifts in simple stage-discharge relations. When discharge measurements indicate a shift in the rating for a slope station, the shifts should be applied to the Q_r rating curve if the station is affected by variable backwater, or to the Q_c rating curve if the station is affected by changing discharge. Extrapolation of the shift curves should be performed in accordance with the principles discussed in Chapter 1 for shifts in simple stage-discharge relations.

3.7 COMPUTER METHODS FOR ANALYSIS AND COMPUTATION OF SLOPE AFFECTED RATINGS

The preceding sections of this chapter have described the theory and methods of computing discharge for stream gauging stations where variable backwater does not allow the use of a simple stage-discharge relation. The methods of computation, as described, are hand methods using calculators and hand-plotted graphs. This is necessary so that the underlying concepts of slope affected ratings are properly understood. Today, however, with the advent of computers it is possible to develop, analyze and apply slope-affected ratings quickly and easily. The theory has not changed, only the method of applying that theory.

In the past, the development of slope-affected ratings required laborious hand plotting and re-plotting of ratings to arrive at an acceptable calibration. Today the ratings can be plotted and replotted quickly and easily with computer programs designed for that purpose, thereby giving the hydrologist more time for the analytical aspects of the ratings. An additional benefit is that computer plots are much less susceptible to errors.

Some ratings for variable backwater conditions may be better analyzed using theoretical equations rather than plotted ratings and hand fitted curves. In addition, ratings may, in some cases, be fitted to a set of data points using a least squares analysis or other curve-fitting technique. Computer analysis for such methods provides additional valuable information about deviations of individual measurements and overall error analysis of the rating itself.

Computation of discharge records using variable slope as a factor with be described in a subsequent chapter of this manual. Emphasis will be placed on computer methods for making these computations.

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CHAPTER 4

FLOW COMPUTATION MODELS FOR UPLAND, BRANCHED AND TIDAL STREAMS

4.1 GENERAL

A discharge rating can sometimes be obtained for upland streams affected by variable backwater and for tide-affected streams if a velocity index is used as a parameter in the rating, along with stage. The ultrasonic and Acoustic Doppler Velocity Meters (ADVM), as described in Volume I, Chapter 8, may in many instances be satisfactory for obtaining a continuous record of velocity. Another approach for computation of discharge where variable backwater exists is through the use of slope ratings, as described in Volume II, Chapter 3. In the past, some empirical methods have also been used; however these are seldom used today and will only be described briefly.

In situations where ratings and slope ratings are not easily calibrated, flow computation models involving the evaluation of the equations of unsteady flow may be used. Over the past 50 years, investigators in several countries have been involved in the development and refinement of these models. The descriptions and examples that follow will be based almost entirely on publications of the United States Geological Survey (USGS). Publications by Amein and Fang (1970), Baltzer and Lai (1968), Cunge, Holly, and Verwey (1980), Fread (1974), Lai, Baltzer, and Schaffranek (1980), Preissmann (1960), Schaffranek (1982), Schaffranek, Baltzer and Goldberg (1981) and Strelkoff (1969) are examples of some of the literature available on the subject of unsteady flow equations and their solution. In particular, the report by Schaffranek (1987) will be used extensively in the following discussions. Some parts of that report, including examples of application, are used verbatim.

In the past, the utility of numerical-simulation modeling was often limited by imposition of certain simplifying assumptions that were both necessary and justifiable at the time. They were necessary because numerical methods and/or computer capacity were deficient and justifiable because parametric evaluation techniques and/or equipment were lacking or inadequate. Today, for the most part, advances in numerical methods, computer technology and hydrologic instrumentation have enabled model engineers to reduce the number of such restrictions, thus producing models that are more nearly formulated on pure hydraulic considerations and have a greater potential to provide more comprehensive flow information. Consequently, the scope and complexity of hydrodynamic problems that are now tractable have expanded.

This expansion of the role of numerical-simulation modeling has stimulated the need for rapid economical and efficient techniques to compile and appraise prototype data and model results. Thus, it is insufficient for a numerical scheme to be developed merely to the state of being a model program. To achieve a state of usefulness as an operationally oriented investigative tool, the model program must be supported by a comprehensive user-oriented data system and must provide a ready means of presenting output results in varied graphical forms.

In view of this need, the USGS has developed a comprehensive, one-dimensional numerical-simulation model that is fully supported by a user-oriented system for modeling. The branch-network flow model, as it is called, is capable of simulating unsteady flow in a single open-channel reach or throughout a network of reaches composed of simple or multiple connected one-dimensional flow channels governed by various time-dependent forcing functions and boundary conditions. Operational modeling capability is achieved by linking the model to a highly efficient storage-and-retrieval module that accesses a data base containing time series of boundary values and by including an extensive set of digital graphics routines. These features help transform the model into a comprehensive tool for practical use in the conduct of hydrologic investigations.

Two illustrative applications of the model are presented. Application to a 274-m reach of Pheasant Branch near Middleton, Wisconsin, United States of America, demonstrates its capability in computing unsteady flow in short, upland-river reaches that can be highly responsive to climatological conditions. Application to a 25-branch schematization of the 50-km tidal river part of the Potomac Estuary near Washington, D.C., United States, illustrates its feasibility in simulating tidal flows in estuarine-type network environments that are frequently subject to extreme freshwater inflows and variable meteorological influences.

4.2 TERMINOLOGY

To facilitate further discussion of the application of the model to either a single riverine channel or a system of channels, a few definitions are necessary. The terms reach and branch are used somewhat interchangeably to mean a length of open channel. The primary subdivision of a reach or branch is referred to as a subreach or segment. A network is defined as a system of open channels either simply connected in treelike fashion or multiple connected in a configuration that permits more than one flow path to exist between certain locations in the system.

4.3 ONE-DIMENSIONAL UNSTEADY FLOW EQUATIONS

One-dimensional unsteady flow in open channels can be described by two partial-differential equations expressing mass and momentum conservation. These well-known equations, frequently referred to as the unsteady flow, shallow water or St. Venant equations (Baltzer and Lai, 1968; Dronkers, 1969; Strelkoff, 1969; Yen, 1973) can be written:

$$B\frac{\partial Z}{\partial t} + \frac{\partial Q}{\partial x} - q = 0 \tag{4.1}$$

and

$$\frac{\partial Q}{\partial t} + \frac{\partial (\beta Q^2/A)}{\partial x} + gA\frac{\partial Z}{\partial x} + \frac{gk}{AR^{4/3}} \tag{4.2}$$

$$Q|Q| - qu' - \varepsilon B_c U_a^2 \cos\alpha = 0$$

in which the momentum coefficient, β, the flow-resistance function, k, and the wind-resistance coefficient, ε, are defined as:

$$\beta = \frac{1}{U^2 A}\int u^2 dA \tag{4.3}$$

$$k = \left(\frac{\eta}{1.49}\right)^2 \quad \text{(or, in SI units, as } k = \eta^2\text{)} \tag{4.4}$$

and

$$\varepsilon = C_d \frac{\varsigma_a}{\varsigma} \tag{4.5}$$

In these equations, formulated using water-surface elevation, Z, and flow discharge, Q, as the dependent variables, distance along the channel thalweg, x, and elapsed time, t, are the independent variables. (Longitudinal distance, x, and flow discharge, Q, are positive in the downstream direction.) Other quantities in the preceding equations are defined as follows: A = area of conveyance part of cross section; B = total top width of cross section; B_e = top width of conveyance part of cross section; C_d, = water-surface drag coefficient; g = gravitational acceleration; q = lateral inflow per unit length of channel (negative for outflow); R = hydraulic radius of cross section; u = flow velocity at a point; u'= x-component of lateral flow velocity; U = mean velocity of flow, = Q/A; U_a = wind velocity; α = wind direction measured from positive x-axis; η = flow-resistance coefficient similar to Manning's n; ς = water density and ς_a = atmospheric density.

Although hydraulic radius, R, is used in equation 4.2 and in subsequent expansions throughout this development, the commonly used substitution of hydraulic depth is employed in the model. This approximation, $R = A/B$, is assumed valid for shallow water bodies, that is, channels having a large width-to-depth ratio.

The momentum coefficient, β, also called the Boussinesq coefficient, is present in the equation of motion to account for any non-uniform velocity distribution (see equation 4.3).

Equations 4.1 and 4.2 are, in general, descriptive of unsteady flow in a channel of arbitrary geometric configuration having both conveyance and overflow (or only conveyance) areas and potentially subject to continuous lateral flow and/or the shear-stress effects of wind. In their formulation it is assumed that the water is homogeneous in density, hydrostatic pressure prevails everywhere in the channel, the channel bottom slope is mild and uniform, the channel bed is fixed (that is no scouring or deposition occurs), the reach geometry is sufficiently uniform to permit characterization in one dimension and frictional resistance is the same as for steady flow, thus permitting approximation by the Chezy or Manning equation.

4.4 MODEL FORMULATION

Numerous varied mathematical methods and corresponding numerical schemes exist that render approximate solutions of the flow equations. However, new methods and alternative schemes that provide more accurate approximations and are inherently more flexible and efficient are continually being sought. In the branch-network model formulation, the flow equations are expressed in finite-difference form using a weighted four-point (box) scheme. This technique, also used by Fread (1974) and by Cunge and others (1980), permits the model to be applied using unequal segment lengths and box-centered to fully forward discretizations. A unique transformation operation is applied to the segment flow equations in the branch-network model, however, to lower the order of the coefficient matrices and thereby reduce computer time and storage requirements. A general matrix solution algorithm is used to simultaneously solve the resultant branch-transformation and boundary-condition equations. The implicit solution method is employed because of its inherent efficiency and superior stability properties. An optional iteration procedure, controllable by user-defined tolerance specifications, is additionally provided to permit improving the accuracy of the computed unknowns.

It is beyond the scope of this report to describe the mathematical procedures and solution techniques in detail. Those readers that are interested in the detailed mathematics involved in the finite difference technique should refer to Fread (1974), Cunge and others (1980), Schaffranek (1987), Schaffranek, Baltzer, and Goldberg, (1981), Preissmann (1960), Amein and Fang (1970) and Baltzer and Lai (1968).

The solution process begins at an initial time by use of specified initial conditions and proceeds in specified time increments to the end of the simulation. Gauss elimination using maximum pivot strategy is employed to solve the system of equations. Iteration within a time step is performed to provide results within user-specified tolerances. The primary effect of iteration is to improve on the quantities taken as local constants within the time step, which in turn increases the accuracy of the computed unknowns. User-defined accuracy requirements are typically achieved in two or fewer iterations per time step.

4.5 BOUNDARY CONDITIONS

To solve the branch-transformation equations implicitly, boundary conditions must be specified at internal junctions located at branch confluences within the network as well as at external junctions located at the extremities of branches, for example, where branches physically terminate or are delimited for modeling purposes. Equations describing the boundary conditions at internal junctions are automatically generated by the model, whereas boundary-condition equations for external junctions are formulated by the model from user-supplied time-series data or from user-specified functions.

Various combinations of boundary conditions can be specified for external junctions. A null discharge condition (as, for example, at a dead-end channel), known stage or discharge as a function of time, or a known, unique stage-discharge relationship can be prescribed. Together, the internal and external boundary conditions provide a sufficient number of additional equations to satisfy requirements of the solution technique.

4.6 MODEL APPLICATIONS

The thoroughness of the equation formulation on which a model is based largely governs the range of complexity of flows it can accommodate. The choice of a numerical computation scheme primarily determines whether or not the model will be stable, convergent, accurate and computationally efficient given that it is correctly and precisely implemented. However, for any model to be useful it must be subsequently transformed into a functional user-oriented simulation system, and its accuracy, reliability and versatility must be adequately proved and demonstrated.

The branch-network model is being used to simulate the time-varying flows of several coastal and upland water bodies in the United States, as identified in Table II.4.1. These represent a broad spectrum of hydrologic field conditions, depicting such diverse hydraulic and field situations as hydropower-plant-regulated flows in a single upland-river reach, tide-induced flows in riverine and estuarine reaches and networks, unsteady flow in a residential canal system and meteorologically generated seiches and wind tides in a multiply connected network of channels joining two large lakes.

Four types of model application are identified in Table II.4.1. The simplest of these is the single-branch type, which is an application to a single reach of channel delimited by a pair of external boundary conditions. The multiple-branch type is an application to a channel, again delimited by a pair of external boundary conditions, but schematized as a series of sequentially connected reaches. The dendritic-network type is an application to a channel system composed of branches connected in treelike fashion. The multiple connected network type is likewise an application to a channel system, but one in which the branches

Table II.4.1. Application of the branch-network flow model in the United States

State	*Water body location*	*Application type*
Alabama	Coosa River near Childersburg	35.2-km multiple branch
	Alabama River near Montgomery	21-branch multi-connected network
Alaska	Knik/Matanuska River Delta near Palmer	20-branch multi-connected network
California	Sacramento River from Sacramento to Freeport	17.4-km single branch
	Sacramento River from Sacramento to Hood	34.3-km multiple branch
	Sacramento Delta between Sacramento and Rio Vista	24-branch multi-connected network
	Threemile Slough near Rio Vista	5.2-km single branch
Connecticut	Connecticut River near Middletown	9.8-km single branch
	Connecticut River downstream from Hartford	41.2-km multiple branch
Florida	Cape Coral residential canal system	16-branch multi-connected network
	Peace River from Arcadia to Fort Ogden	30-km multiple branch
	Peace River from Fort Ogden to Harbour Heights	21-branch multi-connected network
Idaho	Kootenai River near Porthill	54.8-km multiple branch
Kentucky	Ohio River Downsteam from Greenup Dam	21.7-km single branch
Louisiana	Atchafalaya River near Morgan City	8-branch multi-connected network
	Wax Lake Outlet near Calumet	15-branch multi-connected network
	Calcasieu River between Lake Charles and Moss Lake	13-branch multi-connected network
	Quachita River form Monroe to Columbia	78.9-km multiple branch
	Vermillion River from Lafayette to Perry	48.3-km multiple branch
	Loggy Bayou near Ninock	9.2-km single branch
	Mermentan River from Mermetan to Lake Arthur	25.7-km multiple branch
Maryland	Potomac River near Wahington, D.C.	25-branch multi-connected network
Michigan	Detroit River near Detroit	12-branch multi-connected network
	Saginaw River near Saginaw	1.4-branch dentritic network
Missouri	Osage River near Schell City	2.6-km single branch
New York	Hudson River from Albany to Poughkeepsie	9-branch dentritic network
North Dakota	Red River of the North at Grand Forks	1.3-km single branch
South Carolina	Intracoastal Waterway near Myrtle Beach	36.7-km multiple branch
	Cooper River at Diversion Canal	6.3-km single branch
	Cooper River at Lake Moultrie Tailrace	1.5-km single branch
	Back River near Cooper River confluence	2.2-km single branch
South Dakota	James River near Hecla	8.5-km multiple branch
Washington	Columbia River dowmstream from Rocky Reach Dam	3.1-km single branch
Wisconsin	Pheasant Branch near Middleton	0.27-km single branch
	Menomonee River near Milwaukee	0.61-km single branch
	Milwaukee Harbor at Milwaukee	12-branch multi-connected network

are interconnected, thereby permitting multiple flow paths between certain locations in the system.

To illustrate the diverse capabilities of the model, two applications identified in Table II.4.1 are discussed briefly herein. These particular applications were selected to demonstrate the flexibility of the model in accommodating a wide range of hydrologic conditions and field situations.

4.6.1 Columbia River reach at Rocky Reach Dam near Wenatchee, Washington, United States

The branch-network flow model has been used to compute the flow of the Columbia River immediately downstream from Rocky Reach Dam near Wenatchee, Washington, United States. This relatively short reach (3.1 km) is treated as a single-segment branch in the model schematization. Flow in the reach is highly unsteady owing to regulation created by the combined operation of turbines and gates at the dam for the purpose of optimal hydroelectric power generation.

Channel geometry data for the model were abstracted from detailed field surveys, processed by the cross-sectional geometry program and prepared for input to the model. The branch network flow model treats the reach as a single segment; therefore, stage-area-width tables were produced that define the upstream and downstream cross sections at the boundary-value data locations.

Time series of water-surface elevations are used as boundary conditions for the model application. These data are collected on a continuous basis at the field station locations (stations numbered 12-4537.00 and 12-4537.01) identified in Figure II.4.1 near river miles 471 and 473, respectively. The close proximity of the boundary-value stations underscores the importance of precise synchronized recording of the water-surface elevations. The boundary value data are extracted from the time dependent data base during the simulation as required to define the boundary conditions.

The highly unsteady nature of the flow is illustrated in the model-generated plot of computed discharges in Figure II.4.2. As this figure illustrates, the unsteady discharge can vary as much as 2 000 $m^3 s^{-1}$ in less than 2 hours elapsed time. In fact, the discharge has been observed to vary as much as 1 000 $m^3 s^{-1}$ in less than 0.5 hour. This application amply demonstrates the ability of the branch-network flow model to simulate highly varying flow conditions, as may be encountered in regulated upland rivers.

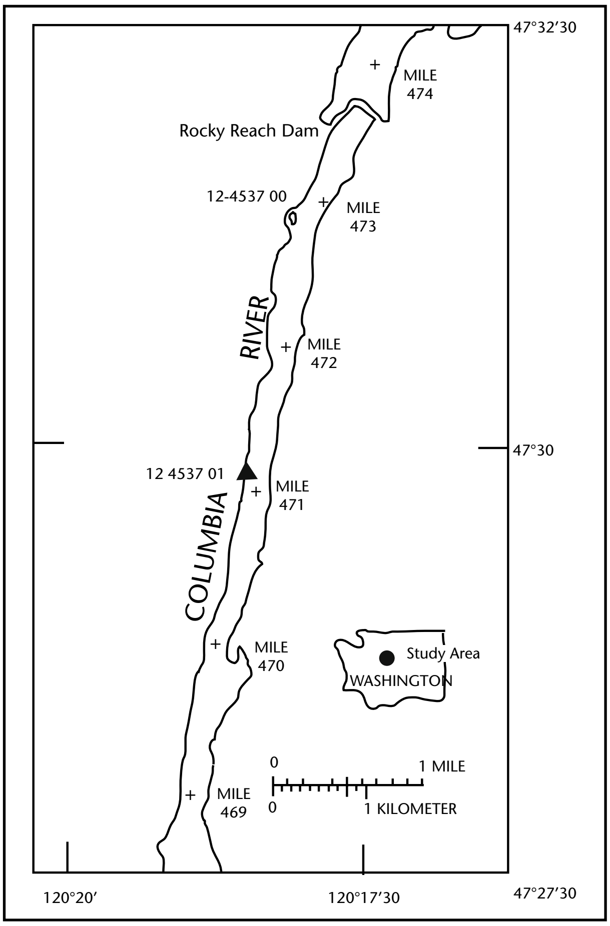

Figure II.4.1. Columbia River reach near Rocky Reach Dam in the State of Washington, United States

4.6.2 Potomac River near Washington, D.C., United States

In October 1977, the Water Resources Division of the USGS instituted a 5-year interdisciplinary study of the tidal Potomac River and Estuary (Callender and others, 1984). The research areas undertaken in this investigation included historical geologic studies, geochemistry of bottom sediments, nutrient cycling, sediment transport and tributary loading, wetland studies, benthic ecology and hydrodynamics. The objective of the hydrodynamics project was to devise, implement, calibrate and verify a series of numerical flow/transport simulation models in support of the other research efforts. To quantify the hydrodynamics of the tidal river, the branch-network model was applied to the 50-km segment of the Potomac, including its major tributaries and inlets from the head of tide at the fall line in the northwest quadrant of Washington, D.C., to Indian Head, Maryland, United States, as shown in Figure II.4.3.

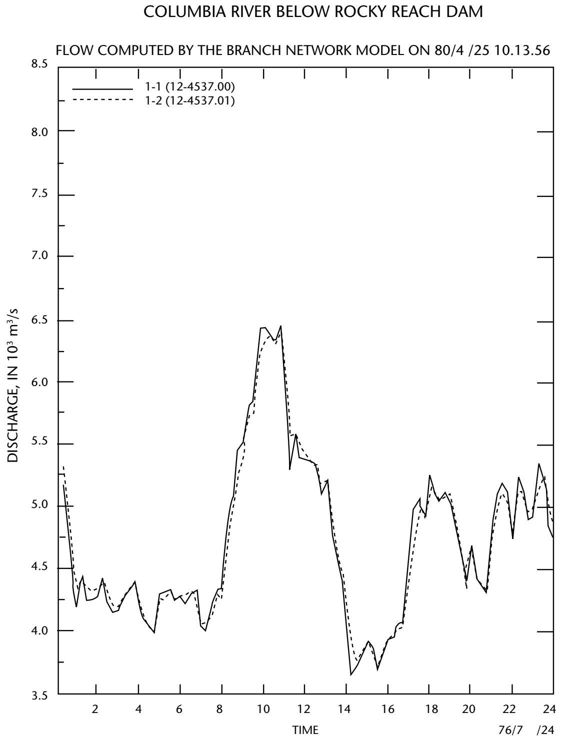

Figure II.4.2. Comparison of computed discharge to observed discharge for the Columbia River near Rocky Reach Dam in the State of Washington, United States

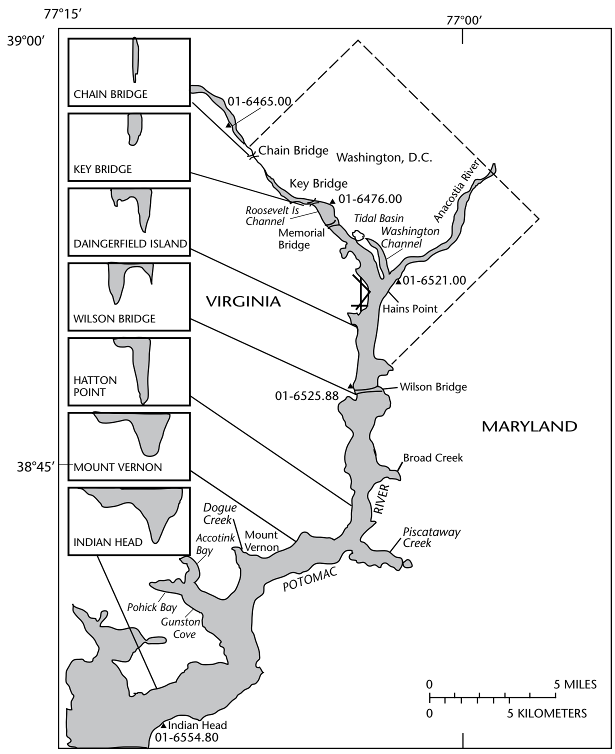

Figure II.4.3. Potomac River near Washington, DC., United States

The Potomac River downstream from Chain Bridge is confined for a short distance (approximately 5 km) to a narrow, deep, but gradually expanding channel bounded by steep rocky banks and high bluffs. Farther downstream the river consists of a broad, shallow, and rapidly expanding channel confined between banks of low to moderate relief. Seven cross-sectional profiles illustrating the channel geometry are plotted in Figure II.4.3. The cross-sectional area and corresponding channel width expand more than forty-fold between Chain Bridge and Indian Head. In general, the depth varies from about 9 m at Chain Bridge to about 12 m at Indian Head.

Flow in the upstream portion of the tidal river is typically unidirectional and pulsating. Bidirectional flow occurs in the broader downstream portion. The location of the transition from one flow pattern to the other varies, primarily in response to changing inflow at the head of tide but also to changing tidal and meteorological conditions.

The tidal river system is schematized as shown in Figure II.4.4. The network is composed of 25 branches (identified by roman numerals) that join or terminate at 25 junction locations (identified by numbered boxes). Junctions that do not constitute tributary or inlet locations in Figure II.4.4 were included in the network schematization to accommodate potential nodal flows (point source inflows or outflows such as sewage treatment outfalls or pump withdrawals) or to account for abrupt changes in channel characteristics.

A total of 66 cross sections were used to depict the channel geometry in 52 flow segments. Whereas the coefficient matrix of segment flow equations would require 15 376 computer words, use of branch-transformation equations reduces the matrix size to 10 000 words. The computational effort required to affect a solution is also proportionally reduced.

In the tidal Potomac River model, flow discharges derived at a rated gauging station (01-6465.00) 1.9 km upstream from Chain Bridge are used as boundary values at junction 1. Water-surface elevations recorded at a gauging station (01-6554.80) at Indian Head are used as the downstream boundary values at junction 19. All other external boundary conditions are fulfilled by specifying that

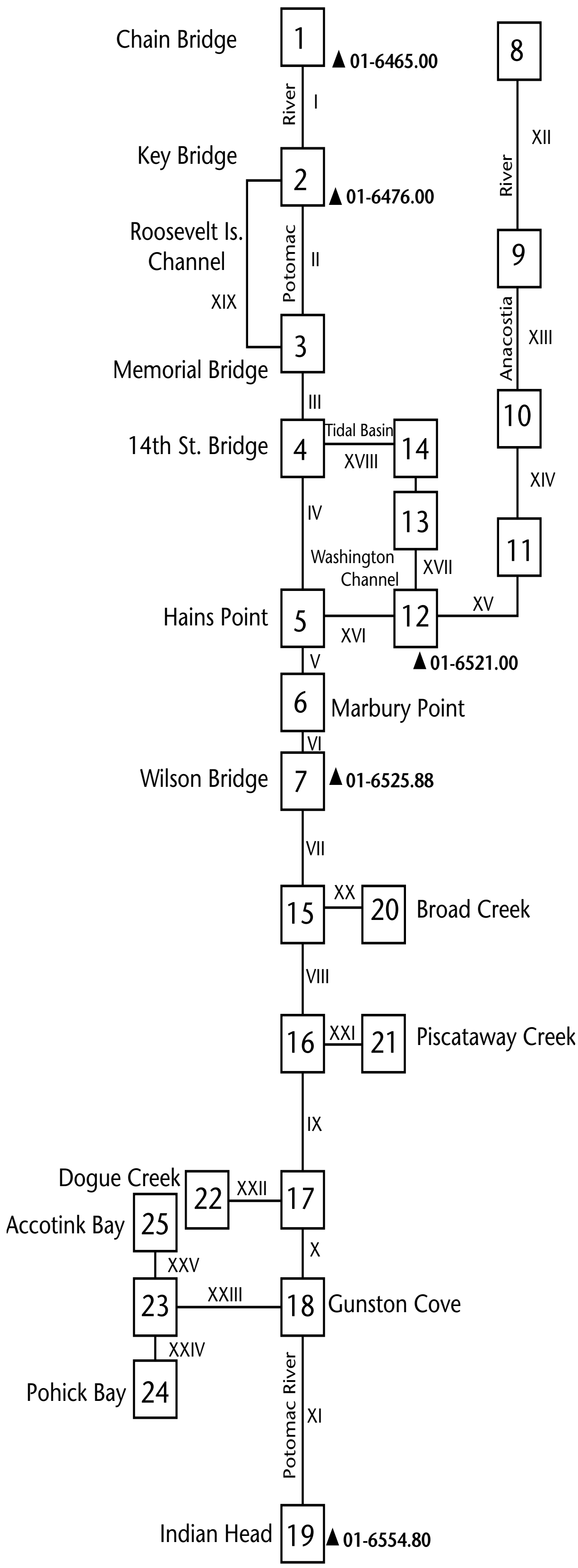

Figure II.4.4. Schematization of the tidal Potomac River system for the branch-network flow model

zero discharge conditions prevail at the upstream tidal extent of the particular channel or embayment.

Water-surface elevations recorded near Key Bridge (station 01-6476.00), near Wilson Bridge (station 01-6525.88) and near Hains Point (station 01-6521.00) were used to calibrate and verify the model (see Figures II.4.3 and II.4.4). Model-computed discharges were also compared with discharges measured for complete tidal cycles at Daingerfield Island, Broad Creek and Indian Head.

In Figure II.4.5, model-computed discharges are plotted against discharges measured at Indian Head from 2015 hours on 3 June to 0830 hours on 4 June, 1981. As is evident from the plot, there is excellent agreement between computed and measured discharges. Computed and measured ebb and flood volume fluxes compare within + 0.6 and – 2.3 per cent, respectively. This application of the model clearly demonstrates its adaptability to the simulation of unsteady flow in a network of interconnected channels.

4.7 EMPIRICAL METHODS

Four empirical methods of rating tidal reaches have been in use, all but one of which were developed before the use of digital computers became commonplace. These methods are no longer in general use for tidal streams today because models such as the branch-network model described in the previous sections of this chapter are easily applied with much better accuracy.

The empirical techniques are:
(a) Method of cubatures;
(b) Rating-fall method;
(c) Tide-correction method;
(d) Coaxial graphical-correlation method.

All of the above methods have their shortcomings which are discussed, where appropriate, in the following sections.

4.7.1 Method of cubatures

One of the oldest methods of computing discharge in tidal estuaries is the method of cubatures (Pillsbury, 1956). The method is based on the equation of the conservation of mass, where outflow at the study station = inflow ± change in storage.

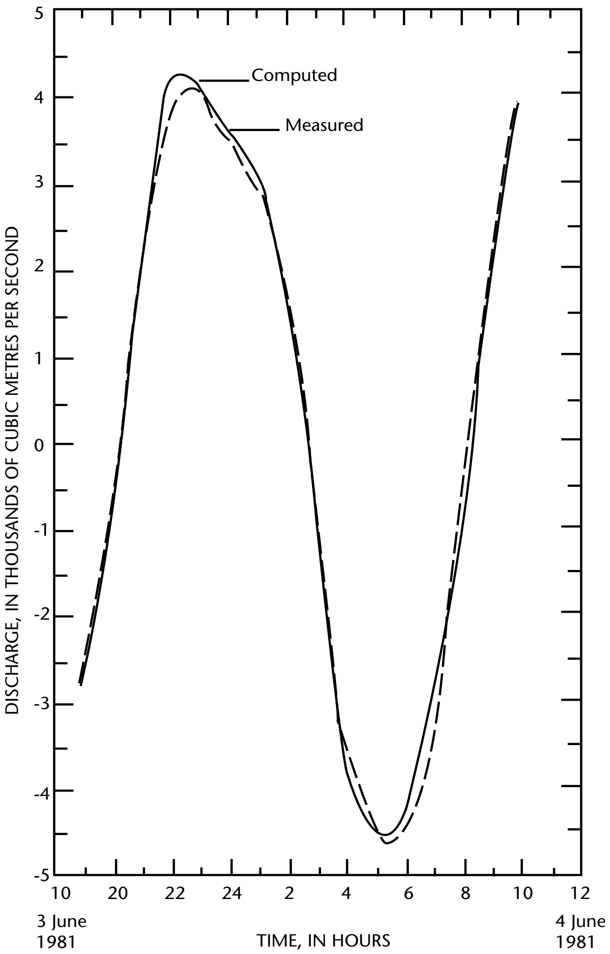

Figure II.4.5. Model-generated plot of computed versus measured discharges for the Potomac River at Indian Head, Maryland, United States, on 3-4 June 1981

The inflow term in the above equation is the freshwater discharge measured at a gauging station at or upstream from the head of tide – that is, a gauging station having a simple stage-discharge relation. The storage term refers to volume of water in the reach between the inflow gauging station and the study station on the estuary. Intermediate stage gauges are usually needed for evaluating the storage term. The gauges are spaced at such distances that no significant error is introduced in the computations by considering the water surfaces between gauges as planes. That requirement ordinarily is met by stations some kilometres apart, but suitably placed with regard to marked changes in the cross-section of the waterway. The differences in the tidal ranges on the opposite shores of a wide estuary may usually be disregarded, but it may be necessary to establish tidal stations on any long tidal tributaries of the main waterway. For convenience in the computations, the tides at all stations should be reduced to the same horizontal datum, preferably taken low enough to make all stages positive.

If existing surveys do not afford reliable data on the areas of the water surfaces between the selected tidal stations, a survey to establish these surfaces is required. Usually such surface areas may be taken as increasing, uniformly from low water to high water, but if there are any considerable tide flats that are exposed at the lower tidal stages, the area at the stage at which such flats are covered should also be found.

Freshwater inflow to the reach from tributary streams is estimated if the tributary flow is relatively small. If the tributary streams are large they are gauged upstream from the head of tide to provide a continuous record of freshwater inflow, just as is done with the principal inflow stream.

The method of cubatures is not only cumbersome for use but the discharge figures obtained are only rough approximations of the true values because of the large errors inherent in computing the storage component of the continuity equation.

4.7.2 **Rating-fall method**

Stage-fall-discharge relations have been used successfully for rating tide-affected streams where acceleration head is a minor factor. The rating-fall method that is discussed in detail in Chapter 3 is used for that purpose. Acceleration head is often a minor factor where the slope reach is located at the upper end of an estuary near the head of tide. Consequently it is usually only at or near such locations that the rating-fall method can be used successfully.

4.7.3 **Tide-correction method**

The tide-correction method assumes that a direct proportionality exists between the cyclic range in stage observed at any two points within a tidal reach. Based on that assumption a relation of mean discharge for a tidal cycle to mean stage for a tidal cycle is developed for the base-gauge site. In calibrating that relation, the mean discharge for a tidal cycle, obtained by averaging several individual measurements made one to two hours apart throughout the cycle, is plotted against adjusted mean stage at the base gauge. The adjustment applied to the mean stage at the base gauge is determined from the difference, at the secondary gauge, between observed mean stage and the stage which is presumed to exist under conditions of least tide fluctuation. That difference, D, is multiplied by the ratio of the stage range at the base gauge to the stage range at the secondary gauge. The product is the stage adjustment required at the base gauge. In practice the secondary stage observations are frequently made at a nearby ocean inlet. Mean sea level is assumed to represent the condition of least tidal fluctuation, and therefore, if all gauges have their datum's set to mean sea level, D is always equal to the mean stage for a tidal cycle at the secondary gauge. Essentially the tide-correction method attempts to approximate the stage which would occur for a particular steady-flow-discharge under a fixed backwater condition.

The tide-correction method of rating a tide-affected stream may be used where reverse flows occur during a part of reach tide cycle because the mean discharge for the cycle is the value used in the computation. It is also applicable to a reach of tidal waterway on which both observation stations are upstream from the mouth of the waterway. Mean-cycle discharge obtained from the rating curve can be plotted against mean-cycle time on a hydrograph sheet, and after connecting the points by straight lines the daily mean discharges can be determined.

The tide-correction method has been satisfactory, though cumbersome, for computing the daily discharge of tide-affected canals in Florida, United States, but efforts to adopt the method for use elsewhere in the United States have generally been unsuccessful.

4.7.4 Coaxial rating-curve method

The coaxial method of graphical correlation to determine discharge in a tidal reach (Rantz, 1963) was developed to fill the need for a simple method of making reasonably accurate on-the-spot determination of streamflow. A method of this kind is required, for example, in the operation of a sewage plant discharging its effluent into a tide-affected stream. The method that was developed, which is basically a graphical method of solving the equations of unsteady flow, fills this need in that readings from a pair of stage gauges can be used to determine momentary discharge directly from a set of rating curves. However, the method is too cumbersome for use in computing a continuous record of discharge for a gauging station. Solution of the theoretical equations of unsteady flow as described in previous sections of this chapter is much better for the latter purpose.

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CHAPTER 5

DISCHARGE RATINGS FOR MISCELLANEOUS HYDRAULIC FACILITIES

5.1 INTRODUCTION

This chapter includes specialized problems in establishing discharge ratings for various hydraulic facilities, using techniques that are not specifically described in Chapters 1 to 4 . In addition, a general description of a computer program that integrates several of the structure ratings into an automated program for computation of flow through a lock and dam structure is presented in this chapter. The hydraulic facilities and computer program are discussed under the following principal headings:
(a) Dams with movable gates;
(b) Navigation locks;
(c) Pressure conduits;
(d) Urban storm drains;
(e) Automated computation of flow through water control structures.

5.2 DAMS WITH MOVABLE GATES

5.2.1 General

Dams are commonly equipped with movable gates for better control of pool stage and outflow. As such a general rule is that are not rated, Instead the channel downstream is rated by the most practicable method. The rating method may be a simple stage-discharge relation (Chapter 1), a velocity-index rating (Chapter 2), a stage-fall-discharge relation (Chapter 3) or by use of the ultrasonic (acoustic) method (Volume I, Chapter 6). However, in some situations none of those rating methods may be satisfactory. For example, consider a river controlled by a series of low navigation dams. In that situation the river profile resembles a huge staircase; successive pools separated by dams. The movable dam crests negate the use of a simple stage-discharge relation. The slope of the water surface in the pools may be too flat for a stage-fall-discharge relation; and velocities may be too low for accurate evaluation by the acoustic Doppler or ultrasonic methods. In that situation the most practicable method of obtaining a continuous record of discharge is to calibrate the flow through or over the movable gates. If boat traffic is heavy and natural inflow is light, a significant part of the discharge may be the flow released through the navigation locks and the lockages must likewise be calibrated. In some cases, computation of turbine flow may also be required.

Calibration of the gates by discharge measurements during periods of small releases of water may be extremely difficult. If boat lockages are infrequent, standard current-meter measurements made downstream by boat, using a low-velocity meter may be adequate. If boat lockages are frequent, the surges in discharge attributable to the lockages may cause unsteady and non-uniform flow conditions downstream. Discharge measurements must then be made as rapidly as possible under conditions that are not conducive to accurate results. A rapid discharge measurement may be made by the moving-boat method using the Acoustic Doppler Current Profiler (ADCP) as described in Volume I, Chapter 6, or by use of a bank of current meters operated from a bridge as described in Volume I, Chapter 5. If velocities are too low for accurate measurement by either of those two methods and if only small quantities of water are being released under the dam-crest gates, the best course of action might be to use the volumetric method discussed in Volume I, Chapter 8 for measuring flow over a dam crest. In using the volumetric method, the barge carrying the calibration tank is kept in place not only by lines operated from the banks but also by an outboard motor on the barge to keep the barge from drifting downstream. The difficulty of measuring low flow under the conditions described above is apparent. At those times it may also be difficult to determine the actual head on the gates because lockages often cause longitudinal seiche-like waves to traverse the gauge pool and those waves travel back and forth over the length of the pool for a considerable period of time.

The flow at movable dam-crest gates may be placed in two general categories: (a) weir flow over the gate or dam crest and (b) orifice flow under the gate. Each of those types of flow may be either free or submerged, depending on the relative elevations of headwater, tailwater and pertinent elements of the dam crest or gate. Listed below are the crest gates that will be discussed:
(a) Drum gates;
(b) Radial or Tainter gates;
(c) Vertical lift gates;
(d) Roller gates;
(e) Movable dams:
  (i) Bear-trap gates;
  (ii) Hinged-leaf gates;
  (iii) Wickets;

(iv) Inflatable dams;
(f) Flashboards;
(g) Stop logs and needles.

A gated dam usually has several gates along its crest. The gates are installed in bays that are separated by piers. All other conditions being equal, the discharge through a single gate, when adjacent gates are open, will be about five per cent greater than the discharge through that same gate when adjacent gates are closed. The various types of gates should be calibrated by discharge measurements. But as an aid to shaping the calibration curves, experimental ratings, where available, are given in the text that follows.

Discharge measurements for the purpose of determining gate coefficients will almost always be made in the downstream channel and will include the flow for all the gates that are open. Furthermore, for given stages upstream and downstream from the gates, the gate coefficient will commonly vary with the gate position or opening. Consequently, if discharge is to be measured with more than one gate open, arrangements should be made, if possible, for all gates to be positioned identically. If the differences in the positioning of the gates are minor and if the gate coefficient does not vary significantly with its positioning, a discharge measurement may be made. For computation of the gate coefficient an average gate position will be assumed for each of the bays carrying flow.

5.2.2 **Drum gates**

A drum gate consists of a segment of a cylinder which, in the open or lowered position, fits in a recess in the top of the spillway. When water is admitted to the recess, the hollow drum gate is forced upward to a closed position. One type of drum gate (Figure II.5.1 (a)) is a completely enclosed gate hinged at the upstream edge. Buoyant forces aid in its lifting. This type of gate is adapted to automatic operation and also conforms closely to the shape of the ogee crest when lowered. A second type (Figure II.5.1(b)) has no bottom plate and is raised by water pressure alone. Because of the large recess required by drum gates in the lowered position they do not adapt well to small dams.

With regard to its calibration, the drum gate resembles a thin-plate weir with a curved upstream face over the greater part of its travel. Given an adequate positioning indicator, the drum gate can serve as a satisfactory stream-gauging control. Its use for that purpose has been investigated by Bradley (1953) and the discussion that follows is taken almost verbatim from Bradley's paper dealing with a drum gate of the type shown in Figure II.5.1(a).

When the drum gate simulates a thin-plate weir, that is, when a line drawn tangent to the downstream lip of the gate makes a positive angle with the horizontal, as shown in Figure II.5.2(a), four principal factors are involved. These factors are H, the total head above the high point of the gate; θ, the angle between the horizontal and a line drawn tangent to the downstream lip of the gate; r, the radius of the gate, or an equivalent radius if the shape of the gate is parabolic and C_q, the coefficient of discharge in the following equation:

$$Q = C_q L H^{3/2} \qquad (5.1)$$

where Q is discharge ($m^3 s^{-1}$) and L is length of the gate (m) normal to the discharge.

The velocity in the approach section was not included as a variable because the drum-gate installations studied were on high dams where approach effects were negligible. It has been shown that when the approach depth measured below the high point of the gate is equal to or greater than

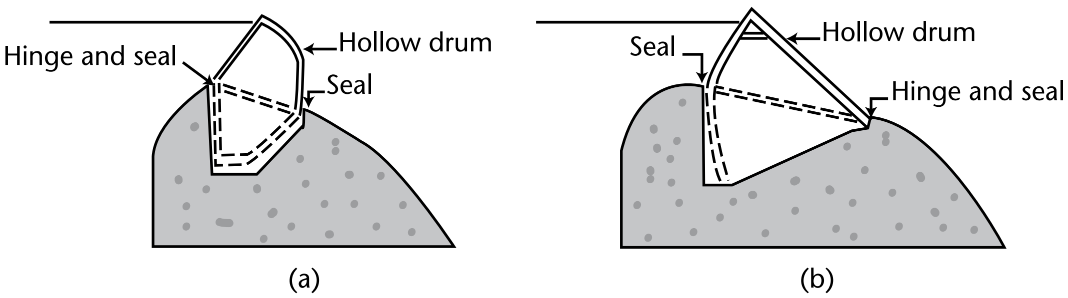


Figure II.5.1. Two types of drum gate

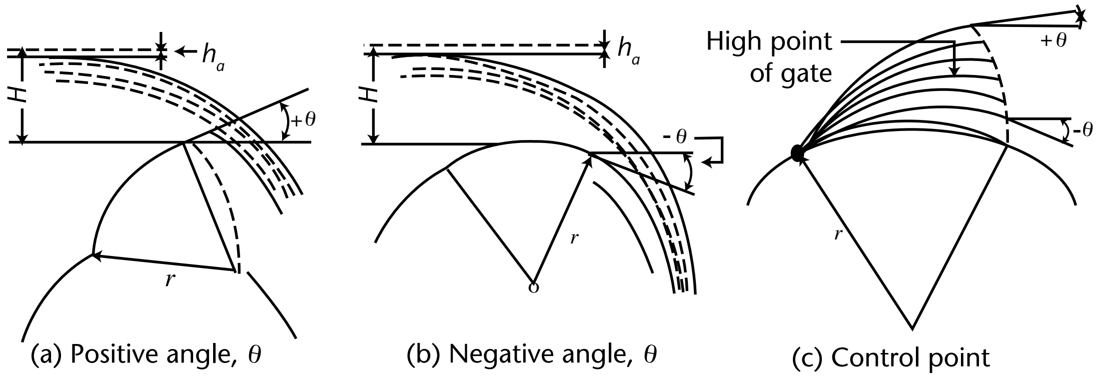


Figure II.5.2. Drum-gate positions (after Bradley, 1953)

twice the head on the gate, further increases in the approach depth produces little change in the coefficient of discharge. Most drum-gate installations are on dams that meet the above depth criterion, particularly when the gate is in a raised position. Therefore, in the usual case of adequate approach depth, the four variables, H, θ, r and C_q completely define the flow over this type of gate when angle θ is positive (Figure II.5.2(a)).

For negative values of θ (Figure II.5.2(b)) the downstream lip of the gate no longer controls the flow. In that situation the control point shifts upstream to the vicinity of the high point of the gate for each setting, as illustrated in Figure II.5.2(c), and flow conditions gradually approach those of the free crest as the gate is lowered. Although other factors enter the problem, similitude in the computation exists down to an angle of about – 15°.

Experimentation with eleven drum gates produced the family of curves for C_q shown in Figure II.5.3. The discharge coefficients in the region between θ = – 15° and the gate completely down are determined by graphical interpolation, a method that will be explained in the example that follows. The effect of submergence of the drum gate on C_q was not investigated because drum gates are invariably used on high dams and the probability of submergence is negligible. The data to be continuously recorded for computing discharge over rated drum gates are reservoir stage and the indication of drum-gate position for each gate.

The method of rating a drum gate on a round-crested weir will now be demonstrated using as an example the plan and spillway cross-section of Black Canyon diversion dam in Idaho, United States of America (Figures II.5.4 and II.5.5). The first step is the determination of the design head of the dam and the corresponding discharge coefficient for the free crest. That is done in accordance with the technique described under the heading *Nappe-fitting method* in the United States Geological Survey (USGS), Manual on computing peak discharge at dams (Hulsing, 1967, pp. 13-23). If a discharge measurement has been made under the condition of flow over a free crest, the results of the measurement are used to check the value of design head and design-head coefficient, using the technique described under the heading *Index-measurement method* in the previously cited Manual by Hulsing (1967, pp. 23-24). The design head (H_0)

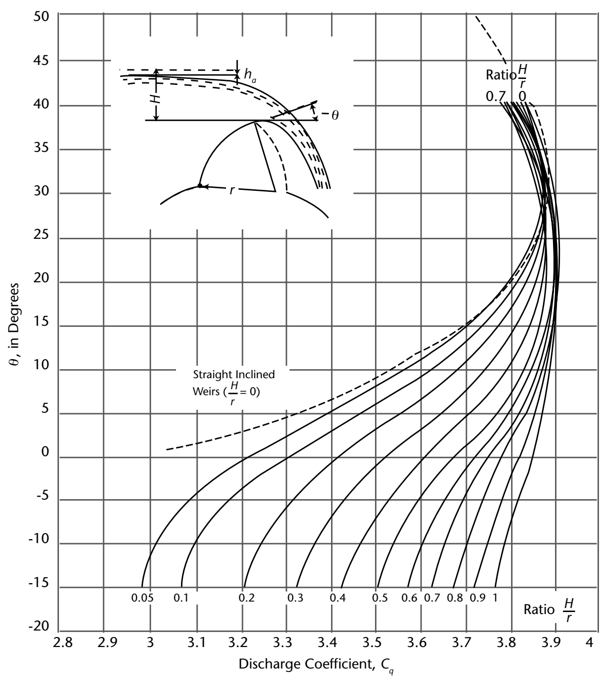


Figure II.5.3. General curves for the determination of discharge coefficients (after Bradley, 1953)

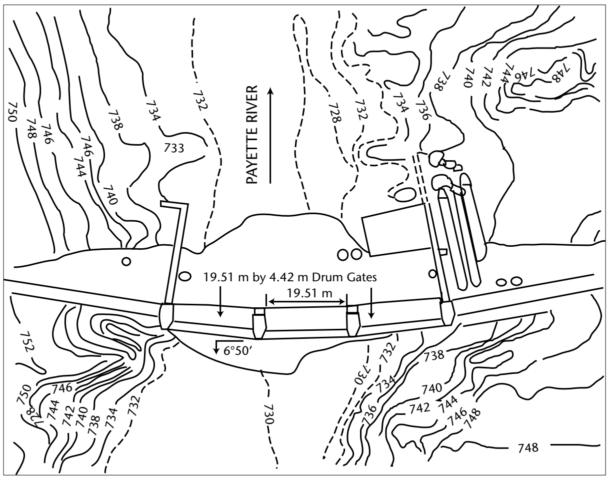


Figure II.5.4. Plan of Black Canyon Diversion Dam in Idaho, United States (after Bradley, 1953)

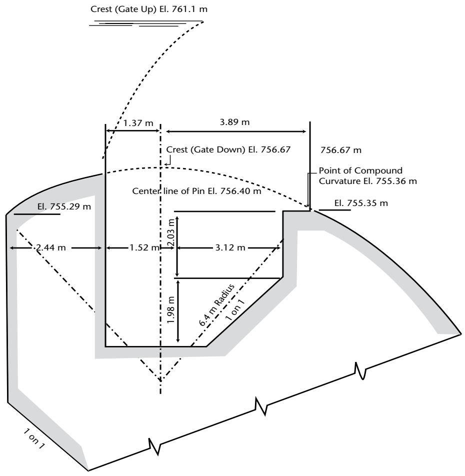


Figure II.5.5. Spillway crest details, Black Canyon Dam in Idaho, United States (after Bradley, 1953)

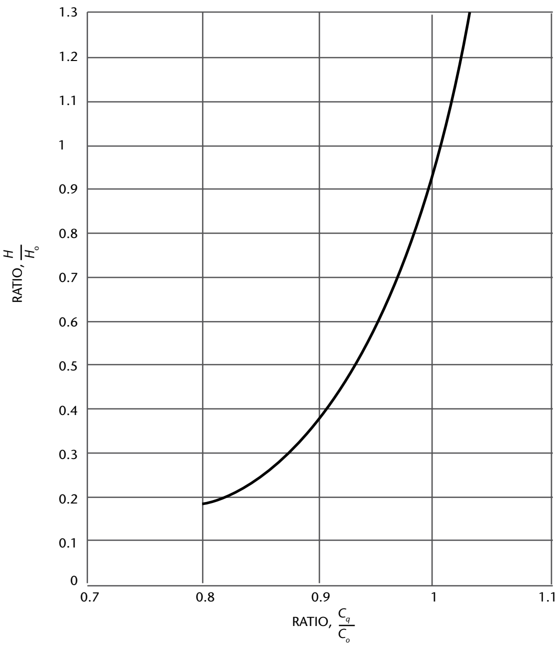


Figure II.5.6. Diagram for determining coefficients of discharge for heads other than the design head (after Bradley, 1953)

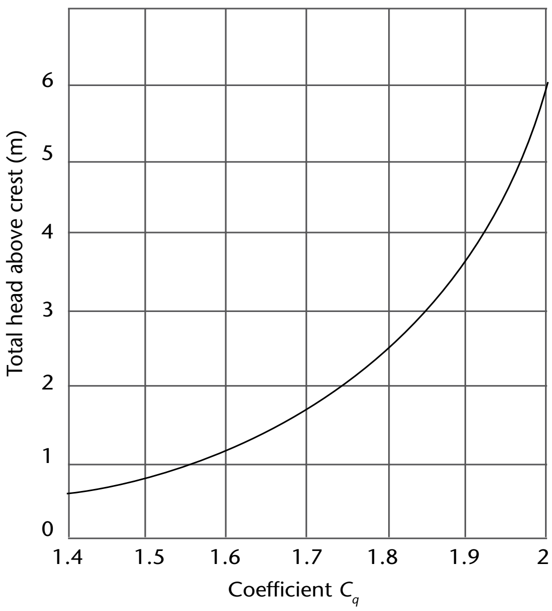


Figure II.5.7. Head coefficient curve, Black Canyon Dam, Idaho, United States (after Bradley, 1953)

of Black Canyon diversion dam was found to be 4.35 m and the corresponding coefficient of discharge (C_0) was found to be 1.92 (C_d = 0.613).

With the coefficient of discharge known for free flow at the design head, the entire free-flow coefficient curve can be established by use of Figure II.5.6. The free-flow coefficient curve for the spillway of Black Canyon diversion dam (H_0 = 4.35 m; C_0 = 1.91) is constructed by arbitrarily assuming several values of H/H_0 and reading the corresponding values of C/C_0 in Figure II.5.6. The method of computation is illustrated in Table II.5.1, and the head-coefficient curve for free flow (gate down) obtained in that manner is shown in Figure II.5.7.

Before considering the rating of the spillway with gates in raised positions it is necessary to construct a diagram, such as that shown in Figure II.5.8, to relate gate elevation to the angle θ for the Black

Table II.5.1. Head and discharge computations for a free crest (Black Canyon Dam in Idaho, United States)

Total head H, in metres	*Reservoir elevation, in metres*	*Ratio[a] H/H_o*	*Ratio[b] C_q/C_o*	*Coefficient C_q*	*Q in cu m per sec.[c]*
(1)	(2)	(3)	(4)	(5)	(6)
5.182	761.848	1.172	1.020	1.963	451.65
4.877	761.543	1.104	1.012	1.944	408.33
4.420	761.086	1.0	1.0	1.921	348.18
3.658	760.324	0.827	0.980	1.883	256.89
3.048	759.714	0.690	0.960	1.844	191.39
2.438	759.104	0.552	0.940	1.806	134.11
1.829	758.495	0.414	0.905	1.731	83.51
1.219	757.885	0.276	0.850	1.633	42.87
0.914	757.580	0.207	0.815	1.566	26.70
0.610	757.276	0.138	0.760	1.458	13.54

[a] H = 4.420 m [b] C_o = 1.92 [c] The discharge for one gate: $Q = C_q L H^{3/2}$, in which L = 19.507 m

Canyon Dam gate. The tabulation in Figure II.5.8 shows the angle θ for corresponding elevations of the downstream lip of the gate at intervals of 0.6 m.

Beginning with the maximum positive angle of the gate, which is 34.883°, the computation may be started by choosing a representative number of reservoir elevations as indicated in column 2 of Table II.5.2. The difference between the reservoir elevation and the high point of the gate constitutes the total head on the gate and values of head are recorded in column 3. Column 4 shows these same heads divided by the radius of the gate, which is 6.3 m.

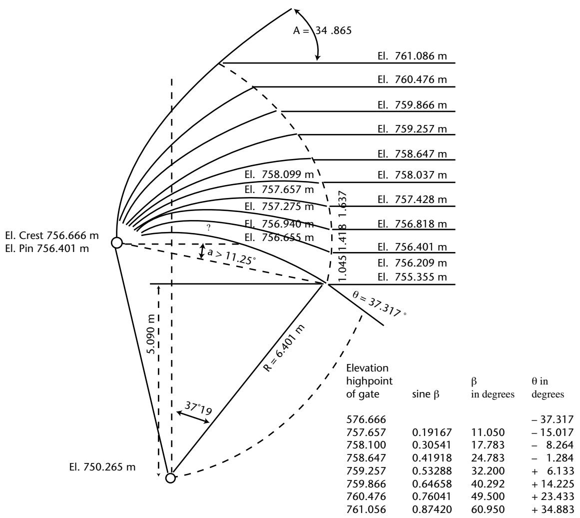


Figure II.5.8. Relation of gate elevation to angle θ (after Bradley, 1953)

The discharge coefficients listed in column 5 of Table II.5.2 of the set of computations designated A, are obtained by entering the curves in Figure II.5.3 with the values in column 4 for θ = + 34.883°. The remainder of the procedure outlined in columns 6 and 7 of Table II.5.2 consists of computing the discharge for one gate from equation 5.1. A similar computation procedure is repeated for other positive angles of θ, as in sets B, C and D of Table II.5.2.

For positive values of angle θ the high point of the gate is the downstream lip of the gate. As the angle θ decreases to negative values the high point of the gate is no longer the downstream lip. In determining the discharge for negative values of θ between 0° and – 15°, the procedure remains the same as was used for positive values of θ, but as mentioned above, the controlling difference between reservoir elevation and high point of the gate is no longer the head above the downstream lip. (see Figure II.5.8.) Discharge computations for negative angles down to – 15.017° are tabulated in sets E, F and G of Table II.5.2.

The plotting of values of discharge, reservoir elevation and gate elevation from Table II.5.2, results in the seven curves in Figure II.5.9 that bear the plotted points, shown by closed circles. The

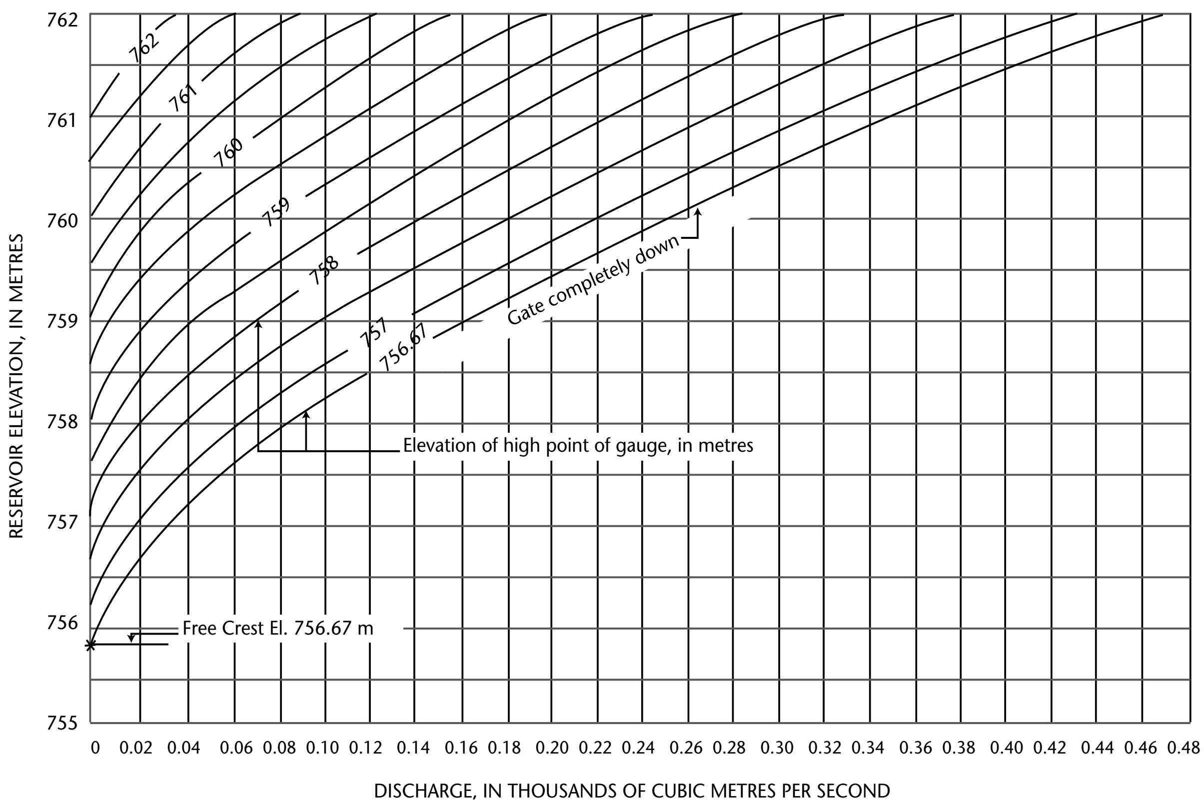


Figure II.5.9. Rating curves for Black Canyon Dam drum spillway in Idaho, United States (after Bradley, 1953)

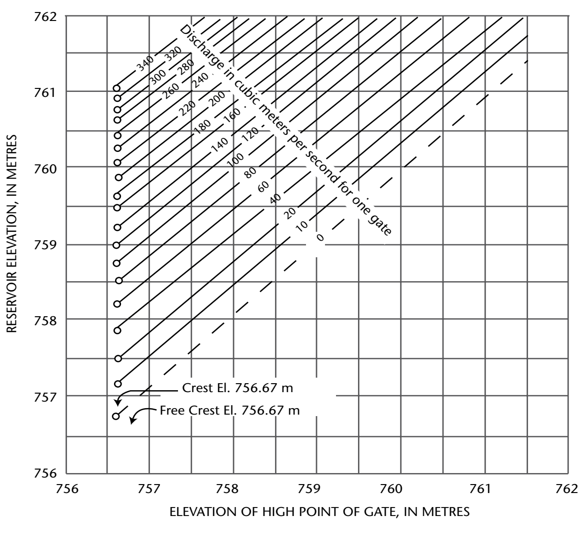


Figure II.5.10. Cross-plotted initial rating curves, Black Canyon Dam, Idaho, United States (after Bradley, 1953)

extreme lower curve, which bears plotted points shown by X's, represents the discharge of the free crest with the gate completely down. The plotted points represent values obtained from Table II.5.1.

The discharge values shown in Figure II.5.9 are for one gate only. When more than one gate is in operation the discharges from the separate gates may be totaled, providing the gates are each raised the same amount. The experimental models used in this study had from one to eleven gates operating, so that a reasonable allowance for pier effect on the discharge is already present in the results.

The intervals between the eight curves in Figure II.5.9 that are identified by plotted points are too great for rating purposes, particularly the gap between gate elevations 757.6 and 756.7 m. That deficiency is remedied by cross-plotting the eight curves for various constant values of discharge, as shown in Figure II.5.10. Fortunately the result is a straight-line variation for any constant value of discharge. The lines in Figure II.5.10 are not quite parallel and there is no assurance that they will be straight for every drum gate. Nevertheless, this uncertainty will not detract appreciably from the accuracy obtained. Interpolated information from Figure II.5.10 is then utilized to construct the additional curves in Figure II.5.9. Figure II.5.9 now shows the rating for the Black Canyon Dam spillway for gate intervals of 0.150 m. For intermediate values, straight-line interpolation is permissible.

Table II.5.2. Head and discharge computations for drum gates in raised positions

Set	*Reservoir elevation in metres*	*H in metres*[a]	*Ratio H-Y*	*Coefficient C_q*	*$H^{3/2}$ in metres*	*Q in m^3/sec*[b]
1	*2*	*3*	*4*	*5*	*6*	*7*
GATE ELEVATION 760.476; θ = + 34.88°						
A	761.390	0.305	0.048	2.131	0.168	6.99
	761.695	0.610	0.095	2.132	0.476	19.79
	762.000	0.914	0.143	2.130	0.874	36.33
GATE ELEVATION 760.476; θ = + 23.43°						
B	760.781	0.305	0.048	2.122	0.168	6.97
	761.086	0.610	0.095	2.129	0.476	19.76
	761.390	0.914	0.143	2.127	0.874	36.27
	761.695	1.219	0.190	2.134	1.346	56.04
	762.000	1.524	0.238	2.137	1.881	78.44
GATE ELEVATION 759.866; θ = + 14.22°						
C	760.171	0.305	0.048	2.036	0.168	6.68
	760.476	0.610	0.095	2.059	0.476	19.11
	760.781	0.914	0.143	2.070	0.874	35.31
	761.390	1.524	0.238	2.098	1.881	76.99
	762.000	2.134	0.333	2.120	3.117	128.90
GATE ELEVATION 759.257; θ = + 6.13°						
D	759.562	0.305	0.048	1.915	0.168	6.29
	759.866	0.610	0.095	1.937	0.476	17.98
	760.171	0.914	0.143	1.971	0.874	33.61
	760.781	1.524	0.235	2.004	1.881	73.54
	761.390	2.134	0.333	2.043	3.117	124.20
	762.000	2.743	0.429	2.082	4.543	184.48
GATE ELEVATION 667.207; θ = – 1.28°						
E	758.952	0.305	0.048	1.768	0.168	5.80
	759.257	0.610	0.095	1.812	0.476	16.82
	759.562	0.914	0.143	1.844	0.874	31.46
	760.171	1.524	0.228	1.905	1.881	69.91
	760.781	2.134	0.333	2.957	3.117	118.99
	761.390	2.743	0.429	2.004	4.543	177.63
	762.000	3.353	0.524	2.040	6.139	244.29
GATE ELEVATION 758.0.98; θ = – 8.28°						
F	758.342	0.244	0.038	1.664	0.120	3.91
	758.647	0.549	0.086	1.711	0.406	13.56
	758.952	0.853	0.133	1.749	0.788	26.90
	759.562	1.463	0.229	1.828	1.770	63.12
	760.171	2.073	0.324	1.893	2.984	110.21
	760.781	2.682	0.419	1.937	4.393	166.02
	761.390	3.292	0.515	1.976	5.973	230.24
	762.000	3.901	0.610	2.007	7.706	301.66

(*continued*)

Set	*Reservoir elevation in metres*	*H in metres*[a]	*Ratio H-Y*	*Coefficient C_q*	*$H^{3/2}$ in metres*	*Q in m^3/sec*[b]
1	*2*	*3*	*4*	*5*	*6*	*7*
GATE ELEVATION 757.657; $\theta = -15.02°$						
G	758.038	0.381	0.060	1.654	0.235	7.59
	758.342	0.686	0.107	1.695	0.568	18.77
	758.647	0.991	0.155	1.739	0.986	33.44
	759.257	1.600	0.250	1.808	2.024	71.42
	759.866	2.210	0.315	1.863	3.285	119.38
	760.476	2.819	0.440	1.913	4.734	176.64
	761.086	3.429	0.536	1.954	6.350	242.05
	761.695	4.039	0.631	1.985	8.116	314.23

[a] *H* is the total head on the gate. [b] The discharge for one gate: $Q = C_q L H^{3/2}$

5.2.3 Radial or Tainter gates

The damming face of a radial or Tainter gate is essentially a segment of a hollow steel cylinder spanning between piers on the dam crest. The cylindrical segment is supported on a steel framework that pivots on trunnions embedded in the downstream part of the piers. The gate is raised or lowered by hoisting cables that are attached to each end of the gate. The cables lead to winches on a platform above the gate. In its closed position the lower lip of the gate rests on the dam crest.

Radial gates on a horizontal surface

Experimental work has been performed to determine discharge coefficients for radial gates that control flow along a horizontal surface (Toch, 1953). The results of those experiments are shown in Figures II.5.11 to II.5.14. Figure II.5.11 is a definition sketch for a radial gate on a horizontal surface. The discharge coefficient, C_d, is defined as:

$$C_d = \frac{q}{b(2gh_0)^{1/2}} \tag{5.2}$$

where q is discharge per unit width of the gate, g is acceleration of gravity and h_0 and b are elements shown in the definition sketch (Figure II.5.11).

Figures II.5.12 to II.5.14 show values of C_d for three values of the ratio, a/r, where a is trunnion elevation and r is gate radius. In the relations shown in the three figures, all pertinent elements have been made dimensionless by using gate radius, r, as a reference. Thus the relative headwater depth is h_0/r, the relative tailwater depth is h_2/r, the relative height of opening is b/r and the relative trunnion height is a/r. Free efflux (flow) occurs when $h_2 < b$; submerged efflux occurs when $h_2 > b$. Each of the three graphs shows values of the coefficient of discharge for:

(a) Free efflux for three values of b/r;
(b) Submerged efflux for two values of b/r when $h_2/r = 0.5$;
(c) Submerged efflux for three values of b/r when $h_2/r = 0.7$.

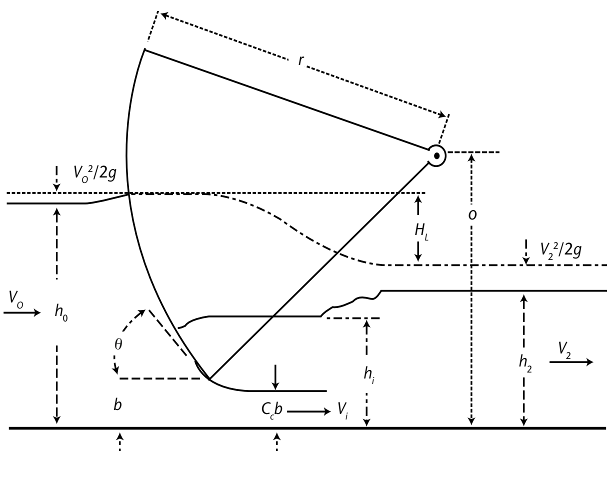


Figure II.5.11. Definition sketch of a radial gate on a horizontal surface (after Toch, 1953)

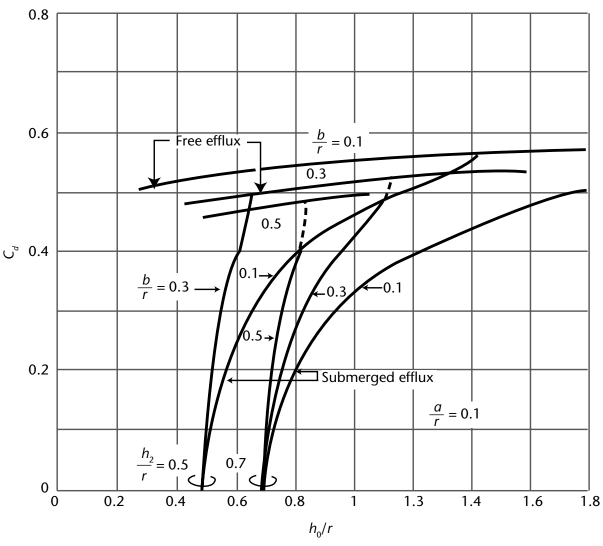


Figure II.5.12. Coefficient of discharge for free and submerged efflux, $\frac{a}{r}$ = 0.1 (after Toch, 1953)

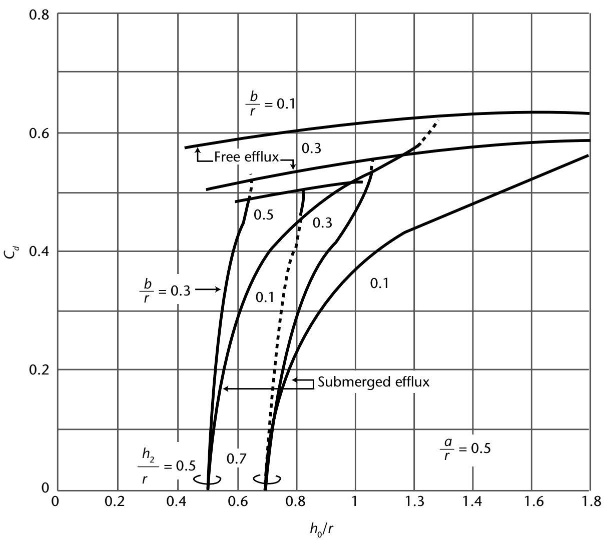


Figure II.5.13. Coefficient of discharge for free and submerged efflux, $\frac{a}{r}$ = 0.5 (after Toch, 1953)

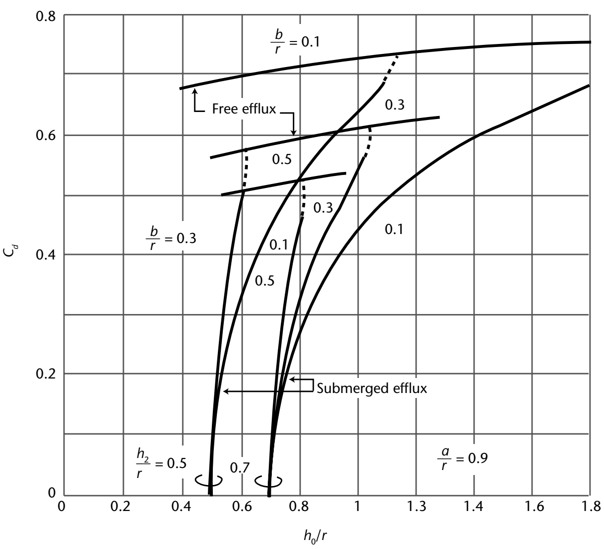


Figure II.5.14. Coefficient of discharge for free and submerged efflux, $\frac{a}{r}$ = 0.9 (after Toch, 1953)

Radial gates on a curved dam crest or sill

More commonly radial gates are used to control the flow over a curved dam crest or over a sill. The discharge coefficients determined for a radial gate on a horizontal surface cannot be transferred to a radial gate on a curved dam crest or sill because of differences in the pressure distribution. The flow under radial gates on a curved crest or sill is controlled by the geometry of three interrelated variables: the crest shape, the gate and the gate setting. Major factors that influence the discharge relations are the position of the gate-seal point with respect to the highest point of the spillway crest and the curvature of the upstream face of the gate. Therefore experimentally derived discharge coefficients for various prototype dams cannot be transferred to other installations unless the several variables involved are similar. Consequently, radial gates will invariably require rating by current-meter discharge measurements.

When radial gates control the flow over a sill or a curved dam crest, five flow regimes may occur:
(a) Free orifice flow;
(b) Submerged orifice flow;
(c) Free weir flow;
(d) Submerged weir flow;
(e) Flow over closed radial gates.

Figure II.5.15 is a definition sketch for the discussions that follow, all of which are concerned with only a single gate. As already mentioned, when discharge measurements for calibration purposes are made with several gates open it is highly desirable that all gate openings be identical, unless of course the gates are all raised sufficiently for their lower lips to be clear of the water. If gate openings are variable under the condition of orifice flow it will be necessary to use an average gate opening in computing discharge coefficients for the gates from the measured discharge.

Definitions of symbols used in the sketch in Figure II.5.15 are: a = elevation difference, trunnion centreline to sill; c = elevation difference, gate reference point (R.P.) to sill; d = elevation difference, gate R.P. to sill with the gate in a closed position;

h_1 = static headwater referenced to gate sill; h_3 = static tailwater referenced to gate sill; h_g = vertical gate opening; r = radius from trunnion centreline to gate R.P.; R = radius from trunnion centreline to upstream face of a Tainter gate; *R.P.* = reference point used as indicator of gate position; $\Delta h = h_1 - h_3$ = static head loss through structure; θ = included angle between radial lines from the trunnion centreline through the R.P. and through the lower lip of the gate; ϕ_L = the angle measured from horizontal to the radial line from the trunnion centreline through the lower lip of the gate with the gate in a closed position; and ϕ_U = the angle measured from horizontal to the radial line from the trunnion centreline through the gate R.P.

Additional symbols used in the text: L = lateral gate length (normal to flow); Q = discharge for one gate; g = acceleration of gravity.

Free orifice flow

Free orifice flow occurs when the lower lip of the raised gate is submerged by headwater but is above the elevation of tailwater. When the radial gate is on a sill, as in Figure II.5.15, free orifice flow occurs under the gate when h_g is less than $2/3/h_1$ and h_3 is less than h_g. Discharge for that condition is computed from the equation:

$$Q = Ch_g L(2gh_1)^{1/2} \tag{5.3}$$

The definition of symbols is given in Figure II.5.15.

Values of C will vary inversely with h_g because the change in slope of the lower lip of the gate, as the gate is raised, progressively decreases the hydraulic efficiency of the orifice. There is also a tendency for C to increase with h_1, particularly at low stages, but that effect is usually minor compared to the effect of h_g. Consequently C can usually be related to h_g alone. In developing the relation, discharge measurements should be made throughout the expected range of h_g and h_1. Values of C are then plotted against h_g and the plotted points are fitted with a smooth curve. For convenience in later computations of discharge, the ordinates of the curve are put in tabular form.

The vertical gate opening, h_g, is computed from the following equation based on gate geometry and the position of the reference point at various gate settings:

$$h_g = R\cos\theta\left(\frac{c-a}{r}\right) + a - R\sin\theta\sqrt{1-\left(\frac{c-a}{r}\right)^2},$$

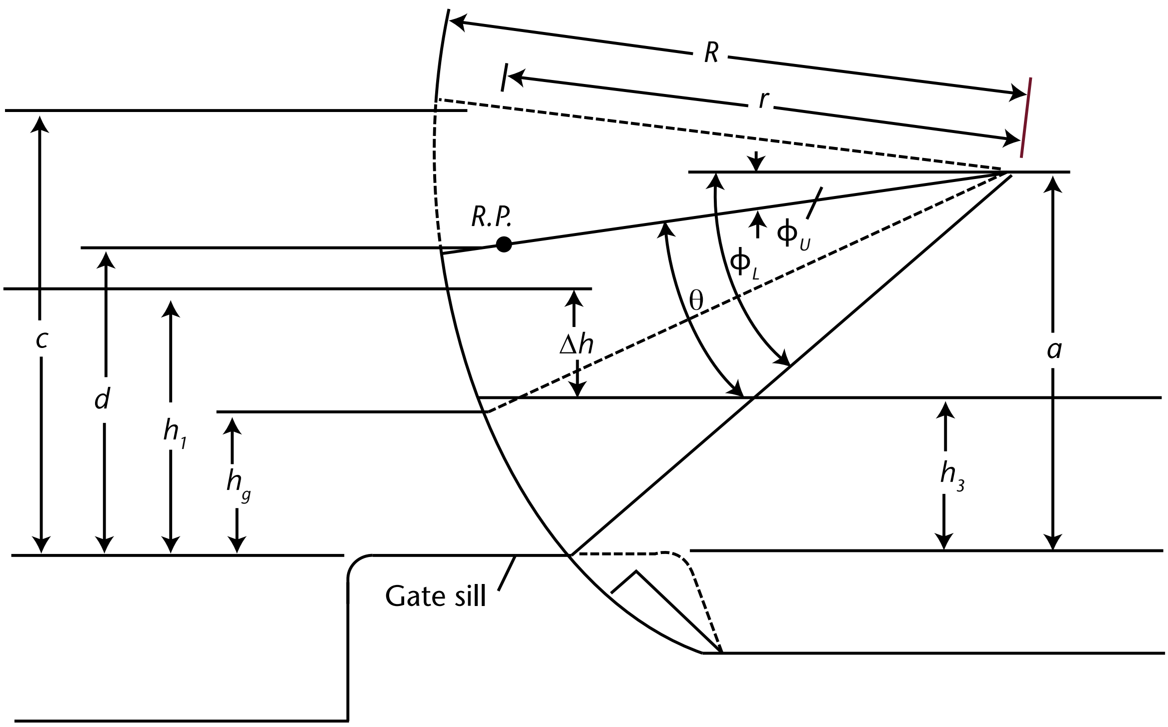


Figure II.5.15. Definition sketch of a radial or Tainter gate on a sill

where

$$\theta = \varphi_L - \varphi_U = \sin^{-1}\left(\frac{a}{R}\right) - \sin^{-1}\left(\frac{a-d}{r}\right)$$

Because C does not vary linearly with h_g it is highly desirable, and often necessary, that all gates be positioned identically during a discharge measurement to avoid the necessity of using an average value of h_g in the computation of C.

Submerged orifice flow

Submerged orifice flow occurs when the lower lip of the raised gate is submerged by both headwater and tailwater. When the radial gate is on a sill, as in Figure II.5.15, submerged orifice flow occurs when h_3 is greater than h_g, and h_g is less than $2/3h_1$. The basic equation for computing discharge is:

$$Q = C_{gs} h_g L (2g\Delta h)^{1/2} \quad (5.4)$$

Values of C_{gs} are determined from discharge measurements, and in addition, values of h_3/h_g and h_3/h_1 are computed for each measurement. For calibration purposes it is desirable to have measurements that cover the range of 1 to 100 for the ratio h_3/h_g, with several in the range of 1 to 2. The value of C_{gs} is a function of h_g, h_1 and h_3, and the complexity of that function depends on the geometry of the hydraulic structure. The geometry may be such that all computed values of C_{gs} show little variation from a mean value When that occurs the mean value of C_{gs} is used in equation 5.4.

However, computed values of C_{gs} will often vary, particularly in the range of 1 to 2 for the ratio h_3/h_g. If that occurs three relations involving C_{gs} are plotted graphically and the one that best fits the plotted points is selected for use. The three relations are: C_{gs} versus h_g; C_{gs} versus h_3/h_1, and C_{gs} versus h_3/h_g.

Quite often the last of the three relations will show the best fit. It will plot as a straight line on logarithmic graph paper and have the general equation:

$$C_{gs} = K(h_3/h_g)^B \quad (5.5)$$

When equation 5.5 is substituted in equation 5.4, the result is:

$$Q = K(h_3/h_g)^B h_g L (2g\Delta h)^{1/2} \quad (5.6)$$

Ordinates of the relation indicated by equation 5.5 are put in tabular form for convenience in later computations of discharge. Because C_{gs} does not vary linearly with h_g it is highly desirable, and often necessary, that all open gates be positioned identically during a discharge measurement to avoid the necessity of using an average value of h_g in the computation of C_{gs} from measured discharge.

Free weir flow

Weir flow will occur when the lower lip of the gate is above the water surface. When the radial gate is on a sill, as in Figure II.5.15, weir flow will occur when h_g is greater than $2/3h_1$ because of drawdown of the water surface at the dam crest. The lower lip of the gate will then be above the water surface. Whether the weir flow is free or submerged will depend on the relative elevations of h_3 and h_1. Free weir flow will occur when the submergence ratio, h_3/h_1, is less than about 0.5 to 0.7, depending on the geometry of the weir crest. The discharge equation is:

$$Q = C_w L h_1^{3/2} \quad (5.7)$$

Values of C_w, which are dependent on the shape of the dam crest, are determined from discharge measurements, and the computed values are then plotted against h_1. Approach velocity head is usually negligible, but even where it is not, its effect is included in the variable coefficient, C_w. Measurements should be made at headwater (h_1) intervals of 0.3 to 0.6 m throughout the expected headwater range to establish the functional relation between C_w and h_1. Information contained in a previously cited report by Hulsing (1967) will usually be helpful as a guide to the probable shape of that relation.

Submerged weir flow

As mentioned above, weir flow is submerged when the submergence ratio h_3/h_1 is greater than about 0.5 to 0.7, depending on the geometry of the weir crest. The discharge equation for that condition is:

$$Q = C_w C_{ws} L H_1^{3/2} \quad (5.8)$$

where C_w is the coefficient previously determined from equation 5.7. Values of C_{ws} must be determined from discharge measurements and expressed as a function of h_3/h_1. Satisfactory definition of the functional relation will probably require 10 to 12 discharge measurements well distributed over the range of h_3/h_1. Information contained in the Hulsing report (1967) will often be helpful in the analysis. If the submergence is greater than 0.95 for much of the time it may be advisable to attempt to develop a relation of discharge to tailwater stage for use during periods of excessive submergence.

Flow over closed radial gates

At extremely high flows the closed radial gate may be overtopped, at which time the discharge over the gate is computed from the general weir equation:

$$Q = CLh^{3/2} \quad (5.9)$$

where h is the head on the upper lip of the gate. The gate itself will act as a thin-plate weir. Values of the discharge coefficient C will vary primarily with the geometry of the gate and with h. The geometry of the dam crest or sill will have a lesser effect on the value of C. Discharge measurements will be required to define the rating for flow over the gate, both for unsubmerged flow (tailwater below the upper lip of the gate) and for submerged flow (tailwater above the upper lip of the gate).

Flow over a radial gate can also occur at low stages if the gate is of the submersible type. A submersible gate is designed to be lowered to allow flushing of upstream debris over the top of the gate. When so lowered, the bottom lip of the gate drops below the normal sill elevation. The upper surface of a submersible gate usually has an ogee or rounded crest.

5.2.4 **Vertical lift gates**

Vertical lift gates are simple rectangular gates of wood or steel spanning between piers on the dam crest. The gates move vertically in slots in the piers, and all but the smallest gates are mounted on rollers to reduce the friction caused by the hydrostatic force on the gate. The vertical lift gate, like the radial gate, must be hoisted at both ends, and the entire weight is suspended from the hoisting cables or chains (United States Army Corps of Engineers (USACE), 1952). Piers must be extended to a considerable height above high water to provide guide slots for the gate in the fully raised position. To reduce the height of the piers required for operating large vertical lift gates, the large gates are often built in two horizontal sections so that the upper section may be lifted and placed in another gate slot before raising the lower section. This design also reduces the load on the hoisting mechanism. Discharge may occur over either one or both sections of the gate or over the spillway crest. Discharge over the spillway crest may occur as weir flow if the gate is raised above the water surface or as orifice flow if the raised gate does not clear the water surface.

The principles that govern the rating of radial gates likewise apply to vertical-lift gates. When the elevation of the lower edge of the raised gate is less than two-thirds of the upstream head, orifice flow occurs. The orifice flow is free if the tail-water is below the lower edge of the raised gate. The orifice flow is submerged if the tail-water is above the lower edge. General equations 5.3 and 5.4 apply to the discharge and values of C and C_{gs} in those equations must be determined from discharge measurements.

If the elevation of the lower edge of a raised gate is greater than two-thirds of the upstream head, weir flow over the dam occurs. If the weir flow is free, equation 5.7 applies. If the elevation of the tailwater causes submergence effect, equation 5.8 applies. The coefficients in the two weir equations are primarily dependent on the shape of the weir crest. Values of the coefficients are determined from discharge measurements, but helpful information concerning them is found in a report by Hulsing (1967).

When a closed gate is overtopped by headwater, the upper edge of the gate acts as a weir and general equation 5.9 is applicable. The upper edge of a vertical-lift gate commonly has the shape of a modified horizontal broad-crested weir. Coefficients of discharge are determined from discharge measurements. Again helpful information is to be found in the Hulsing report (1967).

5.2.5 **Roller gates**

A roller (or rolling) gate as shown in Figure II.5.16 is a horizontal, internally braced, metal cylinder spanning between piers. Rings of gear teeth at the ends of the cylinder mesh with inclined metal racks supported by the piers. When a pull is exerted on the hoisting cable or chain, the gate rolls up the rack (Figure II.5.16(a)). The effective damming height of the cylinder can be increased by means of a projecting apron (Figure II.5.16(b)) which rotates into contact with the dam crest as the gate rolls down the inclined racks (USACE, 1952). A similar apron or rounded lip may be added to the top of the gate (Figure II.5.16(c)).

As in the case of radial and vertical-lift gates, orifice flow will occur under partly raised rolling gates, weir flow over the dam will occur when the gates are raised sufficiently (two-thirds or more of the headwater elevation) to be clear of the water surface, and weir flow over the gates will occur when the closed gates are overtopped by headwater. The principles of rating roller gates are similar to those discussed for radial gates and vertical-lift gates.

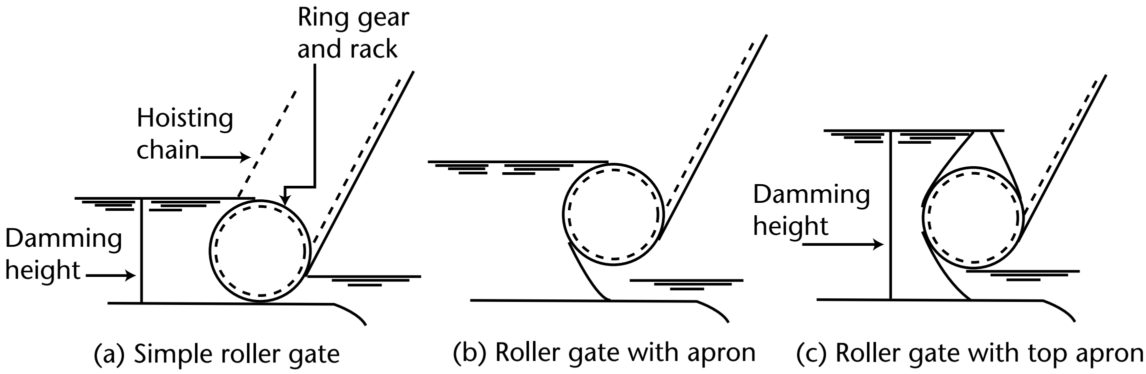


Figure II.5.16. Schematic sketches of roller gates (USACE, 1952)

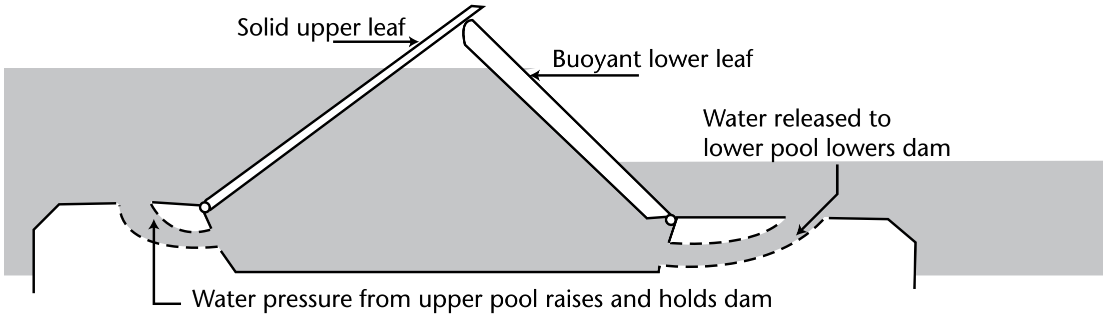


Figure II.5.17. Bear-trap gate (USACE, 1952)

5.2.6 Movable dams

A movable dam consists of a low concrete sill and a damming surface that can be raised above the water surface to maintain a desired pool level or lowered to the sill at higher discharges so as to offer no interference to the flow. The most commonly used gates or damming surfaces are bear-trap gates, hinged-leaf gates, wickets and inflatable dams.

Bear-trap gate

A bear trap gate (Figure II.5.17) consists of two leaves of timber or steel hinged and sealed to the dam or sill. When water is admitted to the space under the leaves they are forced upward. The downstream leaf is hollow so that its buoyancy aids the lifting operation. When the dam is collapsed by the release of water from under the leaves the leaves lie flat (USACE, 1952).

Hinged-leaf gate

A hinged-leaf gate (Figure II.5.18) is a rigid flat leaf hinged at bearings along its lower edge. In its raised position the leaf slopes upward and downstream at an angle of between 20° and 30° from the vertical. When lowered it lies in a nearly horizontal position. The position of the leaf is controlled by a mechanical hoist or by a counterweight device that causes the leaf to rise or fall automatically with a slight incremental change in headwater level.

Wickets

A wicket is a shutter held in position against the water load by a metal prop (Figure II.5.19(a)). It is not intended that water should flow over the wicket at an appreciable depth because the resultant water load will shift to a point above the prop and cause the wicket to overturn or vibrate violently (USACE, 1952). The metal prop, hinged at mid-length of the wicket, sits against a shoulder on a metal fixture (hurter) embedded in the foundation. The wicket is raised by an upstream pull on a hoisting line attached to the bottom of the wicket. This causes the prop to fall into its seat, after which the wicket is rotated into position against the sill (Figure II.5.19(b)). The wicket is lowered by pulling upstream on a line attached to the top of the wicket. The base of the prop is pulled away from its seat and falls to one side into a groove in the hurter in which it can slide freely downstream. Wickets are raised and lowered by use of a boat operating on the upstream side of the dam. Figures II.5.19(c) and II.5.19(d) show improved types of wickets. The Bebout wicket (Figure II.5.19(d)) trips automatically to permit the passage of high flows.

Inflatable dams

An inflatable dam, before activation, is a collapsed nylon/rubber bladder that occupies the full width of the stream and is attached to a concrete sill on the channel bottom. The dam is activated by pumping water into the bladder, thereby inflating it to form a barrier across the channel. The dam is deactivated by releasing water from inside the

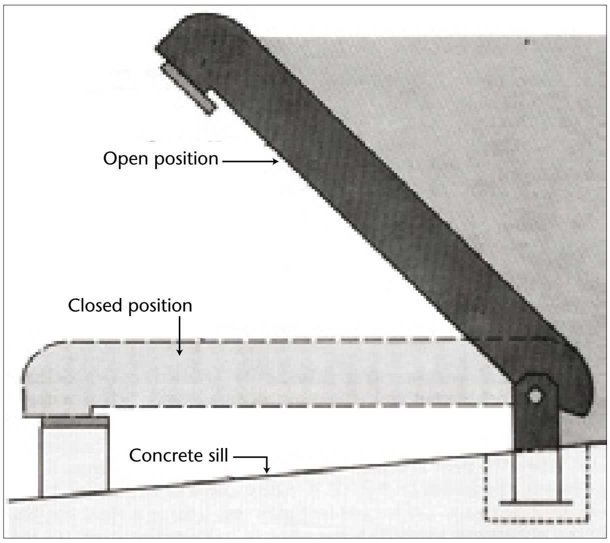
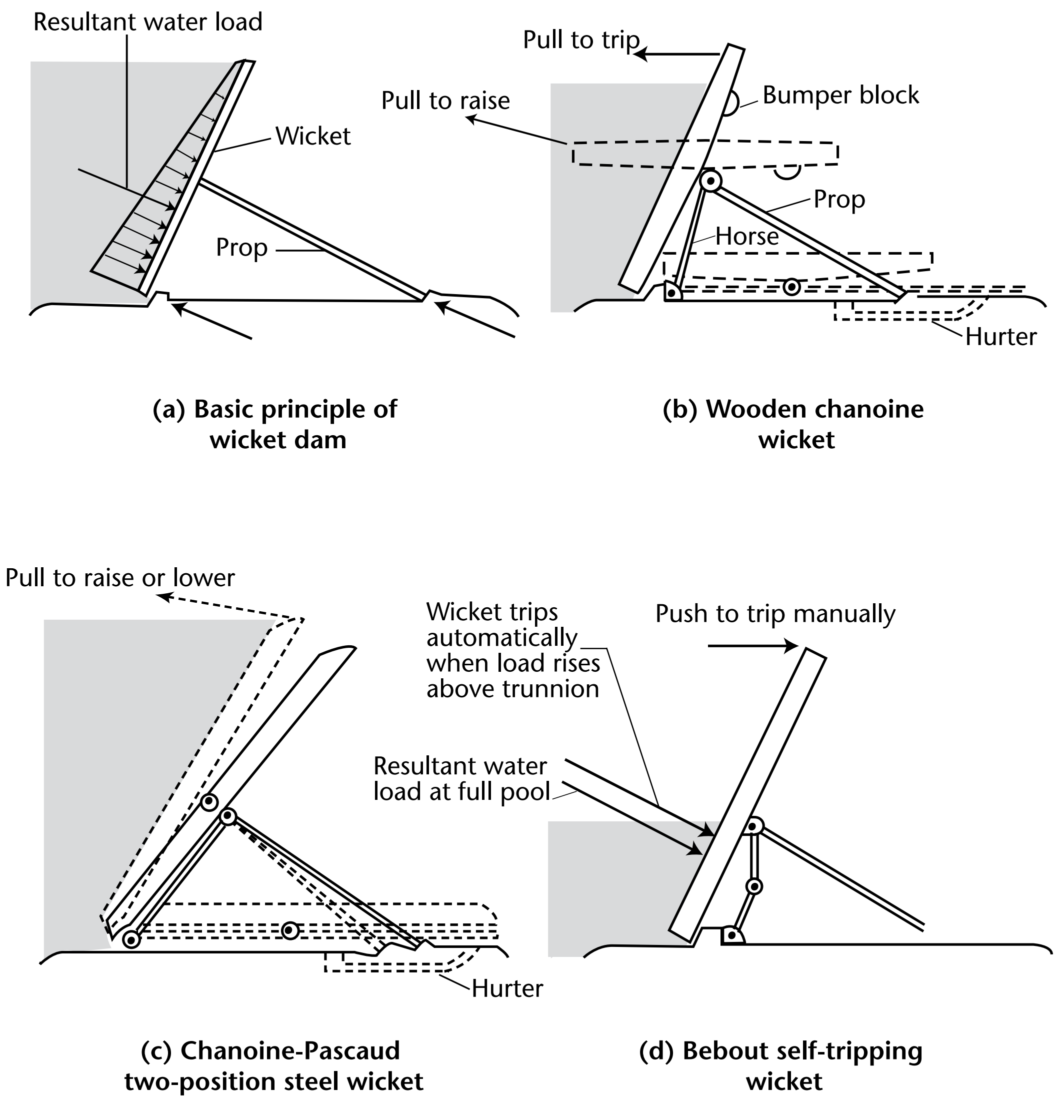


Figure II.5.18. Hinged-leaf gate



(a) Basic principle of wicket dam

(b) Wooden chanoine wicket

(c) Chanoine-Pascaud two-position steel wicket

(d) Bebout self-tripping wicket

Figure II.5.19. Wickets (USACE, 1952)

bladder. Inflatable dams are usually used on shallow streams to maintain a water level in the stream that is sufficiently high to submerge the intake of a diversion works. When the river stage is high the dam is deflated. The inflation and deflation are often automatically controlled in response to the changing stage of the stream. Although it would probably be feasible to determine the rating for an inflatable dam by monitoring both the stream stage and the pressure within the dam bladder, inflatable dams have not been used as gauging-station controls. It is invariably simpler to operate a conventional gauging station on the stream either downstream from the inflatable dam or far enough upstream to be beyond the influence of backwater from the dam.

Discharge characteristics of movable dams

The discharge characteristics of bear-trap gates, hinged-leaf gates and wickets are similar. In their lowered position they act as broad-crested weirs that control the stage-discharge relation over a limited range of low-water stage. The stage at which they become submerged depends primarily on the height of the sill on which they rest. Their discharge ratings in the lowered position will resemble that for a highway embankment (Hulsing, 1967, pp. 26-27) whose general equation is:

$$Q = CbH^{3/2} \tag{5.10}$$

where Q is discharge; C is the coefficient of discharge; b is the width normal to the flow, and H is the total head.

The value of C will be dependent on the elevations of headwater and tailwater, the length of the crest in the direction of flow and the geometry of the crest. For unsubmerged flow (tailwater ≤ 0.7 times headwater) C can be expected to range from about 1.43 to 1.71 depending primarily on the ratio of static head, h, to length of sill in the direction of flow, L. For submerged flow, the free-flow value of C will be multiplied by a factor that ranges from almost zero to almost 1.00, depending on the degree of submergence.

When overtopped in their raised position by headwater, the three types of movable dams, bear-trap gate, hinged-leaf gate and wickets, act as inclined thin-plate rectangular weirs. Figure II.5.20 gives values of the discharge coefficient C in the general weir equation (equation 5.10) for various angles of inclination of such weirs. If the upstream edge of the crest is rounded, the value of C may increase by 5 to 10 per cent.

5.2.7 **Flashboards**

The usual flashboard installation consists of horizontal wooden panels supported by vertical pins placed on the crest of a spillway (Figure II.5.21(a)). Such installations are temporary and are designed to fail when the water surface in the reservoir reaches a predetermined level. A common design uses steel pipes or rods set loosely in sockets in the crest of the dam and designed to bend and release the flashboards at the desired water level. Temporary flashboards of this type have been used in heights up to 1.2 or 1.5 m. Because temporary flashboards are lost each time the supports fail, permanent flashboards are more economical for large installations. Permanent flashboards usually consist of horizontal wooden panels that can be raised or lowered from an overhead cableway or bridge. The bottom edges of the panels are placed in a seat or hinge on the spillway crest, and the panels are supported in the raised position by struts (Figure II.5.21(b)) or by attaching the top edges of the panels to the bridge.

To rate the vertical flashboards shown on Figure II.5.21(a), a value of C = 1.83 is usually used in the general weir equation, equation 5.10.

When the permanent flashboards in Figure II.5.21(b) are lowered the value of C that should be used is that for the free dam crest (no flashboards). The value of C to use when the flashboards are raised and supported by struts is determined from Figure II.5.20, which shows C values for various angles of inclination. If the raised flashboards are supported in an inclined position by a bridge, so

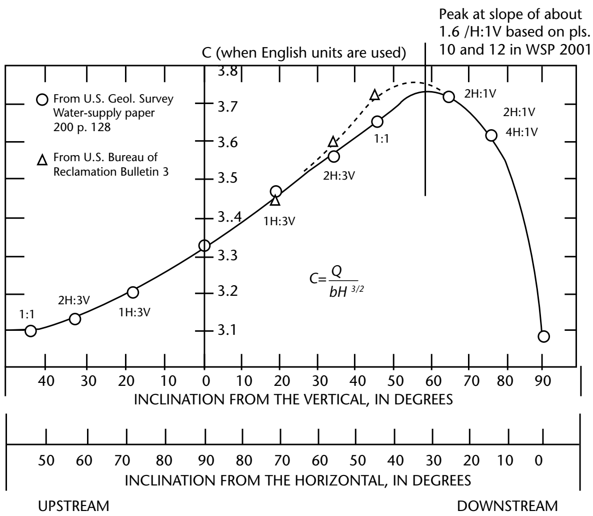


Figure II.5.20. Discharge coefficients for an inclined rectangular thin-plate weir

that the top edge of the flashboards is flush with the upstream edge of the bridge floor, a flat-crested rectangular weir with inclined upstream face is assumed. The bridge floor acts as the flat weir crest and the flashboards act as the inclined upstream face of the weir. Discharge is computed by the use of equation 5.10 and the value of C to be used in that equation can be obtained from Figure II.5.20.

5.2.8 Stop logs and needles

Stop logs consist of horizontal timbers, similar to flashboards, spanning between vertically slotted piers on the dam crest. The timbers may be inserted into or removed from the vertical slots by hand or with a hoist. There is usually considerable leakage between the timbers and considerable time may be required for their removal if they become jammed in the slots. Stop logs are ordinarily used only for small installations where the cost of more elaborate devices is not warranted or in situations where the removal or replacement of the stop logs is expected only at infrequent intervals.

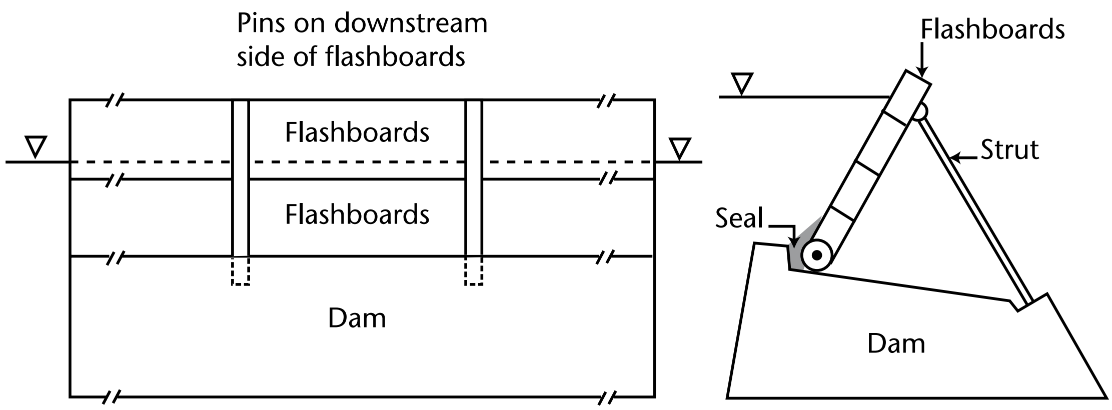
Needles consist of timbers standing on end, with their lower ends resting in a keyway in the spillway and their upper ends supported against the upstream edge of a bridge floor. Needles are easier to remove than stop logs but are difficult to place in flowing water. Consequently they are used mainly for emergency bulkheads that are installed during periods of low flow.

The simple crest shape of stop logs and needles makes it easy to determine the theoretical value of the discharge coefficient C in the general weir equation 5.10, as given by Hulsing (1967) on computing discharge over dams. However, it is usually futile to rate stop logs or needles theoretically because of the appreciable leakage between them.

5.3 NAVIGATION LOCKS

Navigation locks are required for boat traffic to overcome the difference between headwater and tailwater elevations at a dam. The boat enters the open gate of the lock and the lock is closed behind the boat. Valves are used for filling or emptying the locks, as the case may be, to bring the water level in the lock to that of the pool ahead of the boat. The other lock gate is opened and the boat proceeds on its journey. Various lock-filling and lock-emptying systems have been devised as a compromise between two conflicting demands: (a) that the filling time be short so as not to delay traffic and (b) that the disturbances in the lock chamber do not cause stresses in mooring hawsers which might cause the boat or barges to break loose and thereby damage either the boat or lock structure.

The flow through navigation locks is computed as the total volume of water released during a finite time interval, usually one day. The volume of water discharged for any one lockage is the product of the plan or water-surface area of the lock and the difference between headwater and tailwater at the time of lockage. These volumes are summed for the day and divided by 86 400, which is the number of



(a) Temporary type (Front view) (b) Permanent type (End view)

Figure II.5.21. Flashboards

seconds in a day, to obtain the average lockage flow in cubic feet per second or cubic metres per second. Usually it will be sufficiently accurate to compute the daily average lockage discharge, Q_L, by use of the equation:

$$Q_L = \frac{N}{86\,400} A\left(E_h - E_t\right) \tag{5.11}$$

where N is the number of lockages in a day; A is the plan or surface area of the lock; E_h is the daily mean headwater elevation, and E_t is the daily mean tailwater elevation.

If appreciable leakage through the lock occurs between boat lockages, the daily average leakage must be added to the daily average lockage discharge.

Measurement of leakage through navigation locks

If the leakage through the closed lock gates is great it can be measured in the fore-bay with a low-velocity current meter or Acoustic Doppler Current Profiler (ADCP). The leakage will seldom be that great, however, and usually will have to be computed by a volumetric method.

If, for considerable periods of time between lockages, the lockmaster keeps the valves and lower gates closed and the upper gates open, leakage will occur through the lower gates, and it is that leakage, Q_{Lm}, which must be determined. If instead it is the valves and upper gates that are kept closed and the lower gates that are kept open, leakage will occur through the upper gates and it is that leakage, Q_{Um}, which must be determined. If all valves and gates are kept closed it is the equilibrium leakage, Q_{Le}, through the lower gate that must be determined.

Instructions for determining Q_{Lm}, Q_{Um} and Q_{Le} follow. Figure II.5.22 is a definition sketch of a lock.

Definitions:
$h_{Um} = h_{Lm}$ – Maximum head on upper or lower gates for given headwater and tailwater stages;
h_U – Head on upper gate;
h_L – Head on lower gate;
Q_U – Leakage through upper gate produced by h_U;
Q_{Um} – Leakage through upper gate produced by h_{Um};
Q_L – Leakage through lower gate produced by h_L;
Q_{Lm} – Leakage through lower gate produced by h_{Lm};
Q_n – Rate of storage in lock with both gates closed $= Q_L - Q_U$. (When Q_n is negative, the water level rises in lock chamber. When Q_n is positive, the water level falls in lock chamber.);
h_{Ue} – Equilibrium head on upper gate when $Q_U = Q_L$;
h_{Le} – Equilibrium head on lower gate when $Q_U = Q_L$;
Q_{Le} – Leakage through lower gate produced by h_{Le};
$h_U + h_L = h_{Um} = h_{Lm}$;
$h_{Ue} + h_{Le} = h_{Um} = h_{Lm}$.

Field work

(a) Close upper and lower lock gates and open the valve to fill the lock chamber. When the lock chamber is filled, close the valve and open one upper gate slightly;

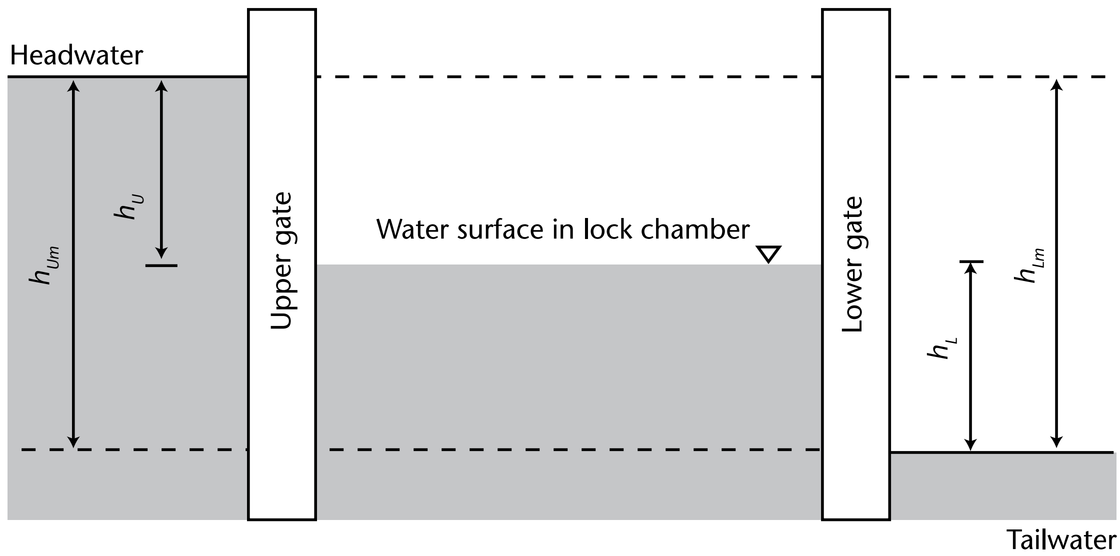


Figure II.5.22. Definition sketch of a lock

(b) Attach the zero end of a steel tape by a small staple to the middle of a long plank. Float the plank in a lock chamber against the lock wall after first setting a reference mark on top of the wall for use as an index for reading the tape. A portable electric-tape gauge is even more satisfactory for reading stages in the lock chamber (see Volume I, Chapter 4);
(c) Record gauge heights in the upper pool and lower pool and the tape reading in the lock chamber;
(d) Close the upper gate. Read the tape immediately after the gate is fully closed and seated, and start a stop watch. Thereafter read the tape and stop watch at intervals of about 0.150 m as stage decreases in the chamber or at minute intervals, whichever comes first. Continue for about ten minutes;
(e) Empty the lock chamber by opening the lower gate and then partly close the lower gate; that is, leave one lower gate slightly open;
(f) Record gauge heights in the upper pool and lower pool and the tape reading in the lock chamber;
(g) Close the lower gate. Read the tape immediately after the gate is fully closed and seated and start a stop-watch. Thereafter read the tape and stop-watch at intervals of about 0.150 m as stage increases in the chamber, or at minute intervals, whichever comes first. Continue for about ten minutes;
(h) Obtain dimensions of the lock chamber for use in computing volumes of water involved in the leakage. That completes the field work.

Computations for Q_{Lm}

(a) Use readings obtained when observations were started with a full lock chamber. Subtract initial tape reading (made with upper gate open slightly) from all tape readings;
(b) Plot adjusted tape readings from step 1 against time in seconds. The first reading made after the upper gate was fully closed is plotted at zero seconds. Too much uncertainty usually exists as to when the gate actually seated to use the closure of the gate as the starting time for the graph (see Figure II.5.23). The plot should be made on a large sheet of graph paper;
(c) Connect the plotted points with a smooth curve. A tangent to the curve at any value of the abscissa represents the rate of change of water-surface elevation at that instant. The rate of change multiplied by the surface area of the lock chamber gives the instantaneous rate of storage in the lock chamber; that is, the difference in rate of leakage out of the chamber through the lower gate and rate of leakage into the chamber through the upper gate. At the instant the upper gate is closed the leakage out of the chamber is at its maximum, Q_{Lm} (full head on the lower gate), and the leakage into the chamber is zero (zero head on the upper gate). As the stage in the chamber falls the leakage out of the chamber decreases because of the decreased head on the lower gate and leakage into the chamber increases because of the increased head on the upper gate. Eventually the leakage into the lock would equal the leakage out of the lock (Q_{Le}) and the stage in the chamber would remain constant;
(d) To obtain the rate of storage at any instant from the tangent of the curve showing the decrease in lock stage with time, construct a diagram showing the storage rate (Q_n) for various tangential slopes. The method of constructing the diagram is demonstrated in Figure II.5.23. The area of the lock chamber is 1183.241 m^2. If the stage in the chamber dropped 0.6 m, the change in volume would be 0.6 x 1183.24 or 712 m^3. If Q_n were 5.66 m^3, the time required for a 0.6 m drop would be 127.5 seconds. A vertical line is drawn at 127.5 seconds on Figure II.5.23 and a diagonal line having a drop of 0.6 m is drawn between the abscissa values of 0 and 127.5 seconds. A tangent to the storage curve having a similar slope would have a Q_n value of 5.66 m^3. Diagonals representing other values of Q_n are added as shown;
(e) Select two points on the storage curve, one near the origin (0 seconds) and the other no more than 0.3 m lower in stage. Draw tangents to those points and use the slopes of those tangents with the tangential rate diagram to obtain the two values of Q_n. To obtain the tangential slope at a point on the curve, use a pair of dividers to lay off short equal distances on the curve on each side of the selected point. A chord connecting the equidistance points will have a slope approximately equal to that of the tangent;
(f) The two values of Q_n obtained in the preceding step will be used to compute Q_{Lm}. No further use will be made of the leakage curve, except that it has value for making a rough check on the basic assumption that will be made in the computations that follow. That assumption is that the leakage through a gate can be treated as though it all occurred at an orifice at the bottom of the gate. In other words:

$$\frac{Q_L}{Q_{Lm}} = \left(\frac{h_L}{h_{Lm}}\right)^{1/2} \quad \text{and} \quad \frac{Q_U}{Q_{Um}} = \left(\frac{h_U}{h_{Um}}\right)^{1/2} \qquad (5.12)$$

or

$$Q_L = Q_{Lm}\left(\frac{h_L}{h_{Lm}}\right)^{1/2} \text{ and } Q_U = Q_{Um}\left(\frac{h_U}{h_{Um}}\right)^{1/2}$$

(g) From Figure II.5.22 and equation 5.12:

$$Q_n = Q_L - Q_U$$

or (5.13)

$$Q_n = Q_{Lm}\left(\frac{h_L}{h_{Lm}}\right)^{1/2} - Q_{Um}\left(\frac{h_U}{h_{Um}}\right)^{1/2}$$

For each of the two values of Q_n, all values in equation 5.13 are known except for the values of Q_{Lm} and Q_{Um}. The known values can be substituted in equation 5.13 to give two simultaneous equations which can then be solved for the desired value of Q_{Lm};

(h) In the preceding step it would be a simple matter to solve for Q_{Um} but we do not do so. Our basic assumption of orifice flow may not be strictly correct and experience has shown that the desired value of Q_{Um} can be computed with much more accuracy by using the field data obtained when observations of leakage were started with an empty lock chamber;

(i) To obtain values of leakage through the lower gate, when the upper gate is open, for other values of total head, use the following equation:

$$Q'_{Lm} = Q_{Lm}\left(\frac{h'_{Lm}}{h_{Lm}}\right)^{1/2} \tag{5.14}$$

where Q_{Lm} and h_{Lm} are values obtained from a leakage test as described above, and Q'_{Lm} is the leakage through the lower gate corresponding to any other value of total head h'_{Lm};

(j) Prepare a rating table of Q_{Lm} versus h_{Lm}.

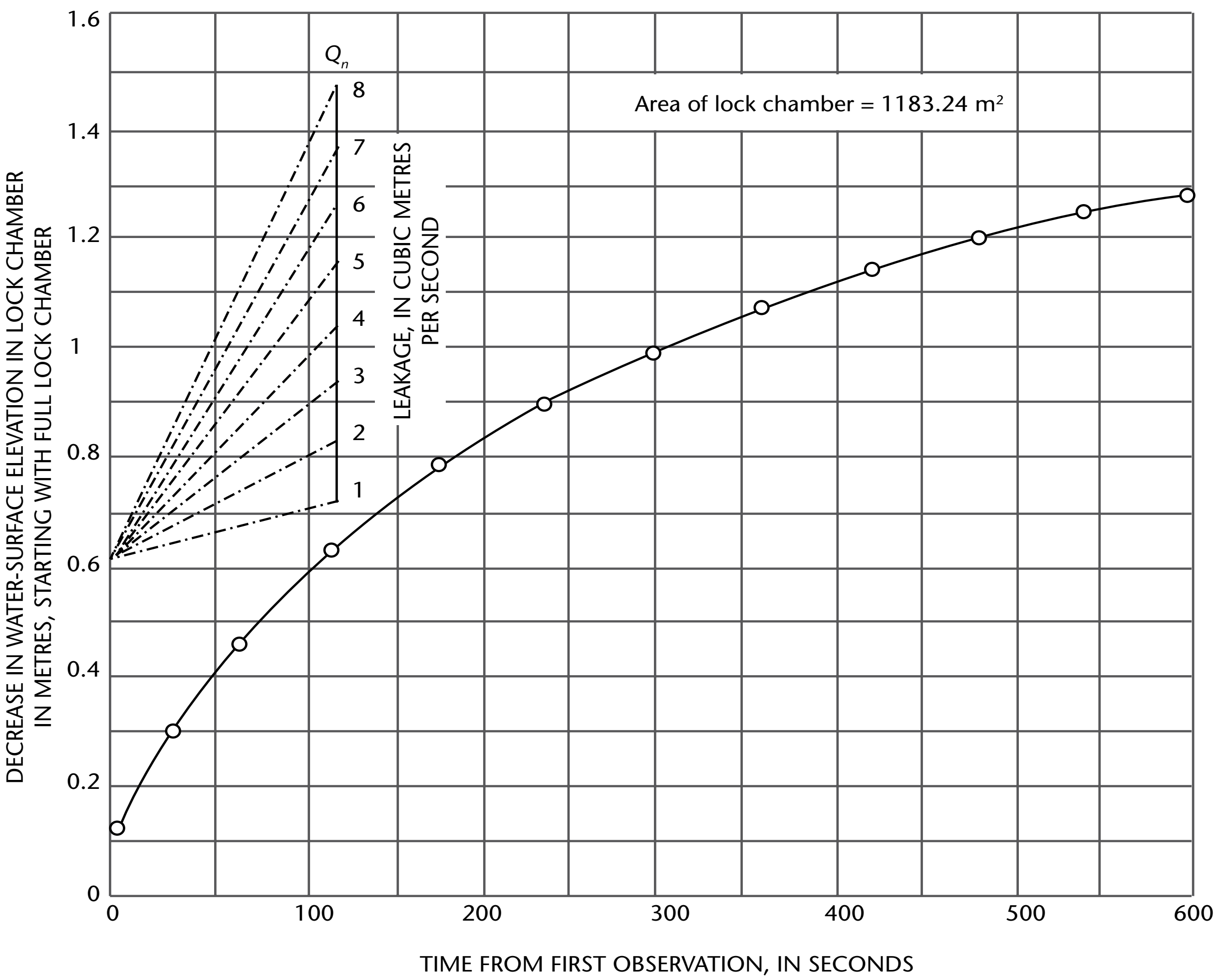


Figure II.5.23. Leakage diagram starting with lock chamber full

Computations for Q_{Um}

(a) Use readings obtained when observations were started with an empty lock chamber. Subtract initial tape reading (made with lower gate slightly open) from all tape readings;
(b) Plot adjusted tape readings from step 1 against time in seconds;
(c) Proceed with computations in a manner analogous to that used in the computation of Q_{Lm};
(d) Obtain Q_n for two points on the leakage curve, one near the origin (0 seconds) and the other no more than 0.3 m higher in stage;
(e) Use equation 5.13 to solve for the desired value of Q_{Um};
(f) To obtain values of leakage through the upper gate, when the lower gate is open, for other values of total head, use the following equation:

$$Q'_{Um} = Q_{Um}\left(\frac{h'_{Um}}{h_{Um}}\right)^{1/2} \qquad (5.15)$$

where Q_{Um} and h_{Um} are values obtained from a leakage test as described above, and Q'_{Um} is the leakage through the upper gate corresponding to any other value of total head h'_{Um};
(g) Prepare a rating table of Q_{Um} versus h_{Um}.

Computations for Q_{Le}

(a) Q_{Le} is the leakage through the lower gate when equilibrium exists; that is, the stage in the lock chamber is constant because $Q_U = Q_L$;
(b) Starting with the equation, $Q_{Ue} = Q_{Le}$, it is a simple matter to transform the equation to:

$$h_{Le} = h_{Lm} \Big/ \left[\left(Q^2_{Lm} \big/ Q^2_{Um}\right) + 1\right] \qquad (5.16)$$

All values on the right-hand side of equation 5.16 are known because in preceding steps Q_{Lm} and Q_{Um} had been computed. Solve for h_{Le};
(c) Obtain the desired value of Q_{Le} from the equation:

$$Q_{Le} = Q_{Lm}\left(\frac{h_{Le}}{h_{Lm}}\right)^{1/2} \qquad (5.17)$$

(d) Use the rating tables for Q_{Lm} and Q_{Um} with equations 5.16 and 5.17, to prepare a rating table of Q_{Le} versus h_{Lm}.

5.4 PRESSURE CONDUITS

5.4.1 General

In one respect the gauging of a pressure conduit is simple in that the cross-sectional area is constant for all discharges. The calibration of the metering device offers difficulty, however, because the discharge measurements require special instrumentation unless they can be made by current meter in the forebay or afterbay of the conduit where open-channel conditions exist.

The following are the metering devices used for pressure conduits:
(a) Mechanical meters:
 (i) Displacement meter
 (ii) Inferential meter;
 (iii) Variable-area meter;
(b) Differential-head meters:
 (i) Constriction meters:
  a. Venturi meter;
  b. Flow nozzle;
  c. Orifice meter;
 (ii) Bend meter;
 (iii) Pressure differential in a reach of unaltered conduit;
(c) Electromagnetic velocity meter;
(d) Ultrasonic (acoustic) velocity meter;
(e) Acoustic Doppler Velocity Meter;
(f) Laser flow-meter.

Changes in the rating of mechanical meters occur only as a result of wear on the moving parts of the meter. Changes in the rating of differential-head meters that are kept clean occur only as a result of changes in perimeter roughness of the conduit with time. The electromagnetic, acoustic and laser velocity meters are complex electronic devices and as such they are subject to the occasional calibration drift that for various reasons affect such devices.

The various meters must be calibrated when first installed and the calibration must be periodically checked thereafter. Methods of measuring discharge for that purpose include:
(a) Pitot-static tubes and pitometers;
(b) Salt-velocity method;
(c) Gibson method.

This section of the Manual closes with a brief discussion of discharge ratings for turbines, pumps, gates and valves, all of which are associated with pressure conduits. In addition, a brief discussion is included that describes a computer program for computing flow through water control structures.

5.4.2 Metering devices for pressure-conduit flow

Mechanical meters

Mechanical meters are widely used in water-distribution systems because of their low cost and small size, but they can only be used to measure a relatively narrow range of discharge. They are not suited for the measurement of very low flow rates because the liquid may pass the meter without moving the mechanical elements. They are seldom used to measure discharges greater than 0.28 $m^3 s^{-1}$ because of high head loss. A large variety of mechanical meters are commercially available, but only the three general types – displacement, inferential and variable-area – will be described here (Howe, 1950, pp. 210-212).

Displacement meters

An elementary form of all displacement meters consist of a single or multiple piston arrangement in which fluid passing through the meter moves a piston back and forth. The movement of the piston is readily registered upon a counting device calibrated in any desired units to give total volume of flow. Such meters can have a fairly large capacity and are accurate if no slippage occurs.

Another commonly used displacement meter is the disk meter which oscillates in a measuring chamber. For each oscillation a known volume of water passes the meter. The motion of the disk operates a gear train which in turn activates a counting mechanism, thereby furnishing a measure of the total volume of flow. When the disk is new the meter is accurate to within 1 per cent, but the meter may under-register significantly as the disk becomes worn.

Inferential meters

Inferential meters are in effect small turbines and are called inferential because the rate of flow is inferred from the speed of rotation of the propeller. An essential element of such meters is a set of guide vanes which may be adjusted to change the calibration of the meter. However, the calibration may inadvertently change if the surface of the propeller blades becomes worn or coated. Although inferential meters normally register only volume of flow, equipment may be added to the meter to indicate instantaneous rate of discharge.

Variable-area meter

The variable-area meter consists of a vertical tapered tube containing a small plunger or float. In some instruments the plunger is completely immersed in a transparent, graduated tube. In others, a stem projects through the end of the conical tube and traverses a scale. In both types the plunger rises as the rate of flow increases, thereby increasing the area around it. By calibration, the position of the plunger can be related to the rate of flow. These instruments are restricted to the measurement of rather small discharges and will not accommodate any great change in viscosity without recalibration. Accuracy within 1 per cent is possible.

Differential-head meters

The flow of fluid through a constriction in a pressure conduit results in a lowering of pressure at the constriction. The drop in piezometric head between the undisturbed flow and the constriction is a function of the flow rate. The venturi meter, flow nozzle and orifice meter (Figure II.5.24) are constriction meters that make use of this principle. The difference in piezometric head may be measured with a differential manometer or pressure gauges. In order that such an installation may function properly, a straight length of pipe at least 10 diameters long should precede the meter. Straightening vanes may also be installed in the conduit just upstream from the meter to suppress disturbances in the flow.

Venturi meters

Venturi meters (Figure II.5.24(a)) are highly accurate and efficient flow meters. They have no moving parts, require little maintenance and cause little head loss (United States Bureau of Reclamation, 1971). They operate on the principle that flow in a given closed-conduit system moves more rapidly through areas of small cross-section (D2 in Figure II.5.24(a)) than through areas of large cross-section (D1 in Figure II.5.24(a)). The total energy in the flow, consisting primarily of velocity head and pressure head, is essentially the same at D1 and D2 within the meter. Thus the pressure must decrease in the constricted throat, D2, where the velocity is higher; and conversely the pressure must be greater at D1, upstream from the throat, where the velocity is lower. This reduction in pressure from the meter entrance to the meter throat is directly related to the rate of flow passing through the meter, and is the measurement used to determine flow rate. Tables or diagrams of the head differential versus rate of flow may be prepared, and flow indicators or flow recorders may be used to display the differential or the rate of flow.

The relation of rate of flow, or discharge, to the head and dimensions of the meter is:

$$Q = \frac{CA_2\sqrt{2gh}}{\sqrt{1-r^4}} \qquad (5.18)$$

where A_2 = cross-sectional area of the throat, in m^2; h = difference in pressure head between upstream pressure-measurement section and the downstream pressure-measurement section, in metres; g = 9.81 metres per second per second; r = ratio of the throat diameter to pipe diameter = D_2/D_1 and C = coefficient of discharge for the venturi meter.

The coefficient of discharge for the venturi meter will range from an approximate value of 0.935 for small throat velocities and diameters, to 0.988 for relatively large throat velocities and diameters. (See Figure II.5.25.)

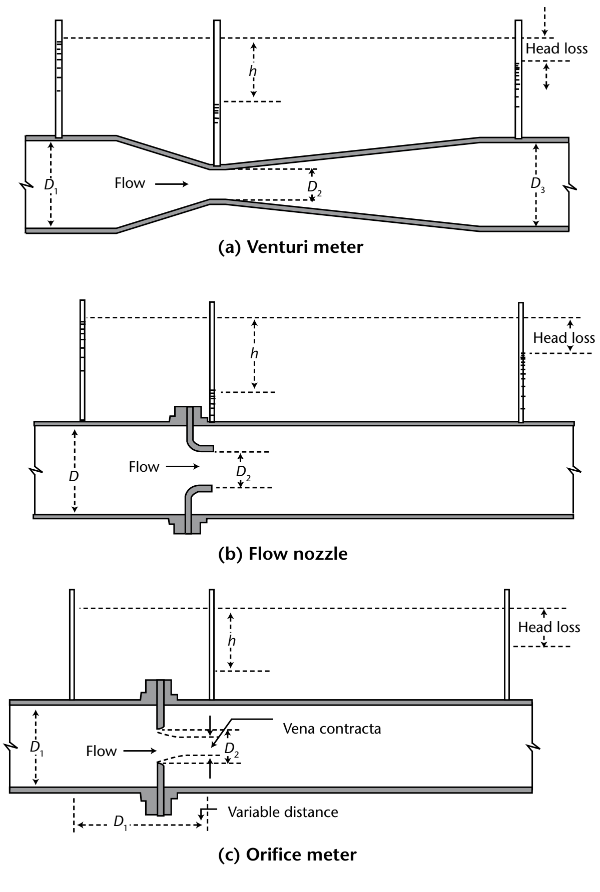


Figure II.5.24. Three types of constriction meters for pipe flow (Courtesy of United States Bureau of Reclamation)

Flow nozzles

Flow nozzles operate on the same basic principle as venturi meters. In effect, the flow nozzle is a venturi meter that has been simplified and shortened by omitting the long diffuser on the outlet side (Figure II.5.24(b)). The streamlined entrance of the nozzle provides a straight cylindrical jet without contraction so that the coefficient is almost the same as that for the venturi meter. In the flow nozzle the jet is allowed to expand of its own accord and the high degree of turbulence created downstream from the nozzle causes a greater loss of head than occurs in the venturi meter where the long diffuser suppresses turbulence.

The relation of rate of flow to the head and to the dimensions of the flow nozzle is defined by equation 5.18, the same equation as used for venturi meters. The symbols have the same meaning as for the venturi meter, except that C in equation 5.18 is the coefficient of discharge for the flow nozzle.

The coefficient of discharge for the flow nozzle will range from 1.0 to 0.97 or less. Inasmuch as the flow conditions at the entrance to the throat are similar to those of the venturi meter, the coefficients should be nearly the same with the same diameter ratio, r. The upstream pressure connection is frequently made through a hole in the wall of the conduit at a distance of about 1 pipe diameter upstream from the starting point of the flare of the nozzle. The pressure observed is that of the stream before it has begun to turn inward in response to the inlet curvature of the nozzle. The downstream pressure connection may be made through the pipe wall opposite the end of the nozzle throat.

Orifice meters

A thin-plate orifice inserted across a pipeline can be used for measuring flow in much the same manner as a flow nozzle (Figure II.5.24(c)). The upstream pressure connection is often located at a distance of about 1 pipe diameter upstream from the orifice plate. The pressure of the jet ranges from a minimum at the vena contracta – the smallest cross-section of the jet – to a maximum at about four or five conduit diameters downstream from the orifice plate. The downstream pressure connection, the centre connection shown in Figure II.5.24(c), is usually made at the vena contracta to obtain a large pressure differential across the orifice. The location of the vena contracta may be determined from data provided in standard hydraulic handbooks.

The relation of rate of flow to the head and dimensions of the metering section is defined by equation 5.18, the same equation as used for venturi and orifice meters. The symbols have the same meaning for all three meters, except that C in equation 5.18 is the coefficient of discharge for the flow nozzle.

For pressure taps located one pipe diameter upstream from the orifice plate and at the vena contracta, the coefficient of discharge ranges from 0.599 for an r value of 0.20, to 0.620 for an r of 0.71 . The principal disadvantage of orifice meters, as compared to venturi meters or flow nozzles, is their greater loss of head. On the other hand they are inexpensive and are capable of producing accurate flow measurements.

Bend meters

Another type of differential head meter is the bend meter, which utilizes the pressure difference between the inside and outside of a pipe bend. The meter is simple and inexpensive. An elbow already in the line may be used without causing added head loss. For best results a bend meter should be calibrated in place. The meter equation is:

$$Q = C_d A \sqrt{2gh} \tag{5.19}$$

where C_d is the coefficient of discharge; h is the difference in piezometric head between the outside and inside of the bend at the midsection, and A is the cross-sectional area of the pipe.

For best results it is recommended that the lengths of straight pipe upstream and downstream from the bend be equal to at least 25 pipe diameters and 10 pipe diameters, respectively. Lansford (1936) experimented with 90° bends and concluded that if calibration of a 90° bend is not feasible, results at moderate to high Reynolds numbers that are accurate to within 10 per cent can be obtained from a simple formula for Q, in which:

$$C_d = \sqrt{r/2D} \tag{5.20}$$

where D is the pipe diameter and r is the centreline radius of the bend.

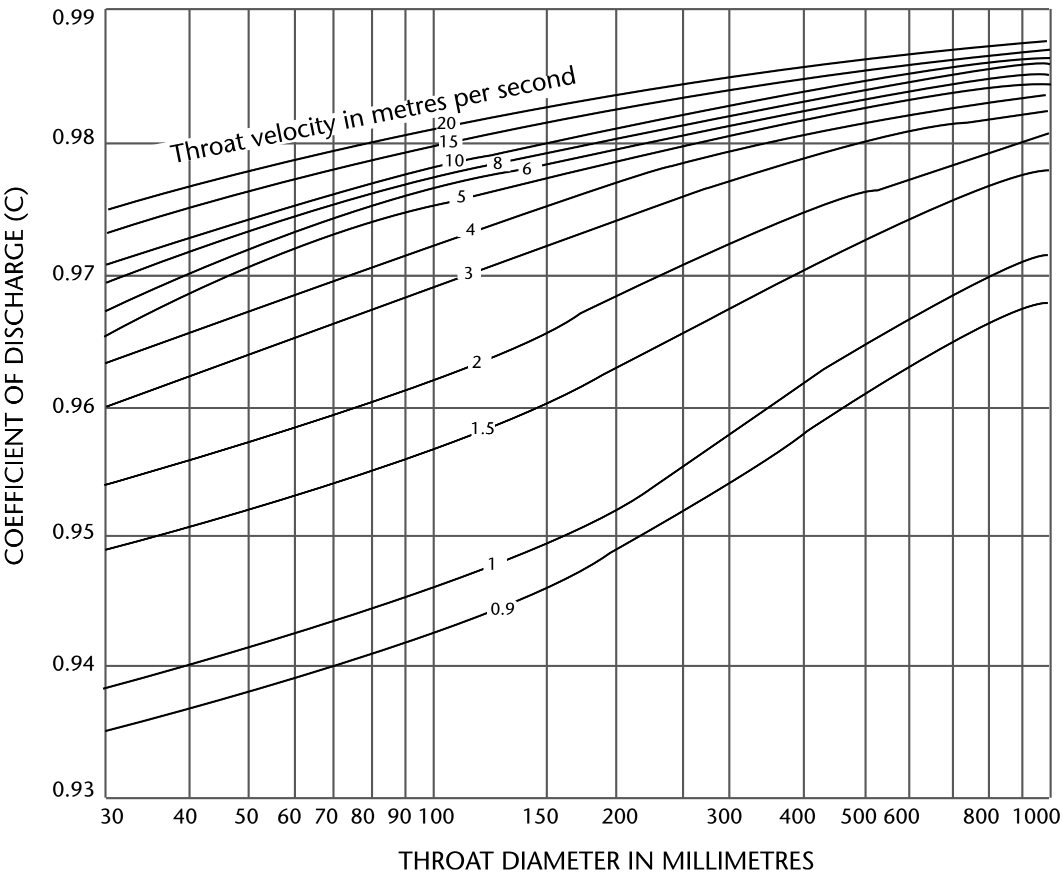


Figure II.5.25. Discharge coefficient for Venturi meter, based on throat diameter and throat velocity

Pressure differential in a reach of unaltered conduit

If a pressure-conduit system has high velocities and low pressures it may not be practical to install a venturi meter in the line because cavitation will occur in the throat along with excessive vibration. In that situation the installation of a manometer between two piezometer taps in the conduit, several hundred feet apart, may be the most feasible method of metering the flow. One, but preferably two discharge measurements would suffice to rate the manometer and a third measurement could be made to check the rating equation which is:

$$Q = K\sqrt{\Delta h} \qquad (5.21)$$

where Q = discharge; K = a constant, and Δh = head differential.

If two discharge measurements are used in the initial calibration, the two computed K values, which should agree closely, are averaged.

In the case of reaction turbines the discharge may be metered by a manometer that measures the pressure drop in the scroll case. The scroll case of a reaction turbine has a decreasing diameter, being largest at its upstream end where it is joined to the penstock. A set of piezometer taps is installed at each end of the scroll case forming, in effect, a type of venturi section. Discharge is computed by use of equation 5.21 where K is determined from discharge measurements, preferably made over the complete range of output, and simultaneous observations of the pressure drop. The calibration should remain constant as long as the turbine efficiency does not change.

Summary of differential-head meters

Differential-head meters are very satisfactory metering devices as long as they are kept clean and the velocities in the conduit are high enough to give significant pressure differentials between the two piezometer taps.

Electromagnetic Velocity Meter

Electromagnetic velocity meters for measuring flow in pressure conduits are commercially available. The principle of the electromagnetic velocity meter was explained in Volume I, Chapter 5, but to repeat briefly, when a fluid which is an electric conductor moves across a magnetic field at 90° an electromotive force is produced in the fluid at right angles to both the flux of the magnetic field and the velocity of the fluid. The induced voltage is proportional to the average velocity of the fluid, V. If the pipe is a conductor, as it usually will be, an insulating liner must be installed in the metering section and the probes must contact the water. Two or more discharge measurements are required to calibrate the meter.

Ultrasonic (acoustic) Velocity Meter

Acoustic velocity meters for measuring flow in pressure conduits are commercially available. The principle of the ultrasonic (acoustic) method was explained in Volume I, Chapter 6, and will not be discussed further, other than to state that better results are apparently obtained with the transducers of the meter in direct contact with the fluid stream than are obtained with the transducers mounted on the outside of the conduit walls (Schuster, 1975). The acoustic velocity meter is self calibrated but may be check calibrated by discharge measurements if necessary.

Acoustic Doppler Velocity Meters

Commercially available Acoustic Doppler Velocity Meters (ADVM) could be used for measuring flow in pressure conduits. Classes of instruments for this application include the Acoustic Doppler Velocimeter (ADV) described in Volume I, Section 5.3.8 and the ADVM described in Volume I, Section 6.3. Both instruments use the same basic theory of operation, in that they compute water velocities using the Doppler shift of sound transmitted underwater and reflected off moving particulates suspended in the water. The theory of operation is described in more detail in the relevant sections of Volume I and is not discussed further here. The distinguishing feature of the ADV versus the ADVM is that the ADV measures velocities in a small volume thus may be considered a point meter while an ADVM measures velocities in one or more larger sample volumes. Instruments with multiple sample volumes provide current profiles in the vertical or horizontal depending on how the instrument is physically mounted. In theory either instrument could be used in pressure conduits to compute discharge. Calibration by discharge measurements would likely be needed. The main consideration for using acoustic Doppler instruments is to avoid contamination of the acoustic signal by physical boundaries, namely, the walls of the conduit. Another consideration is to use a physically small instrument to minimize flow disturbance in the measured volume.

Laser flow-meter

Laser (light amplification by stimulated emission radiation) beams have been used for studying the turbulent characteristics of flowing liquids and for determining the velocity of fluid flow (Schuster, 1970). The Doppler principle, which involves a measurable shift in the frequency of the light rays under the influence of an external velocity imposed on the system, underlies the operation of the laser flow-meter. The flowing water scatters part of a beam of light (laser) directed through it. By comparing the frequencies of the scattered and unscattered rays, collected in receiving lenses on the opposite side of the stream, the velocity of the water (hence the discharge) can be calculated. In laboratory experiments the instrument has measured fluid flows as slow as a few millimetres per second and as fast as 300 m or more per second. The device is commercially available for measuring discharge in both open channels and pressure conduits.

5.4.3 Discharge-measurement methods for meter calibration

Measurement of discharge by pitot-static tubes and pitometers

Pitot-static tubes and pitometers may be classed as differential-head meters, but they are seldom used for continuous-flow measurement. Instead they are usually used for calibrating other metering devices in place and for intermittent measurements. Pitot tubes and pitometers indicate the velocity head at a point in the conduit cross-section.

The operation of pitot-static tubes or pitometers is based on the principle that the increase in head at the mouth of a bent tube facing upstream is a measure of the velocity head of the oncoming flow. The most commonly used type of pitot-static tube (Figure II.5.26(a)) consists of two separate, essentially parallel tubes, one for indicating total head, P_t (sum of static and velocity heads), and the other for indicating only static (pressure) head, P_s. Manometers are commonly used to measure these heads, the velocity head being the difference between the static head and the total head. A pressure transducer may also be used instead of the manometer for measuring the differential head. Where pitot-static tubes are used for continuous-flow measurement, oscillograph or digital recording of the electrical signal from the transducer provides a continuous record of the changes in head.

The general equation for pitot-static tubes and pitometers is:

$$V = C_1\sqrt{2g\Delta h} \tag{5.22}$$

where V = velocity; C_1 = coefficient, g = acceleration of gravity and Δh = observed velocity head differential.

The coefficient C_1 will vary with the dimensions and geometry of the meter, but the instruments are usually individually rated by the manufacturer in the manner that current meters are rated, and the value of C_1 is therefore known. For the pitot-static tube shown in Figure II.5.26(a) the value of C_1 usually ranges from 0.98 to 1.00.

Another commonly used type of pitot device is the Cole pitometer (Figure II.5.26(b), which consists of two tubes headed in opposite directions. The tubes can be rotated so that the instrument may be inserted through a small bushing in a pipe. When

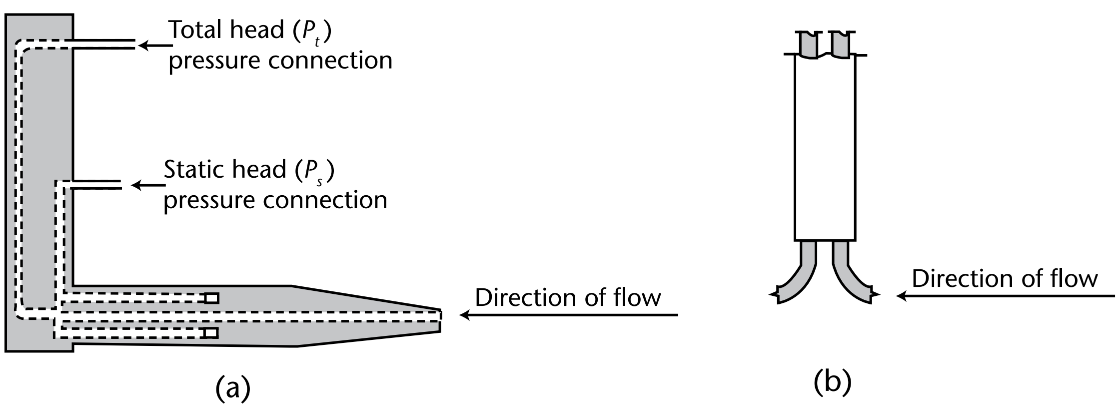


Figure II.5.26. Schematic drawing of (a) pitot-static tube and (b) Cole pitometer

in operating position the downstream tube registers a negative pressure because its opening is in the wake of the instrument. The differential of the water columns is therefore considerably greater than $V^2/2g$. The value of C_1 in equation 5.22 usually ranges from 0.84 to 0.87.

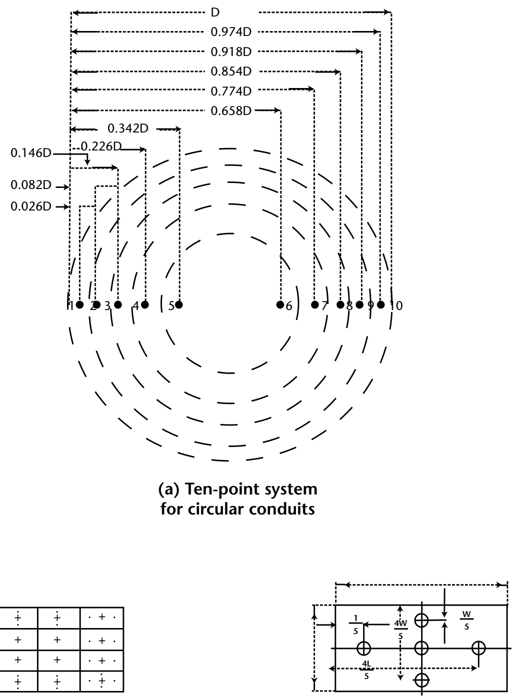
Reinforced pitot tubes and pitometers have been used successfully in pipes up to five feet in diameter having flow velocities of 1.5 to 6 m s^{-1} (United States Bureau of Reclamation, 1971). Even larger pipes can be traversed by a pitometer by having access ports on both sides of the pipe and by probing to or past the conduit centreline from each side. The principal disadvantage encountered is that relatively large forces push on the tube when flow velocities are high, making positioning and securing of the instrument difficult. Dynamic instability may also occur, causing the tube to vibrate and produce erroneous readings. At moderate flow velocities the measurements are accurate.

The most common pressure conduit is the circular pipe. For a constant rate of flow the velocity varies from point to point across the stream, gradually increasing from the walls to the centre of the pipe. The mean velocity is obtained by dividing the cross-sectional area of the pipe into a number of concentric equal area rings and a central circle. The standard 10-point system is shown in Figure II.5.27(a). More divisions may be used if large flow distortions or other unusual flow conditions exist. Observations are made at specific locations in these sub-areas (Figure II.5.27(a)) and mean velocity is computed from the equation:

$$V_{mean} = C_1\left(\sqrt{2g}\right)\left(\sqrt{\Delta h}\right)_{average} \qquad (5.23)$$

The mean velocity in rectangular ducts can be determined by first dividing the cross-section into an even number, at least 16, of equal rectangles geometrically similar to the duct cross-section and then making a pitot-tube observation at the centre of each sub-area (Figure II.5.27(b)). Additional readings should be taken in the areas along the periphery of the cross-section in accordance with the diagram in Figure II.5.27(c). Mean velocity is then computed from equation 5.23.

When using pitot-static tubes or pitometers it must be remembered that at low velocities, head differentials are small and errors in reading head differentials will seriously affect the results. Also the openings in the tubes are small and foreign material in the water, such as sediment or trash, can plug the tubes.



(a) Ten-point system for circular conduits

(b) System for rectangular conduits, where at least 16 divisions must be used

(c) Additional points for data in areas around periphery of the rectangular conduit

Figure II.5.27. Locations for pitot-tube measurements in circular and rectangular conduits. (Reproduced from B.S. 1042, Flow measurement (1943), by permission of the British Standards Institution)

Measurement of discharge by salt-velocity method

Discharges in conduits flowing full may be determined from the known dimensions of the conduit and velocity observations made by the salt-velocity method. Basically, the method uses the increased conductivity of salt water as a means of timing the travel of a salt solution through a length of conduit. A concentrated solution of sodium chloride is suddenly injected into the conduit at an injection station. At two downstream stations electrodes are connected to a recording ammeter. An increase in the recorded electric current occurs when the prism of water containing the salt passes the electrodes (Figure II.5.28). The difference in time, t, between the centres of gravity of the recorded salt passage is obtained from the recorder chart as shown in Figure II.5.28. The discharge is equal to the volume of the conduit between the two electrodes divided by time, t, in seconds. It is not necessary that the conduit be uniform.

The brine-injection system that is used is quite complex (Figures II.5.29 and II.5.30). A turbulence-creating device (turbulator) is also sometimes used

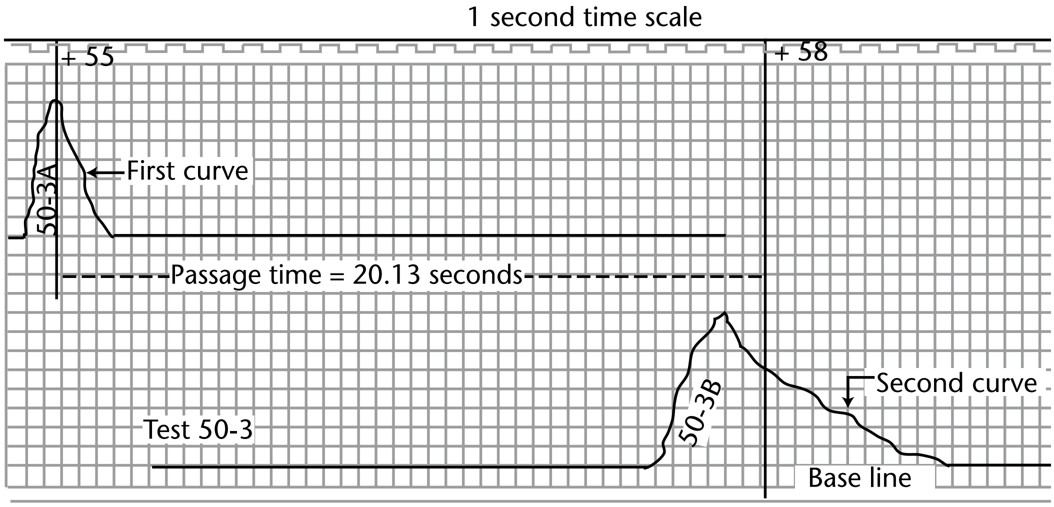


Figure II.5.28. Sample record of a salt cloud passing upstream and downstream electrodes in the salt-velocity method of measuring flows in pipelines. (Courtesy of United States Bureau of Reclamation)

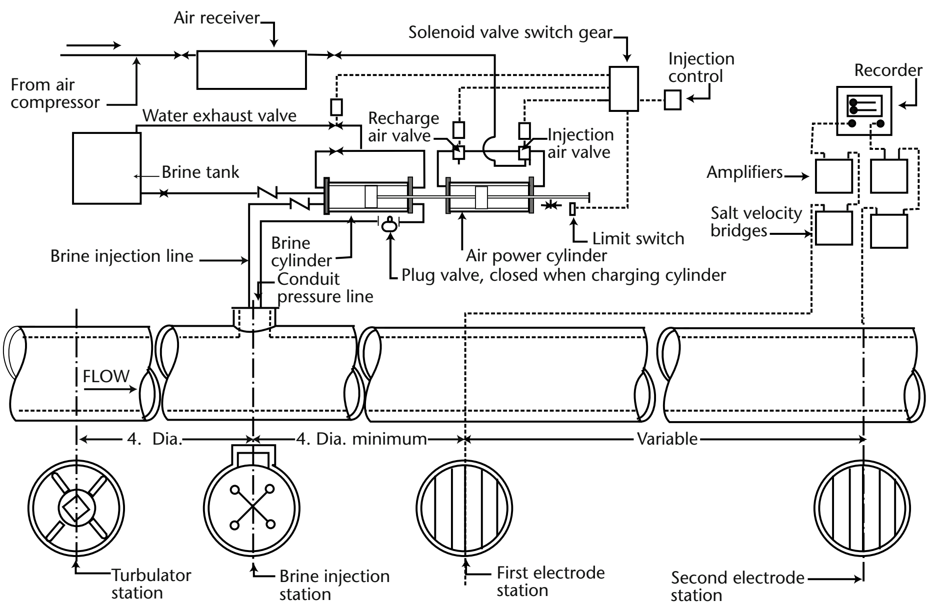


Figure II.5.29. General arrangement of salt-velocity equipment for pressure conduits (Courtesy of United States Bureau of Reclamation)

to insure adequate mixing of the brine and water by the time the upstream electrode station is reached. The required equipment and techniques have been described in detail by Thomas and Dexter (1955).

Measurement of discharge by the Gibson method

The Gibson method was developed for computing the discharge of a conduit or penstock controlled by a valve, turbine or regulating device located at the downstream end. The pressure conduit must extend at least 7.5 m and preferably much more upstream from the valve or regulating device, but the conduit need not be of uniform cross-sectional area. The underlying principle of the method is that the pressure rise that results from gradually shutting off the flow in a conduit is an indication of the original velocity of the water (Howe, 1950, pp. 209-210).

The Gibson apparatus (Figure II.5.31(a)) consists of:

(a) A mercury U-tube connected to the penstock just upstream from a gate;

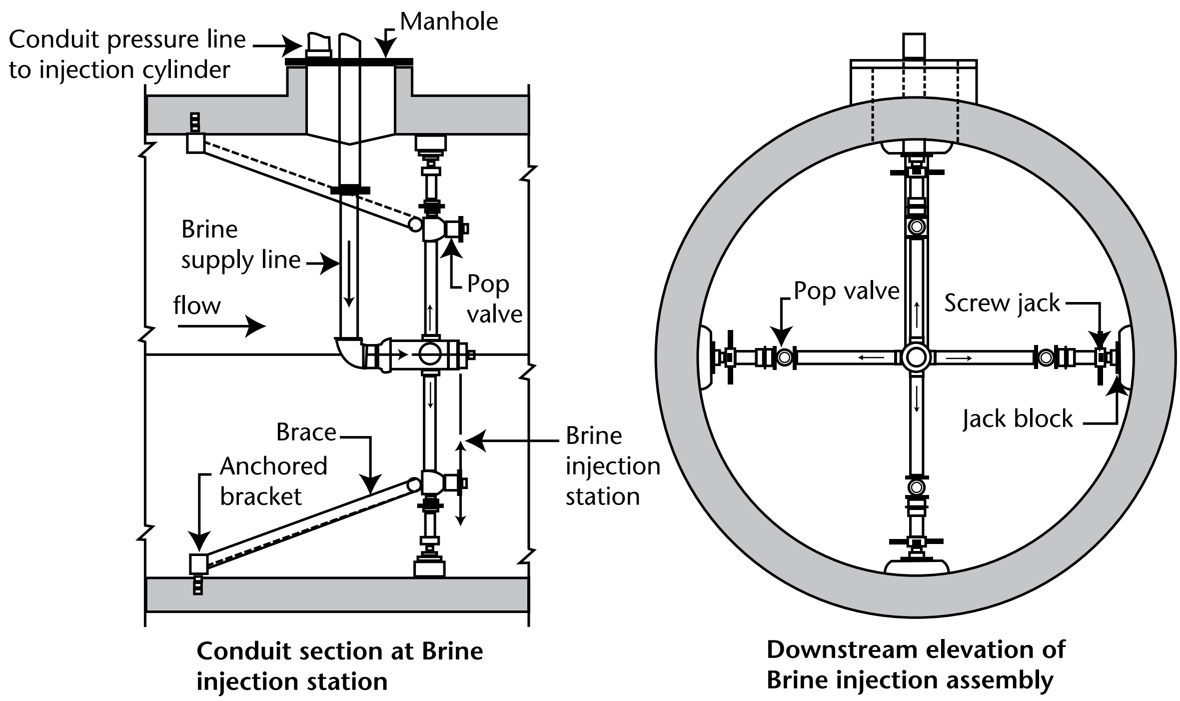


Figure II.5.30. Brine injection equipment in conduit (Courtesy of United States Bureau of Reclamation)

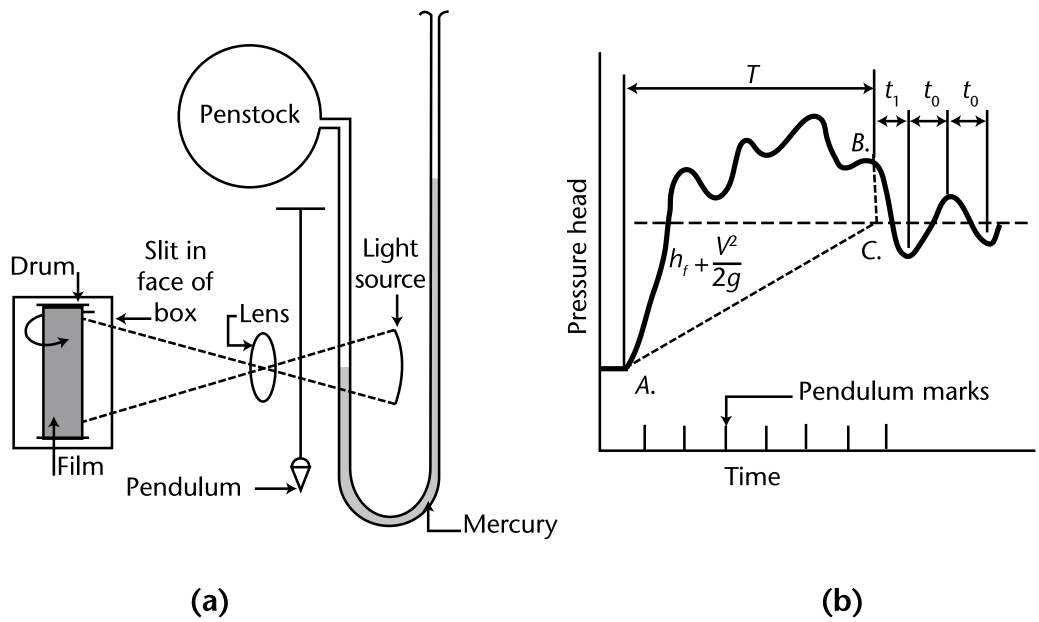


Figure II.5.31. Gibson apparatus and pressure-variation chart (after Howe, 1950). Reprinted by permission of John Wiley and Sons Ltd.

(b) A light source behind the U-tube;
(c) A pendulum that swings in front of the box;
(d) A narrow slit in the box directly behind the U-tube.

Light shines through the U-tube and exposes a film on a rotating drum unless blocked by the pendulum or the mercury in the tube. During a test, the film therefore registers the fluctuation of the mercury column and the time intervals indicated by the pendulum (Figure II.5.31(b)). The period of deceleration, T, terminates when the oscillations become symmetrical (point B, Figure II.5.31(b), where $t_1 = t_0$). An integration of the area $ABCA$ leads directly to the discharge through application of the equation:

$$Q = \left(\frac{\pi D^2}{4}\right)\left(\frac{g}{L}\right)(areaABCA) \tag{5.24}$$

in which Q is the discharge, D and L the diameter and length of conduit, and g the gravitational constant. The lower boundary of the area AC (practically a straight line) must be located by a trial-and-error process which is somewhat time-consuming but which nevertheless gives an accurate location of the line.

It is generally agreed that the Gibson method is very accurate. As an application of the momentum principle this might be expected. The personnel requirements are not great because only one operator is required to run the instrument. Neither is cost of the equipment excessive. A series of tests consumes considerable time, however, because of the necessity for alternately shutting down the flow and bringing it back to a steady rate. Nevertheless, it must be concluded that the Gibson method offers a fairly simple and accurate approach to certain measurement problems that might otherwise be difficult.

5.4.4 Calibration of turbines, pumps, gates and valves

The calibration of a reaction turbine by the measurement of pressure drop in the scroll case has already been discussed. However, in some hydraulic systems it may be desirable, or perhaps necessary, to consider the turbine, pumps, gates or valves themselves as flow-meters for the system. To do that it is required that the pertinent hydraulic element be calibrated. The calibration is often done in the laboratory using hydraulic models, but it is preferable that the hydraulic element be calibrated in place, or check the laboratory-derived calibration by field measurements of discharge. For field calibration, discharge measurements are made by one of the three methods discussed above, if they cannot be made by current meter in the forebay or afterbay of the system where open-channel conditions exist.

In the case of turbines or pumps, relations of discharge versus power are generally desired. They may be defined by observing the metered power output or input during periods when discharge measurements are made for various load conditions. Suitable curves or tables may be developed from these test data to show the discharge (Q) that occurs for specific types of operation. Curves or tables may also be prepared from model test data, if the test data can be verified by a few discharge measurements. The calibration will change with time if there is a change in the efficiency of the turbine or pump resulting from long service or from other factors that cause deterioration.

If the range of operating conditions for a pump or turbine is narrow the calibration is simplified. For example, in such a situation where power input or output is metered, a simple relation of discharge versus power divided by head may be adequate. For a pump operated by an internal-combustion engine, where power was not metered but rotational speed was automatically recorded, the following calibration scheme has been used. For the most commonly used rotational speed, $(RPM)_r$, a base rating of discharge (Q_r) versus head was defined by current-meter discharge measurements. To obtain the discharge (Q_m) for other rotational speeds, $(RPM)_m$, an empirical adjustment relation of Q_m/Q_r versus $(RPM)_m/(RPM)_r$ was defined by the discharge measurements. (The method of defining the two relations is similar to that used in the constant-fall method of rating open-channel discharges, discussed in Chapter 3.) The use of head in the pump rating is analogous to the use of stage in the open-channel method. The use of rotational speed of the pump is analogous to the use of fall in the open-channel method. After the two relations have been defined, to obtain the discharge (Q_m) for a given head and a given rotational speed, $(RPM)_m$, the ratio $(RPM)_m$ to $(RPM)_r$ is first computed. That ratio is then used in the adjustment relation to obtain the ratio Q_m/Q_r. The value Q_r is the discharge corresponding to the given head in the base rating. The desired discharge (Q_m) is then computed by multiplying Q_r by the ratio Q_m/Q_r.

For gates and valves, relations of discharge versus gate opening for various appropriate heads are desired. They may be defined by observing the gate or valve openings during periods when discharge measurements are made for various operating heads. Measurements made over the full range of gate openings and heads will provide the data for establishing the required curves or tables. Generally the relations are in the form of discharge *(Q)* for gate openings expressed as a percentage of full openings, for pertinent operating heads. Curves or tables may also be prepared from model test data, if the test data can be verified by a few discharge measurements. As with turbines and pumps, the calibrations for gates and valves are subject to change with time as wear or deterioration occurs.

5.5 URBAN STORM DRAINS

Quantitative studies of urban storm runoff have been handicapped by a lack of proper instrumentation for metering the flow in sewers. An ideal sewer flow-meter should have the following characteristics:

(a) Capability to operate under both open-channel and full-flow conditions;
(b) Known accuracy throughout the range of measurement;
(c) Minimum disturbance to the flow or reduction in pipe capacity;
(d) Minimum requirement of field maintenance;
(e) compatibility with real-time remote data-transmission;
(f) Reasonable construction and installation costs.

Over the years many devices have been tested for use as sewer flow-meters. Wenzel (1968) has reviewed the methods and devices tested, weirs, depth measurement, depth and point-velocity measurements, dilution methods and venturi flumes, and found that all have disadvantages of one kind or another. Of those devices one of the most favourable was the flat-bottom venturi flume specifically designed for flow measurements in conduits by Palmer and Bowlus (1936). That flume has a throat of trapezoidal cross-section, a flat bottom, and upstream and downstream side and bottom transitions. The flat bottom permits debris to flow smoothly through the throat and the transitions reduce the head loss substantially below that which would be caused by a weir, for example.

Wenzel (1968), in his study, concluded that further effort in designing some new modifications of a venturi flume offered the greatest promise of success in developing a more satisfactory flow-metering device for urban storm drains. Accordingly three new variations of a venturi section have been designed and laboratory tested in the United States. The three types are briefly described below.

In recent years, ADVM have been used on a limited basis for sewer flows. These seem to have great potential for sewer applications and will no doubt find success.

5.5.1 United States Geological Survey sewer flow-meter

The United States Geological Survey (USGS) meter is a U-shaped constriction made to be inserted in a circular Pipe, as shown in Figure II.5.32. The symmetry of the design permits fabrication in two half-sections for easy transportation and installation. Moulds are available for fibreglass prototypes in pipe sizes from 0.61 to 1.52 m.

The overall length from toe to heel is 1.75 pipe diameters. The throat length, equal to one pipe diameter, and the approach and getaway apron slopes of 1 on 3, resemble venturi meter specifications. The constriction, in fact, is a venturi flume for open channel flows. For pressure flows it may be considered to be a modified venturi meter.

For sub-critical open-channel flows, the constriction dams up the flow, which then passes through critical depth as it spills through the throat. If the oncoming flow is supercritical, two conditions are possible: a hydraulic jump may be forced to form, which then spills through the throat and continues downstream as supercritical flow or, on steeper slopes, the oncoming flow may remain supercritical throughout the entire constriction. As discharge increases the water surface on the upstream side rises, touches the top of the pipe, and fills the upstream pipe, while the downstream side continues to flow part full. A discharge rating is available for each of these open-channel conditions.

Further increases in discharge trigger full-pipe conditions, which are also well rated. It is for these pressure-flow conditions that the question of head loss becomes of interest. Head loss, or backwater, is taken to be the increase in the upstream piezometric grade line caused by the presence of the constriction in the sewer line. For this constriction shape, the head loss is expressed as a function of the throat velocity head, H_L:

$$H_L = 0.04\frac{V^2}{2g} \qquad (5.25)$$

The constriction is considered to be self-cleaning. Inasmuch as sewers are generally laid to a self-scouring slope, any silting upstream from the constriction is expected to flush out on the next rise. The deposition of silt would have a negligible effect on the rating for small discharges, and no effect for high discharges.

The curved floor in the throat, parallel to the circumference of the pipe rather than being horizontal, retains some self-cleaning ability. It is a

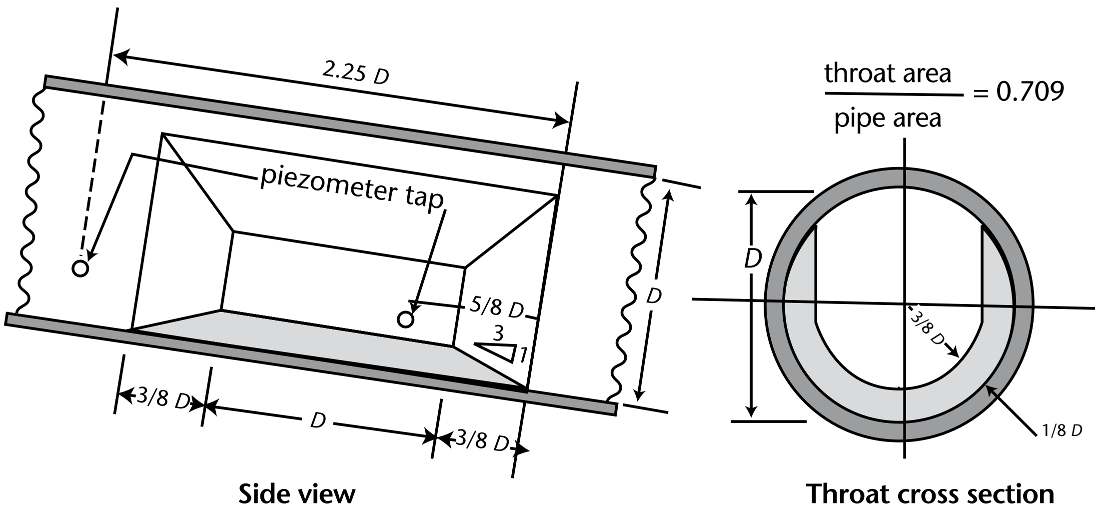


Figure II.5.32. Sketch of United States Geological Survey flow-meter in a sewer

compromise between a V-notch base which would have excellent rating sensitivity for small discharges but a tendency to clog with small debris, and the other extreme, a horizontal floor as in a Palmer-Bowlus trapezoidal constriction. The floor thickness, one eight of the pipe diameter, provides enough height to produce and maintain a stable hydraulic jump and also provides enough constriction (throat area is 0.709 of pipe area) to produce an adequate pressure drop for full-pipe flows. Yet it is low enough to maintain open-channel flow for a larger range of discharge than would be maintained by a thicker constriction. By leaving the upper part of the pipe unconstricted, a quick transition from open-channel to full-pipe flow conditions is assured and pressure build-up upstream from the constriction and head loss are minimized.

The pressures in the approach and in the throat of the constriction are measured remotely by pressure transducers. Dry nitrogen gas is bubbled at a constant rate through tubes to the two piezometer openings. The pressure at each opening is reflected to the head of the gas column where the transducer is located.

Data from the flow meter are entered into the system and converted to two digital numbers proportional to the two pressures measured. The two transducer outputs are applied to a dual analogue input amplifier that transforms them to analogue voltage levels, which are then applied to analogue-to-digital converters. Provisions are made so that one may compress, expand or shift the range at the analogue section.

The format under which data are recorded is dependent upon the conditions indicated by the system data inputs. The system logic inhibits data recordings during dry-weather, no-flow conditions. When flow begins in the sewer being monitored by the system, the pressure at the approach tap will increase. During the period when this pressure exceeds a preset value, as indicated by the corresponding analogue voltage exceeding a programmed level, recordings will be continuous on a 1-minute cycle. One or more recording precipitation gauges and an automatic water sampler are included in the instrumentation for studying urban storm runoff.

It is desirable that the meter be calibrated in place by current-meter discharge measurements. However, as a guide to the probable meter rating and for use until field calibration is completed, the following laboratory discharge equations are presented. The coefficients shown are for use with S.I. units:

(a) Pipe flowing full:

$$\frac{Q}{D^{5/2}} = 1.4553\left(\frac{\Delta h}{d}\right)^{0.517} \tag{5.26}$$

where Q is discharge; D is pipe diameter and Δh is the head differential between piezometer readings.

The constant 1.4553 includes the constant for the acceleration of gravity. The exponent 0.517 fits the laboratory data better than the theoretical exponent 0.5;

(b) Open-channel flow:

(i) Supercritical regime:

$$Q/d^{5/2} = 1.4553(h_1/d)^{1.58} \tag{5.27}$$

where h_1 is the depth above pipe invert at the upstream piezometer.

(ii) Subcritical regime – slope of culvert < 0.020:
For $h_1/d \geq 0.30$

$$Q/d^{5/2} = 0.722(h_1/d - 0.191)^{1.7564} \quad (5.28)$$

For $h_1/d < 0.30$

$$Q/d^{5/2} = 0.291(h_1/d - 0.177)^{1.3784} \quad (5.29)$$

(iii) Subcritical regime slope of culvert:

$$Q/ad^{5/2} = 0.273(h_1/d)^{2.708} \quad (5.30)$$

where

$$a = 2.15 + \left[(9.4943)(10^{11})(Slope - 0.008)^{6.7562}\right] \quad (5.31)$$

(c) Transitional flow between open-channel flow and full-pipe flow:

$$Q\Big/d^{5/2} = 2.6 \pm \left(\frac{|0.590 - h_2/d|}{0.164}\right)^{1/2} \quad (5.32)$$

where h_2 is the depth above the flow-meter invert at the downstream piezometer.

5.5.2 Wenzel asymmetrical and symmetrical flow-meters

A generalized drawing of the asymmetrical venturi section devised by Wenzel (1975) is shown in Figure II.5.33. The symmetrical venturi section differs from the asymmetrical type shown by having identical constrictions on either side of the vertical centreline of the pipe. The constriction consists of a cylindrical section, whose radius is greater than that of the pipe, with entrance and exit transitions having a slope of 1 on 4. The cylindrical section intersects the pipe wall a distance *S* from the centreline, thereby maintaining the invert region free of obstruction so that self-cleaning is facilitated. In all laboratory tests a constant value of 0.1 was maintained for *S/D*, but the ratio *r/D* was varied to provide various ratios of throat area to pipe area for testing. A throat length between 2.25*D* and 4.0*D* is recommended. The upstream piezometer tap is located approximately *D/*3 upstream from the beginning of the entrance transition; the downstream piezometer tap is located approximately at the centre of the throat. As mentioned earlier, no information on the field performance of the Wenzel flow-meters is as yet available.

5.6 AUTOMATED COMPUTATION OF FLOW THROUGH WATER CONTROL STRUCTURES

Flow through a dam and/or water control structure can involve several different types of computational processes, such as those described in preceding chapters and in the preceding paragraphs of this chapter. For instance, a lock and dam structure may have flow through various types of gates, free-flow over spillways, navigation locks, power generation turbines, pumps, siphons and sluices. Modern computer technology has provided the means to develop programs that compute flow quickly, automatically and simultaneously for each of the separate structures in a water control system. One such program, entitled DAMFLO.2, has been in use by the USGS for several years. It is beyond the scope of this Manual to give a complete and detailed description of this program, however a general description is given in the following paragraphs. Complete documentation and descriptions of the program components can be found in a report by Sanders and Feaster (2004). Program DAMFLO.2 computes, tabulates and plots flows through sluice and Tainter gates, crest gates, lock gates, spillways, locks, pumps and siphons using hydraulic equations like those given in preceding sections of this chapter.

5.6.1 General description of Program DAMFLO.2

DAMFLO.2 is a set of programs created using procedures and language developed by the SAS Institute (Statistical Analysis System) (1993). The SAS software was used because of its integrated plotting, interactive tabling, data entry by forms and various statistical, date-time and data-handling programs.

DAMFLO.2 is an example of the use of an integrated software package that produces an integrated data-processing program. The program is integrated in that all tables and files of computed unit-value and daily-value flows, as well as data plots, appear automatically without the need to run several programs. Practically all of the tabulated data, including flow types and error flags, are presented graphically and by interactive table so that the hydrographer will not have to scan hundreds of printed data items.

Coefficients and ratings for the hydraulic equations used in DAMFLO.2 to compute flow through the various controls at a dam are calibrated using flow measurements. Time-varying data, such as water surface elevations upstream and downstream of the dam, and gate openings are recorded in the field at or near the dam and are stored in the database in a single file as sequential groups of data types. Fixed data that describe the physical features of the various structures are stored in a SAS database using the SAS forms capability.

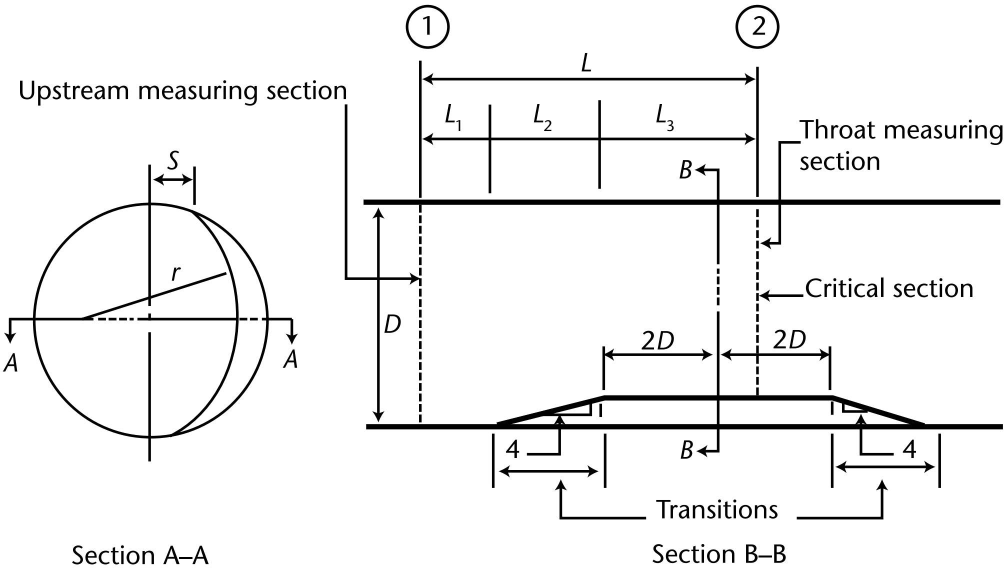


Figure II.5.33. Sketch of Wenzel asymmetrical flow-meter in a sewer (after Wenzel, 1975)

Programs that compute flow through each outlet are executed by separate commands stored in a single file for each structure. This file is created by the hydrographer, external to SAS, using a text editor. DAMFLO.2 is considered modular in the sense that any configuration of outlet structures can be modeled.

Weir, Tainter-gate, sluice-gate, locks and crest-gate flows are computed in DAMFLO.2 using methodology documented by Collins (1977), Stuthman and Sanders (1982) and Sanders and Feaster (2004). These methods are, for the most part, identical to the hydraulic equations given in this chapter and other parts of this Manual. Some variations may be noted. Submerged weir flow is computed using the unit-fall method as described in Chapter 3 of this Manual and as described by Kennedy (1984). Methods of adjusting free-weir flow for submergence are used, as described by Hulsing (1967). The program also allows negative weir and orifice flows at low-head dams when the tailwater is higher than the headwater. Flow data computed for non-standard outlets can be stored in a separate database, then retrieved, and added to flows computed for the standard outlets by DAMFLO.2.

5.6.2 Time-varying input data

Time varying data, such as water surface elevations and gate openings, are specified in data files at appropriate time intervals such as every 30 minutes, or every 60 minutes and so forth. The program requires only that the retrieval of the time-varying data groups be sequential by type in one file. For example, all date, time and gauge-height data would be sequentially filed for a headwater gauge-height data type (data group 1), followed by all date, time and gauge-height data for a tailwater gauge-height data type (data-group 2), followed by all date, time and gate-opening data for the gate-opening data type for a Tainter gate (data-group 3) and so forth. The order of the data groups within the time-varying data file can be specified in any order by the hydrographer.

The hydrographer can select time intervals for computations that differ from the time intervals of the time-varying input data, within certain constraints. This is especially convenient when the time-varying input data are recorded at different time intervals. Time varying data can be interpolated by linear or stair-step interpolation. In stair-step interpolation, such as would be used for Tainter gate data, a preceding input value is held constant until changed by a succeeding input value.

5.6.3 Fixed input data

Physical dimensions of the various structures, such as gate widths, sill elevations, lock dimensions and so forth are provided as fixed data. Ratings and hydraulic coefficients are also provided as fixed input data. The fixed data are divided into three groups, stage limits, hydraulic ratings and physical and hydraulic parameters.

The fixed input data for stage (or gauge height) consist of minimum and maximum expected stage, maximum expected rate of change of stage and datum corrections. Each set of fixed input data for stage is referenced using a stage-gauge identification number, which should be assigned sequentially beginning with 1.

The fixed input data for hydraulic ratings consist of rating type and a set number. The set number refers to the outlets (or outlet) for which the hydraulic rating applies. Hydraulic ratings are available for five different outlet types: (1) Tainter gates, (2) spillways, (3) pumps, (4) lock gates and (5) crest gates. For example, if the outlet type is a crest gate, the hydraulic ratings available are spillway head versus discharge or head-over-gate versus discharge.

Physical and hydraulic parameter data at a dam are the data needed to define the physical properties of the outlets, such as sill elevations and gate widths and various hydraulic coefficients and computational methods associated with the outlets.

5.6.4 **Program output data**

The program output allows for three options. Option 1 will provide primary computation tables and plots, and will send them to a computer screen. Option 2 will send the computations and plots to a printer. Option 3 will print the primary computations tables, without printing the plots.

The unit-value primary computations display input data, submergence ratios, computed flows, flow types, and warning messages. The hydrographer can specify the time interval for the tabulations. The daily value primary computations display daily minimum, daily maximum and daily mean statistics of flow, and summarize warning messages. Daily means of all instantaneous flows are computed using trapezoidal sub-areas of the hydrograph. The trapezoidal method is not used for lockage flows because those flows are mean flows over a computational time interval and are not instantaneous.

Total computed unit-value flows also are plotted with appropriate warning flags. It is possible to plot tailwater elevations against total computed outflow to evaluate the amount of backwater downstream of the dam, or more specifically, the lack of backwater at high stage when the dam outlets are totally submerged and submergence ratings are no longer valid. A hydrograph of daily minimum, maximum and mean flows also is produced by the program.

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CHAPTER 6

ANALYSIS AND COMPUTATION OF DISCHARGE RECORDS USING ELECTRONIC METHODS

6.1 GENERAL

In many countries streamflow records are published annually. The 12-month period used, which is known as the water year, does not usually coincide with the calendar year. In the western hemisphere the water year runs from 1 October to 30 September and is designated by the calendar year of the last nine months. For example the 2006 water year runs from 1 October 2005 to 30 September 2006. The following considerations govern the choice of the 12 months that will constitute the water year. The 12-month record is essentially an inventory of the water supply. As with any inventory, it should be made when the stock on hand (available water resource) is at a minimum. That is the case in most of the western hemisphere on 30 September, when the growing season is at an end. Not only are groundwater, soil moisture and surface storage at or near a minimum on that date as a result of heavy water use during the preceding summer, but the replenishing rains of autumn have not yet begun and streamflow is also near minimum. In short, the 12-month period to be used as the water year is determined by the climatic regime of the region. For the publication of data however, some countries, notably the United Kingdom of Great Britain and Northern Ireland, use the calendar year.

A daily record of stage and discharge, along with momentary values of peak discharge and minimum flow, is computed for the water year from the record of stage and the discharge rating for the gauging station. Prior to the early 1960s, almost all streamflow data were processed by hand and desktop calculators. After that time, some of the processing steps, such as drawing rating curves, were accomplished by hand methods and transferred to the computer by keying in the necessary data. The type of stage recorder determined whether the computations were performed manually or by an electronic computer. Since the early 1970s, computer analysis became more prevalent, and today almost all discharge records are computed by automated methods. In many instances, rating curves are still being developed by hand plotting methods, and entered to computer programs by keyboard; however, even these are becoming less frequent.

Instrumentation for the collection and field recording of time-series data has attempted to keep pace with computer capabilities for processing the data; however, a noticeable lag has resulted. The evolution of data-collection methods shows a progression from analog recorders to digital recorders and finally to Electronic Data Loggers (EDLs) and Data-Collection Platforms (DCPs). Even so, many digital recorders still are in use as primary instruments and graphic recorders are frequently used as backup instruments. Part of the reason for this is a lag in the acceptance of electronic data loggers, which is due to lack of funds to support a full conversion. With a mixture of instrumentation still in use, it becomes important that data-processing software be able to accommodate the various formats of input for time-series data.

Field measurements traditionally have been recorded on paper forms. This form is still an accepted mode for these types of measurements. However, electronic data files from hydroacoustic instruments and electronic field notebooks have been developed that may eventually become the standard for recording field notes. Processing software must be able to accept both types of input: keyboard entry from field notes recorded on paper forms and direct entry from electronic data files and field notebooks.

In addition to changing instrumentation, increased capabilities have developed for the analysis of streamflow information. Traditionally, streamflow information is produced through the use of stage-discharge relations, with adjustments for shifting controls. For some stations more complex computation procedures are used to account for variable backwater and rate-of-change in stage. Structures such as dams, spillways and turbines, are used at some stations to measure streamflow. The use of electromagnetic velocity meters, acoustic velocity meters and Acoustic Doppler Velocity Meters (ADVM) has increased the ability to continuously monitor stream velocity, and thereby provide an index of variable backwater. Unsteady-flow models, such as the Branch-Network Dynamic Flow Model (BRANCH) by Schaffrannek and others (1981), also have been accepted as methods to compute streamflow records. An unsteady-flow model uses detailed hydraulic characteristics of a stream reach and has the capability to provide streamflow information at virtually every location in the stream reach. This capability is a distinct advantage over the traditional gauging station that provides information at only one location.

There is increasing need for streamflow information on a real-time, or near-real-time, basis. This has led to remote sensing and transmitting systems where data are received in the office within minutes, or at the most hours, of the time of occurrence. These data usually are processed immediately upon reception in the office using automated computer systems. In many instances the real-time data are made available through the internet to almost anyone. Information of this type should be classified as operational, having more uncertainty than information that are subjected to verification, interpretation and review. Operational data and information should be reviewed before publication and archiving.

Changing technologies of data collection and processing require changes in computer software. There is no doubt that this will be a continuing process as new and better computer technologies become available. To produce an accurate and consistent data base, it is important that certain procedures be standardized. The traditional hand methods, and some of the more recent computer methods, have been described in various publications such as Rantz and others (1982), Kennedy (1983), Kennedy (1984), Marsh and Stephenson (1976) and World Meteorological Organization (WMO) (1971). Many of the field and office procedures, as well as the equipment, described in those reports are still valid. In particular, the concepts and theory of surface-water analysis are accepted. However, much of the information in those reports applies to processing techniques where hand methods are used either totally or partially. The following sections of this Manual describe standards for data analysis and processing that can be incorporated into computer programs. As a basis for these standards, the report by Sauer (2002) has been used as a major source of information and many sections of that report are used verbatim in the following sections.

In the United States of America, the computer program used for computing and processing streamflow records is known as the Automated Data Processing System (ADAPS) (Dempster, 1990; United States Geological Survey (USGS), 2005). Other countries, such as the United Kingdom, Canada and France, have developed their own versions of automated computer programs for computing streamflow records. Even though the various international versions may be different in respect to procedures or sequence for inputting field-recorded data, computing unit and daily values of streamflow, outputting data tables and other aspects of analysis and computation, the end result is essentially the same.

6.2 SURFACE WATER DATA AND INFORMATION

Surface-water data and information are composed of a number of measured and computed variables. The words data and information, as used in this Manual, are intended to have special meanings. The term data is used for the results obtained from the measurement of a basic variable, which cannot be repeated. Data can be accepted, qualified or rejected, but they cannot be modified without compromising their identity. Any change or modification of a data value converts that value into information. For example, if an original measurement of gauge height is corrected for sensor error (such as drift related to time, gauge height, temperature or other factors), the new value of gauge height is information. Another example would be the use of a gauge-height value and a relation of gauge height to discharge to compute a value of discharge. The computed discharge value is information. Unlike data, information can be modified, as would be the case if a stage-discharge relation were revised.

The term unit value is used to denote a measured or computed value that is associated with a specified instantaneous time. In addition, unit values generally are part of a time-series data set. For surface- water records, unit values for all parameters always should be instantaneous values. Some parameters, such as velocity, tend to fluctuate rapidly and a true instantaneous value would be difficult to use in the analysis and processing of the records. Some instruments are designed to take frequent (for example, every second) readings, temporarily store these readings and then compute and store a mean value for a short time period. For these situations, the field instruments should be programmed to record mean unit values for very short time intervals (one to two minutes) so they can be considered for practical purposes to be instantaneous unit values.

Daily values are measured or computed values of a parameter for a specific date only. The time of the daily value is not required, although for certain daily values, time sometimes is stated. Examples of daily values are daily mean value, maximum instantaneous value for a day and minimum instantaneous value for a day. In the case of maximum and minimum instantaneous values for a day, the time of the value usually is stated.

6.3 ESTABLISHING A SITE IN THE ELECTRONIC PROCESSING SYSTEM

The processing of field data varies depending on the type of gauging station, such as field collection equipment, computational methods, information output requirements and other aspects of each station. For instance, there are several types of stage data collection methods, such as observer data, Analog Digital Recorders (ADR), EDL and DCP. These were described in Chapter 4 of Volume I. Different types of ratings are also possible, such as simple stage-discharge ratings, stage-area ratings, ratings, stage-fall ratings, control structure ratings and others as described in the first 4 chapters of Volume II and in Chapters 6 and 7 of Volume I. Various output requirements may also be a factor, although daily values of discharge are usually the final result. Other output requirements are sometimes required, such as unit values of stage and discharge, daily values of reservoir surface area and contents and various tide data.

The first step, therefore, is to establish the gauging station in the electronic processing system. Each gauging site must be established to accommodate the various input, computation methods and output requirements specific that station. The specific requirements for a gauging station may change from time to time and the electronic processing system should be able to accommodate these changes. The electronic processing systems used by most countries are versatile enough to accommodate most of the combinations of data input and output.

It should be noted that the sequence of steps may vary from that shown in the following sections. For instance, different types of gauging stations may require a different sequence of steps or computations. Also, some steps such as analyzing the data are a continuing process from the beginning to the end of the water year. The final written document that summarizes the station analysis may not be finalized until the end of the water year.

6.4 ENTRY OF FIELD DATA TO THE ELECTRONIC PROCESSING SYSTEM

The second step required for the processing of surface-water data is the entry of field data and information to the electronic processing system. This process will include unit value data and field measurement data and information. Field measurements can include discharge measurements, gauge-datum leveling data, crest-stage gauge data, channel and control cross-section data and other miscellaneous data and information.

6.4.1 Unit value data

The recording of unit value data has evolved from simple hand written notes (observer data), to analog recorders, to digital recorders, to sophisticated programmable data loggers and to direct data transmission to the computer. Although the trend today is toward the use of programmable data loggers and direct data transmission, digital recorders still are widely used and some use of analog recorders and hand written observer records. Therefore, the electronic processing system must accommodate each of these types of data.

Preparation of unit value data for electronic processing should follow a basic sequence. However, because different methods are available for collecting and recording field data, there may be instances where the preferred sequence cannot be followed. The following sequence is advised:

(a) A copy of the original, unedited unit values should be archived before any editing, conversions or computations are made. All editing, conversions and computations should be performed using an electronic copy of the original data;
(b) The unit values should be translated into a standard format;
(c) The unit value times should be corrected for clock errors, if applicable;
(d) Conversions to UTC time should be made so that all unit value data can be related to standard time or daylight savings time, as required;
(e) The unit values prepared in this manner then can be used for all further computations, analysis and archiving, as described in this Manual.

Various types of unit value data can be entered into the electronic processing system. These data include unit values of gauge height, velocity or velocity index, spillway gate opening or index, turbine pressures, navigation lockages and other readings associated with structures. For some gauge sites, multiple data sets of unit values may be available for a given parameter. For instance, a stream affected by backwater may have two gauges at different locations for the purpose of measuring gauge height. Unit values of gauge heights or elevations are required for almost all gauging sites.

6.4.2 Sources of unit value data

The sources of unit value data are described in detail in Chapter 4, Volume I. A brief description of the methods of obtaining, recording and entering unit value data to the electronic processing system is given in the following paragraphs. Each set of unit values must be identified as to the source and method of acquisition.

Observer data

At some gauge sites gauge readings are made by an observer. These readings are recorded, along with date and time of the reading, on a preprinted form. Such readings may be used as the primary set of unit values for the station or for backup and verification of another measuring and recording method. The hand written unit values made by an observer must be entered into the electronic processing system by direct keyboard entry. The date and time must be entered for each unit value and the time zone designation must be entered for each set of unit values.

Analog recorders

Analog recorders are frequently used to record the gauge height, or other parameters, as sensed by a float, pressure system or other device connected to the recorder. Analog recorders provide a continuous trace of the measurements on a graphical chart that is driven by a clock to provide a time scale. Unit value data from these charts are entered to the electronic processing system through the use of an automatic, or hand operated, digitizer. The digitizer enters unit values from the chart at time intervals specified by the hydrographer. Beginning and ending dates and times, and the time zone designation, must be entered for each segment of chart that is digitized. Analog records may be used as the primary unit values for a station, but are more frequently used for backup and verification of unit values collected with a different method.

Automated digital recorders

The Automated Digital Recorder (ADR) is a device that records data on a narrow paper strip by punching a series of holes that are digitally coded to represent the unit value. The paper strip advances after each punch and data are recorded at a specified time interval, commonly 5, 15 or 60 minutes. Other time intervals may be used but the time interval is uniform for each gauge. Unit value data are entered to the electronic processing system by passing the paper strip through a digital tape reader. Starting and ending dates, times, and the time zone designation must be entered for each processing period. ADRs are frequently used as the primary recording instrument for a gauge site but are also used as backup and verification for other types of instruments.

Electronic data loggers

Various types of Electronic Data Loggers (EDLs) are in use for recording unit value data. These devices receive data from a sensing instrument and record the unit value in electronic memory. Data are extracted from the data logger either by removing the memory chip or by reading data from the memory into an external storage module or field computer. Because of the many configurations and types of data loggers currently in use, and because changes occur frequently, it is not practical to attempt a description in this Manual. The process of entering data from these types of recorders primarily is electronic. Electronic data loggers have the advantage over analog recorders and ADRs because they can be programmed to sense and record according to pre-defined rules. A recording system of this type results in a variable time interval between unit values, and necessitates the recording of the time and date associated with each unit value. If the recording time interval is constant, then most electronic data loggers do not record the time and date associated with each unit value. For either method, variable or constant recording interval, the starting and ending date and time must be entered for the period of record being processed. Electronic data loggers frequently are used for the primary recording instrument but in some cases they may be used only for backup.

Data-collection platforms

Data-Collection Platforms (DCPs) are field systems that store data electronically for a relatively short time (two to four hours) and then transmit it to an office computer. For some types of DCPs, storage may be comparable to an electronic data logger and the data can be retrieved in similar fashion. DCPs are frequently operated in conjunction with an electronic data logger, ADR or analog recorder. A variety of gauge and recorder configurations is possible. Where two or more recorders are used, one should be designated the primary instrument; the DCP is frequently given that distinction. In some instances the DCP is the only instrument used and the primary record is received directly in the office. Unit value data transmitted and received by satellite automatically are tagged with date and time, which is determined from Universal Coordinated Time (UTC).

Other

Unit value data that are stored on other computer systems can be transferred to the electronic processing system by use of card images or other standard formats. One of the recorder types described above usually is designated as the primary recorder for computing the primary records of gauge height, discharge, reservoir contents or other parameters. A second recorder is frequently operated in conjunction with the primary recorder and is designated the backup recorder. In the event of the malfunction of the primary recorder, the electronic processing system should allow the entry of unit values from the backup recorder as a substitute for the primary recorder. These substitute unit values should be identified with a flag as to the source of the backup records. These records also should be subject to all further analysis, such as time corrections, parameter value corrections and others, as described in the following sections.

6.4.3 Unit value recording time interval

The time interval between recorded unit values may be a constant value or variable. The programmable data logger allows the recording interval to be varied according to user-specified rules. The variable time interval can be based on the value of the parameter being recorded, the time length since the last recording, the rate of change of the parameter value being recorded, the value or rate of change of some other parameter or some combination of these. The electronic processing system should be able to accommodate either method of data recording, constant or variable time interval.

6.4.4 Time system requirements

The time system used in most field data-collection systems is based on the local time at each gauging location. For most of the United States, the local time is a changing time system where the clock is advanced one hour in the spring, and set back one hour in the fall. The time during the summer period commonly is referred to as daylight savings time, and the remainder of the year as standard time. The advent of the satellite DCP has required the use of UTC for DCP field instruments. Additionally, some gauge sites are operated year around on local standard time without making the change for daylight savings time. Consequently, there is a mixture of time systems being used. The surface-water electronic processing system must accommodate the entry of data in any of the time systems. Therefore, all data entry must include a designation of the time system at which the data were recorded.

All times, both for time series data and for measurement, data will automatically be converted to UTC time for storage in the electronic processing system. Therefore, time adjustments for the one-hour daylight savings time offset (as used in the United States) automatically will be accounted for when times are converted to UTC. The hydrographer will be able to perform computations, such as for daily mean values of streamflow, using any specified time system. The electronic processing system automatically will make the necessary time conversions, including changes between standard and daylight savings times, prior to making the computations. Likewise, unit values of gauge height, discharge or other parameters would be retrieved using a time system selected by the hydrographer.

6.4.5 Standard format

All unit value data stored in the electronic processing system should conform to a standard unit value format. This format essentially means that the electronic processing system should convert all unit values to engineering units and assign times and dates based on the time system used for field recording of data. Time adjustments for the purpose of converting the unit value times to standard UTC time are made automatically. Time corrections made for clock errors should be made after the data are converted to a standard format. Parameter value corrections are made on the basis of hydrographer instructions after data are entered in the electronic processing system. Additional details regarding time and parameter corrections are described in following sections.

6.4.6 Field measurement data

Various types of field measurements are made at surface-water gauging stations, each providing various kinds of data and information. These include measurements of stream discharge, leveling for gauge datum checking, crest-stage gauge measurements, channel and control cross-section measurements and other miscellaneous data and information. Usually each type of field measurement is recorded on a form designed especially for that type of measurement. The electronic processing system should be able to receive, process and store the field measurement data and information so the data can be used in other parts of the electronic processing system.

Most field data are recorded on paper forms and must be transferred to the electronic processing system by keyboard. Field data and information that are recorded electronically in a field computer will require an interface between the field computer and the office computer to transfer the data automatically.

Discharge measurements

The electronic processing system should have the capability to receive and store essentially all of the data and information recorded on discharge measurement note sheets. This capability should include the information shown on the front sheet of the notes and the detailed data shown in the body of the notes. In the case where discharge measurements are recorded in electronic field computers, the electronic processing system would receive the data and information automatically through an interface.

Although the electronic processing system should be able to receive all data (front sheet and inside body) from a discharge measurement recorded on paper forms, it is not mandatory that the inside body data and information be entered. This part of the measurement is not normally used in the processing of daily discharge records. The main purpose for entering the data and information from the inside body would be for computational checking and for special studies.

The original measurement is either the data or information recorded on paper notes or in an electronic field notebook. If the measurement was recorded on paper, those original paper notes are saved for archival. If the measurement was recorded electronically, the first electronic copy entered to the processing system becomes the archival copy. For this reason it is mandatory that the entire measurement recorded in an electronic field notebook, including all of the individual data elements, be entered in the electronic processing system.

Discharge measurement entry requirements

Discharge measurement data will be acquired from one of several different methods as described in Chapters 5 through 9 of Volume I. The input forms presented to the user of the electronic processing system should be designed to conform to the measurement method. That is the input form for measurement summary information for a specific method of measurement (for example, a wading measurement with a mechanical current meter) would have input items specific to that method of measurement and would omit input items that are not applicable to that method. The specific measurement data on the inside of the discharge measurement, although not mandatory, would be entered on separate input forms.

Although separate input formats are used for the various types of measurements, all measurements, including indirect measurements, should be numbered consecutively and maintained in one file of discharge measurements. The numbering sequence should begin with one for the first discharge measurement of record, and continue consecutively throughout the period of record, with all discharge measurements numbered in chronological order. Discharge measurement numbers may contain alphabetic characters (for example, 127A, 127B and others) to allow insertion of a measurement in an established sequence. Renumbering of discharge measurements should be discouraged.

The rapid development of new instruments for measuring discharge, particularly acoustic instruments, has presented a challenge for developers and maintainers of discharge computation data processing programs. For example, the enormous amount of data available from an ADCP measurement must be distilled into the most important information for processing programs to use. Also, it is desirable to identify the type of instrument being used.

Gauge datum leveling

Leveling for the purpose of establishing or checking the datum of reference marks, benchmarks, staff gauges, wire-weight gauges and other gauge features is performed routinely or when problems arise at most gauging stations, as described in Chapter 4 of Volume I. Guidelines for leveling procedures as performed in the United States are described by Kennedy (1990). The electronic processing system should provide capability to accept leveling data and should be able to produce an analysis and summary of the leveling information.

Crest-stage gauge data

Crest-stage gauges are special gauges capable of recording the highest level of a flood peak. These gauges may be operated independently as a partial record site, or they may be operated as a continuous record site to verify the peak gauge height. A special note sheet is usually used to record data and information for crest-stage gauges. The electronic processing system should be able to accept these data.

Channel and control cross sections

Data defining cross sections of the stream channel and/or control are useful in rating curve analysis. Unsteady-flow model methods of computing stream discharge must have cross-section data at intervals along the stream reach for which the model is defined. The electronic processing system should allow input of items necessary for defining the cross-section location and the descriptors for each cross section. In addition Manning coefficients may be required and should be variable, both horizontally and vertically. For some cross sections that are considered section controls, a weir coefficient (*C*) should be an optional entry, which also may be variable with stage. Transverse stationing for cross sections should begin on the left bank of the stream and increase from left to right. If survey data are entered with transverse stationing that increases from right to left, the electronic processing system should provide an automatic conversion of the data to the left-to-right format. The electronic processing system also should accommodate input of cross-section data that were collected and recorded electronically.

Miscellaneous field notes

Miscellaneous field notes occasionally are made at most gauge sites. These may be just a gauge reading, a measurement of some feature or variable, a record of maintenance or simply written comments. The electronic processing system should allow entry of these notes.

6.5 VERIFICATION AND EDITING OF UNIT VALUES

Unit values for the various parameters, such as gauge height and velocity, must be carefully checked and verified before being used in further analysis. Erroneous or suspicious data may require editing and appending special identification codes (flags) to individual values. Before any editing is performed, the original unit values should be set aside for archiving. This section of the Manual describes techniques for verification and editing, which includes time corrections, unit value corrections, datum adjustments and various comparisons and cross-checking. All verification, editing and time corrections must be performed on a copy of the original data, and not on the original. This copy will become the work file, and also will be archived following completion and finalization of the records.

6.5.1 Times and dates

Unit values of gauge height and other streamflow parameters generally are recorded in field instruments at a fixed time interval, such as every 15 minutes, one hour and so forth. The time and date associated with each unit value are not always recorded, but are determined on the basis of the initial time and date, and the recording time interval. Times and dates are recorded for each unit value when field recorders are programmed for variable time-interval data. Field instrument clocks are fairly reliable, but occasionally clock errors will result. True times and dates are those noted by the hydrographer using his watch and calendar at the time the field instrument is serviced. Servicing would be at the beginning and end of a record period and occasionally at intermediate points of a record period. The hydrographer should also note the time-system designation, such daylight savings time whenever the time and date are noted. Times, dates and time system designations noted by hydrographers will be used as the basis for making time corrections, standard and daylight savings time adjustments and conversion to UTC of the unit value data.

Data acquired by satellite DCP installations will have UTC times and dates assigned automatically. These times and dates are considered accurate and do not need adjustment or correction.

6.5.2 Time corrections and adjustments

Time corrections to account for clock errors may be necessary for unit value data recorded in the field. In addition, all unit value times must be adjusted to UTC time for purposes of archiving. These time corrections and adjustments do not apply to data collected by way of a satellite DCP because those data are considered correct as collected.

Clock error corrections

The simplest case of clock error is where the beginning time and date are correct and the ending time and date are incorrect by a known amount. Lacking any evidence of intermediate clock or recorder problems, it usually is assumed that the clock error is a gradual and uniform error. The correction for this type of error should be prorated uniformly throughout the record period.

A somewhat more complex case involves a clock or recorder malfunction somewhere in the middle of the record period or where the clock was set wrong at the beginning of a record period. One or more instances of intermediate clock problems may result in some cases. The time-correction procedure should allow the hydrographer to assign time and date values at more than one place within a record period, and the electronic processing system should adjust all intervening unit value times accordingly. Occasionally, it may not be possible to determine why the time for a record is incorrect, or at what point in a record that timing problems occurred. A hydrographer may need to make arbitrary time assignments based on their best judgment.

In some cases intermediate time and date readings may be available from discharge measurement notes or miscellaneous field notes when the gauge was visited but the record was not removed. The electronic processing system should automatically retrieve dates and times from the field note entries for checking clock performance. This requires that the unit value file has been marked in some way so the hydrographer can identify the place in the record where the correct times and dates apply. Such readings would be treated the same as described above and corrections would be made by linear proration between adjacent readings.

Past methods for making time corrections provide a method whereby occasional unit values are dropped, or added, in order to account for a time error. This method is not considered as good as the linear proration method and should not be used.

The standard time-correction method, or linear proration method, described herein will result in unit values of gauge height (or velocity or other parameter) that will not be on the even hour, or 15 minutes or other even time. This is not considered detrimental to the record. If unit values of gauge height (or other parameter) are needed on the even hour or other even time interval, they can be obtained by interpolation between the time-adjusted values.

Time differences caused by a change to or from daylight savings time should not be treated the same as clock error. If a clock error exists during a period of record where the time changed because of daylight savings time, the clock error should first be prorated by assuming a uniform time designation for all of the period of record being processed. The electronic processing system should adjust times and dates input from field notes to the same time designation. The clock error is then corrected according to the hydrographer's instructions. After clock error corrections are made, the record is automatically converted with the electronic processing system to UTC time for storage and archiving. No unit values would be dropped or artificially added because of the daylight savings time change.

Universal coordinated time (UTC) adjustments

All data and information should be stored with UTC. Therefore, following the standard time-correction method for making clock error adjustments, the electronic processing system should automatically adjust all local times to UTC. This is a simple process of adding the UTC time offset to the recorded local times. The recorded local times must have a time-zone designation as part of the input to define the time-zone system used for recording.

Unit values used in other analyses, such as computation of daily values, will adjust the UTC times to whatever time system is designated by the hydrographer. In this way, the electronic processing system can produce records on the basis of any designated time system. The time adjustments resulting for a period where time changes from standard time to daylight savings time and for a period where time changes from daylight savings time to standard time is illustrated in Figure II.6.1. Also shown in this figure are unit values that would be used for computing daily values for days that change between standard time and daylight savings time. Note that all recorded unit values are used in the computations and none are dropped or artificially added. The day when time changes into daylight savings time will contain 23 hours and the day when time changes out of daylight savings time will contain 25 hours.

6.5.3 Parameter value verifications

Unit values of gauge height and other parameters that have been automatically measured and recorded by field instruments always should be carefully inspected and verified before accepting them for further analysis and computations. Various methods are available to make this task relatively easy. The most frequently used methods are threshold comparisons, rating comparisons, direct reading comparisons and graphical methods. Of these, graphical methods are the most versatile and can be easily adapted to any of the other methods.

Example A. Time changes from standard time to daylight savings time

Unit Value	Local Date	Local Time	UTC Time	Time Zone
xxxx	04/03	2300	0400	EST
xxxx	04/03	2400	0500	
xxxx	04/04	0100	0600	
xxxx		**0200**	0700	EST
xxxx		**0400**	0800	EDST
xxxx		0500	0900	
xxxx		0600	1000	
xxxx		0700	1100	
xxxx		0800	1200	
xxxx		0900	1300	
xxxx		1000	1400	
xxxx		1100	1500	
xxxx		1200	1600	
xxxx		1300	1700	
xxxx		1400	1800	
xxxx		1500	1900	
xxxx		1600	2000	
xxxx		1700	2100	
xxxx		1800	2200	
xxxx		1900	2300	
xxxx		2000	2400	
xxxx		2100	0100	
xxxx		2200	0200	
xxxx		2300	0300	
xxxx	04/04	2400	0400	
xxxx	04/05	0100	0500	
xxxx		0200	0600	
xxxx		0300	0700	
xxxx	04/05	0400	0800	EDST

24 unit values used to compute daily value for 4 April

Example B. Time changes from daylight savings time to standard time

Unit Value	Local Date	Local Time	UTC Time	Time Zone
xxxx	10/24	2300	0300	EDST
xxxx	10/24	2400	0400	
xxxx	10/25	**0100**	0500	EDST
xxxx		**0100**	0600	EST
xxxx		0200	0700	
xxxx		0300	0800	
xxxx		0400	0900	
xxxx		0500	1000	
xxxx		0600	1100	
xxxx		0700	1200	
xxxx		0800	1300	
xxxx		0900	1400	
xxxx		1000	1500	
xxxx		1100	1600	
xxxx		1200	1700	
xxxx		1300	1800	
xxxx		1400	1900	
xxxx		1500	2000	
xxxx		1600	2100	
xxxx		1700	2200	
xxxx		1800	2300	
xxxx		1900	2400	
xxxx		2000	0100	
xxxx		2100	0200	
xxxx		2200	0300	
xxxx		2300	0400	
xxxx	10/25	2400	0500	
xxxx	10/26	0100	0600	
xxxx	10/26	0200	0700	EST

26 unit values used to compute daily value for 25 October

Figure II.6.1. Comparison of time systems where daylight savings time is used. (Coordinated Universal Time (UTC); United States Eastern Standard Time (EST); Eastern Daylight Savings Time (EDST))

Threshold comparisons

A threshold is a minimum or maximum value that can help detect unit values that might be erroneous. Thresholds can be compared directly to unit values, or to differences between adjacent unit values. Testing a period of record against a set of thresholds is performed automatically with the electronic processing system. The hydrographer is alerted whenever a unit value exceeds the threshold value. Thresholds can be established by the hydrographer or they can be automatically computed based on a period of record.

The set of thresholds should consist of:
(a) A high-value threshold;
(b) A low-value threshold;
(c) A maximum difference threshol;
(d) A flat-spot threshold (maximum time for constant values).

Thresholds should be used to detect values that are unusual and outside the normal expected range of the data. For instance, an ADR punch recorder malfunctions and punches additional holes in the paper tape, which translates to unit values outside of the expected range of values. The threshold check should alert the hydrographer to this condition. Maximum and minimum threshold values should be set at or near the maximum and minimum values actually experienced during the past three to five years of record. The difference threshold also should be set at or near the largest valid difference during the past three to five years.

Selection of threshold values should be based, if possible, on an analysis of the observed record for the past three to five years. This analysis should be performed with the electronic processing system and should furnish listings of the 20 highest and lowest peak unit values during the period. The electronic processing system also should provide the 20 greatest differences between consecutive unit values, and the 20 longest time periods during which there was no change in unit values. This type of analysis would provide data for the hydrographer to use in selecting appropriate thresholds and would be performed every three years, or whenever it is desired to change thresholds.

Threshold checking, if used primarily for the purpose of identifying unit values that are outside the range of most experience, is a very valuable tool for identifying erroneous unit values. However, caution should be exercised if high-value thresholds are set too low or low-value thresholds set too high so that many unit values within the range of experience are identified by the threshold test. In this case, the hydrographer always should apply other methods to verify unit values that have failed the threshold test.

An important consideration of unit value verification is the public display of unit values on World Wide Web pages, which is rapidly becoming more prevalent. For example, the USGS displays near real-time stage and discharge unit-value data from most of the continuous stream gauges it operates. The USGS uses thresholds that are designed to act as Web filters: data values outside the threshold limits are not displayed yet they remain in the database and thus preserved in the event that they are not erroneous.

Rating comparisons

The rating comparison identifies all unit values that exceed the high end or fall below the low end of the rating currently in use. This comparison can be performed automatically with the electronic processing system because ratings are stored in the electronic processing system. This test would alert the hydrographer to possible erroneous unit values as well as to the possible need to extend the rating currently in use.

Direct reading comparisons

Various types of direct readings may be available for comparison and verification of recorded unit values. These include actual gauge readings made by an observer or hydrographer, readings obtained from maximum and minimum indicators, high water mark readings and crest gauge readings. All of these various direct readings should be input to the electronic processing system and automatically displayed to the hydrographer in conjunction with the unit values being verified.

At some gauging stations auxiliary and/or backup gauges are operated in conjunction with the primary gauge. In many cases, the records from these gauges can be used as an independent check to the primary record.

Graphical comparisons

Graphics can be the most important and easily used method to verify a period of unit values. All of the methods described in the previous sections should be incorporated into a graphic system to automatically scan and review a period of record for the purpose of verification. The primary record of unit values should be plotted as a time series, with a unit-values scale that allows the hydrographer to see each value clearly and that does not distort the general shape of the record. The time scale should automatically default to the time zone normally used for the station, but there should be provision for the hydrographer to change to any other time zone. A basic plot of unit values can be used to identify erroneous data by an experienced hydrographer. With the addition to the plot of thresholds, rating limits, observer and hydrographer gauge readings, high water marks, maximum and minimum indicator readings and auxiliary gauge records, much more can be done to verify the primary record.

The recorded unit values should be plotted and considered the base plot. The processing system should plot all direct gauge readings by observers and hydrographers at the correct time on the base plot. High and low thresholds, high and low rating limits, high-water mark readings, maximum and minimum indicator readings and crest-stage gauge readings should be plotted at their respective elevations as a horizontal line that extends throughout the period of record being verified. This process will allow the hydrographer to compare these readings to peaks and troughs in the primary record. Auxiliary and backup records should be plotted as a time series for comparison to the primary record. The plotting system should use different colors and symbols to easily distinguish the various components. Unit values that trigger the difference threshold and the flat spot threshold also should be easily identified by color or symbol. When evaluating the potential for ice-affected periods, stage unit value plots that include air temperature, precipitation and water temperature values, if available, are very helpful in determining whether or not discharge computations should be adjusted for ice effects.

6.5.4 Parameter value corrections

The verification process described in the previous sections will sometimes identify unit values of gauge height or other parameters that are either erroneous or suspected of being erroneous. By definition, an erroneous gauge reading results when the recording instrument does not record the true parameter value that occurred in the stream, lake or other water body. A base, or reference gauge, usually is used for determining the true parameter value.

An erroneous gauge reading can result from either instrument errors, datum errors, or both. Instrument errors are those errors resulting from a malfunction, incorrect setting, incorrect calibration, or other problem with the recording instrument. An instrument error usually can be detected by comparing a recorded parameter value with a corresponding reference gauge reading. Datum errors, on the other hand, are those errors resulting from a change in the reference gauge and apply only to gauge heights or elevations. A datum error usually can be detected only by running levels to the reference gauge, using a stable benchmark of known elevation as a reference.

Another distinction between datum errors and instrument errors is that datum errors generally occur over many months or years, whereas instrument errors occur over a few days or weeks. Consequently, corrections for datum errors and instrument errors usually are made separately. However, correction for datum errors should use the same methods as those used for instrument errors as described below for instrument error corrections.

When a parameter value, or series of values, has been determined to be erroneous, it may be corrected, or edited, if the hydrographer has a sufficient basis for doing so. Editing of individual unit values should be allowed with the electronic processing system at any of the verification steps, including the graphical display. In the graphical display the hydrographer should edit unit values directly on the graph, or in a supplemental table of unit values. In addition to correcting and editing unit values, the electronic processing system also should allow the hydrographer to flag unit values in such a way that they will not be used in further analysis.

Datum adjustments and conversions

The gauge datum of a gauge site is usually an arbitrary datum, unique and specifically selected as a convenient working reference for each gauge site. The datum frequently is located at a level just below the lowest expected gauge height or just below the gauge height of zero flow. For some stations, such as at reservoirs and coastal streams, the gauge datum may not be arbitrary, but is established to be the same as mean sea level or other common datum. In any case, there are times when datum adjustments must be made to correct a datum error. Also, there are some stations for which it is necessary to convert an arbitrary datum to a known datum, such as mean sea level. These are described in the following sections.

Adjustments for gauge datum error

Gauge datum adjustments generally are considered to be corrections applied to recorded gauge heights and water-surface elevations to make them consistent with the gauge datum. Physical movement of a gauge or gauge structure can sometimes occur, causing an error of gauge readings in relation to the gauge datum. Such a change may be over a long period of time, such as from settling or subsidence, or the change may be sudden, such as from an earthquake, flood damage or accident. Whether the change is gradual or sudden, the result is the same, in that the gauge no longer records gauge heights and elevations that are correct in relation to the original gauge datum. Gauge movement, relative to gauge datum, is quantitatively measured by leveling from stable reference marks or benchmarks of known elevation. Leveling procedures for surface- water gauging stations are well established and are described by Kennedy (1990).

Datum errors should be carefully analyzed to determine the best method to make corrections. Frequently, it cannot be determined when a datum error occurred and the best method of correction is to prorate it uniformly throughout the period in question. If a specific time of occurrence can be defined then the correction can be made starting at that time and carrying the correction forward until the datum is restored. As a general rule, corrections for gauge-datum errors of 0.02 ft, or 0.006 mm, or less are not applied except in cases where smaller gauge-datum errors are critical in correctly defining another parameter, such as for reservoir contents computations. Small errors of this kind usually are absorbed by ratings and rating shifts.

Conversion to a common national datum

In addition to making datum adjustments for the purpose of correcting gauge-height values that are incorrect because of a change of the base gauge, it is sometimes necessary to convert recorded gauge heights to a different datum, such as a national datum. The most common conversion is where the recorded values must be converted to mean sea level. This type of conversion requires that a constant value be added to, or subtracted from, the recorded gauge heights throughout the record period. A gauge datum adjustment for gauge movement also may be needed at times. In such cases two simultaneous adjustments would be needed.

Instrument error corrections

Recording instruments and parameter sensors may, at times, produce erroneous gauge readings for a number of reasons. For example, float tapes may slip, recorders may punch incorrectly, gauge drawdown because of high velocity may occur, stage or velocity sensors may drift because of temperature and the recorder may even be set wrong by the hydrographer. These, and numerous other causes, will result in erroneous unit values of gauge height, velocity or other parameters.

The electronic processing system must provide easy and quick ways to make corrections when instrument errors are identified. Corrections should be possible through a graphical interface, such as the one described above for review and verification, and also with a tabular format. The hydrographer should be able to make corrections to individual unit values, or to sequences of unit values. Three types of corrections should be available for use:
(a) Constant value corrections;
(b) Parameter (usually stage) variable corrections;
(c) Time variable corrections.

Constant value corrections

Constant value corrections are simply the addition or subtraction of a constant value from a sequence of unit values. The hydrographer should be able to specify the constant value correction to be used and the dates and times for which the correction is to be applied. The electronic processing system then should apply the correction automatically.

Parameter variable corrections

Certain types of parameter errors may vary according to the value of the parameter. For instance, for some gauging stations the stage measurements may not reflect actual river stage because of drawdown caused by high flow velocity near the gauge intake. The resulting stage error is directly related to the velocity, which in turn is often related to the stage. A relation between stage and stage-correction can sometimes be defined that is reasonably consistent for long time periods and can be used to determine the gauge-height correction on the basis of the recorded stage.

Parameter variable corrections require a relation between the parameter and the correction. The hydrographer should be able to input this relation to the electronic processing system, along with a starting date and time, and if needed an ending date and time. The electronic processing system should calculate and apply the corrections automatically. When a correction relation of this type is entered, and no ending date and time are specified, then it should be continued in use until such time that an ending date and time are specified.

A parameter variable correction relation should be defined by entering point pairs of parameter and corresponding corrections for as many points as necessary through the intended range of correction. The processing system should automatically interpolate corrections that are needed between the input points. If parameter values occur below the lowest point of the correction relation, then the correction value for the lowest point of the relation should be used for all corrections below this point. Likewise, the correction values above the highest point of the correction relation should be the same as the highest correction value of the relation. Alternatively, the correction relation can be entered as an equation. Upper and lower limits of the input parameter should be specified for the equation. The correction values corresponding to these limits should be held constant when parameter values are less than the lower limit or greater than the upper limit.

Time variable corrections

Time variable corrections are corrections that are distributed between specified dates and times. This type of correction usually is referred to as time proration. Time proration should apply to singular correction values and to parameter variable correction relations. Likewise, time variable corrections should apply to datum corrections as well as instrument error corrections.

Corrections that do not vary with parameter value are considered a singular correction for a given point in time. However, such a correction may vary with time. For example, at the beginning of a time series of unit values, a correction of + 0.15 ft is defined, which does not vary with stage. At a subsequent date and time, a correction of + 0.10 ft is defined, which likewise does not vary with stage. The electronic processing system should allow the hydrographer to make a linear, time proration between these two correction values and defined times.

Corrections that vary with parameter value (as defined by a parameter variable correction relation) sometimes gradually may change shape or position with time. The electronic processing system should allow time proration between two consecutive parameter variable correction relations. Time proration between two correction relations should be made on the basis of equal parameter values. For example, assume that a correction relation is entered with a date and time. A second correction relation is entered with a subsequent date and time. At some intermediate date and time, assume that the gauge height is 4.23 ft. Correction values are determined from each of the two correction relations for a gauge height of 4.23 ft, resulting in two correction values, one at the start of the proration period and one at the end of the proration period. The correction that applies to the intermediate date and time, for the gauge height of 4.23 ft., is determined by time interpolation between the two correction values.

Additive corrections

Sometimes, more than one correction for the same period of unit values may be needed. For instance, a datum correction may be needed during the same period of time that a parameter variable correction relation is needed. If both corrections are defined, and the dates and times overlap, the electronic processing system automatically should apply both corrections simultaneously for the overlapping period. In other words, all corrections that are defined for the same date and time, or for the same type of correction, become additive. There should be no limit as to the number of corrections that can be used for a given date and time, but it is not likely that more than two or three would be required.

Flagging of unit values

Corrections cannot always be determined for unit values, and in fact, corrections are not always desired for unit values. For certain situations it is recommended that daily values be estimated rather than attempting to correct, or estimate, unit values. In these situations, the hydrographer should be able to flag specific unit values to specify the reason they are not used. The flags also will be an indicator in other parts of the electronic processing system, such as the primary computations, to ignore the unit values for certain kinds of computations. The following flags are recommended:

(a) Affected – This flag is for unit values that are correct and representative of the true stage (or other parameter), but because of some irregular condition the rating is severely affected and may not be applicable. This flag should be used for severe conditions of backwater from irregular downstream conditions, backwater from ice and other conditions. The flag should not be used for normal shifting control conditions;
(b) Erroneous – This flag is for incorrect unit values. For instance, the float is resting on mud in the stilling well, and the recorded unit values do not represent the stage in the stream;
(c) Missing – This flag is reserved for situations where unit values were expected, but because of some malfunction of equipment where no data were recorded;

(d) Estimated – This flag is used for estimated unit values. It should be automatically attached to unit values that are changed by the hydrographer.

The first three types of flags defined above are intended primarily for the original, archivable unit values. These flags will document, for historical purposes, the evaluation and interpretation of the validity of the recorded unit values. They also should be carried forward for the analysis and computation of records. In the analysis and computations, it may be desirable to estimate unit values in certain situations. The fourth type of flag is reserved for estimated values, which may replace affected, erroneous or missing data. The estimated flag only will be used for unit values in data sets generated subsequent to the original data set. Unit values flagged as affected or erroneous should not be used in the primary computations.

6.6 VERIFICATION AND ANALYSIS OF FIELD MEASUREMENTS

Field measurement data and information that are entered into the electronic processing system include discharge measurements, gauge datum leveling measurements, crest-stage gauge data, channel and control cross-section data and miscellaneous field notes. All of these data usually are entered by keyboard, except that some discharge measurements are entered from electronic field computers. Various computations and comparisons should be made to verify the accuracy and insure the consistency of the information. The following sections describe some of the verification, computations and cross checking that should be performed with the electronic processing system. Errors resulting from data entry and incorrect computation should be corrected by the hydrographer.

It is important to emphasize that measurement data (that is, depth, width and velocity data) should not be deleted or erased from the original notes, which in most cases are the paper note sheets. Editing of data that are entered from paper notes to the electronic processing system is permitted, provided the data were entered by keyboard. This editing allows for correction of keyboard entry errors without compromising the integrity of the original paper notes. On the other hand, data entered electronically, such as from an electronic field computer, should not be edited, changed or deleted because once they are entered to the electronic processing system they become the original copy which will be used for archiving. It is assumed that no errors occur during an electronic transfer. All information in measurement notes (for example, computed values such as area, velocity, width, discharge and others) may be edited and changed regardless of the entry method. These values should be arithmetically correct and based on the original data.

6.6.1 Discharge measurement checking

All discharge measurements should be checked wherever possible for arithmetic errors, logic errors and other inconsistencies with the electronic processing system. In addition, the electronic processing system should compute the standard error for regular current meter measurements. If a rating is available for the gauging station, the electronic processing system should compute the shift of the measurement from the rating. The shift analysis would apply to stage-discharge, slope and rate-of-change in stage and ratings. Most of the following checking and computation steps apply only to standard current meter measurements.

Arithmetic checking

A summary of the numerical results of a discharge measurement is entered to the electronic processing system from what usually is referred to as the front sheet of the measurement. Most of the information on the front sheet are computed from the field measurement data that are part of the inside body of the measurement. For discharge measurements recorded on paper forms, the computations are made by the hydrographer in the field with a calculator. If an electronic field notebook was used for recording the discharge measurement data, then the computations were made automatically by the field notebook and little or no arithmetic checking is required.

When original computations are made on paper forms, the following checks of the inside part of the measurement should be made with the electronic processing system:

(a) Subsection width – The width for a subsection is computed as one-half the distance between the preceding vertical stationing and the succeeding vertical stationing. For verticals at the edge of a channel or bridge pier, the subsection width is computed as one-half the distance to the adjacent vertical;
(b) Point velocities – If a current meter rating or equation has been entered for the current meter used in making the discharge measurement, then each point velocity should be checked;

(c) Mean velocity for each vertical – The mean velocity for each vertical is computed as follows:
   (i) For the one-point method, the mean velocity is equal to the point velocity at the 0.6 depth. If the point velocity was measured at a depth other than the 0.6 depth, then the mean velocity for the vertical is computed by multiplying the point velocity by the method coefficient. If a method coefficient has not been entered for the vertical, then the electronic processing system should warn the hydrographer and provide an opportunity to enter a method coefficient. The hydrographer can choose to ignore the warning;
   (ii) For the two-point method, the mean velocity is equal to a mean of the point velocities for the 0.2 and 0.8 depths;
   (iii) For the three-point method, the mean velocity is equal to a weighted mean of the 0.2 depth velocity, the 0.6 depth velocity and the 0.8 depth velocity, where the 0.6 depth velocity is given double weight;
(d) Subsection mean velocity – The mean velocity for each subsection is computed as the product of the mean velocity of the vertical and the horizontal angle coefficient. If a horizontal angle coefficient is not entered for the vertical, then the electronic processing system should assume a value of 1.00;
(e) Subsection area – The area for each subsection is computed as the product of the subsection width and the depth at the vertical;
(f) Subsection discharge – The discharge for each subsection is computed as the product of the subsection area and the subsection mean velocity;
(g) Total width – The total width for each channel is computed by summing the subsection widths;
(h) Total area – The total area for each channel is computed by summing the subsection areas;
(i) Total discharge – The total discharge for each channel is computed by summing the subsection discharges;
(j) Total number of verticals – The total number of verticals for a measurement is simply a count of the number of verticals, and includes the beginning and ending points where depth often is equal to zero;
(k) Average velocity – The average velocity for each channel is computed by dividing the total discharge by the total area;
(l) Totals for multiple channels – When the discharge measurement has two or more channels, such as for a braided stream, or a flood measurement that has a main channel and one or more overflow channels, the grand total of width, area, discharge and number of verticals is computed. These grand totals are the values used to summarize the discharge measurement on the front sheet. The average velocity for the measurement is the grand total of discharge divided by the grand total of area.

Logic and consistency checking

Information entered to the electronic processing system from one part of the discharge measurement notes should be automatically compared and cross checked with information from other parts of the measurement to verify that it is logical and consistent. The electronic processing system should alert the hydrographer when inconsistencies occur and provide an opportunity to make a change. In addition, when specific information items are entered, the electronic processing system then should limit the entry of other items so that the choices are consistent. For instance, if the type of measurement is entered as a wading measurement then the choices for equipment entry would be limited to the various types of wading rods. A listing of some of the possible logic and consistency checks are given below:

(a) Compare measurement sequence number with measurement date and time – Measurement numbers generally are in sequential order according to date and time;
(b) Compare measurement mean gauge height(s) to gauge readings – The mean gauge height should be a value that falls between the lowest and highest gauge readings recorded during the course of making the discharge measurement;
(c) Compare gauge-height change to gauge readings – The gauge-height change should be the difference between the gauge heights at the start and end of the discharge measurement;
(d) Compare gauge-height change time to start and end time – The gauge-height change time should be the difference between the start and end time of the discharge measurement;
(e) Compare stream width on summary input to stream width for inside note input – The stream width on the summary input should be exactly the same as the stream width computed and entered for the inside note input. For multiple channels the stream width should be the sum of individual channel widths;
(f) Compare stream area on summary input to stream area for inside note input – The stream area on the summary input should be exactly the same as the area computed and entered for the inside note input. For multiple channels

the stream area should be the sum of the individual channel areas;

(g) Check mean velocity – The mean velocity should be the checked by dividing the measured discharge by the stream area;

(h) Compare number of sections on summary input to number of sections for inside note input – The number of sections should be the total number of verticals used for making the discharge measurement. This total includes each end section of the measurement, even though depth and velocity at these points may be zero. For multiple channels, the number of sections should be the sum of the sections for individual channels;

(i) Check adjusted discharge – If an adjusted discharge is entered, the electronic processing system should compute an adjusted discharge based on the adjustment method, if stated. This computed value should be compared to the entered value;

(j) Check average time of point velocities – The average time of point velocities on the summary input should agree with the average of the time of current-meter revolutions entered for the inside note input;

(k) Compare gauge height of zero flow to gauge readings – The gauge height of zero flow should be less than the mean gauge height of the discharge measurement, and less than the gauge heights in the gauge-height table, except in the cases of a zero flow measurement.

6.6.2 Special checking procedures for other types of discharge measurements

Some discharge measurements are made under conditions that require computational procedures that are different than the standard open-water, current-meter discharge measurement described in preceding sections. In some cases the differences are minor but in other cases the measurement method is completely different. Also, some measurement methods use highly specialized equipment and recording methods that differ entirely from those of standard discharge measurements. The following sections describe some of the verification, editing and computations that should be performed with the electronic processing system for each of the various types of measurements.

Ice measurements

Ice measurements, in most respects, are the same as standard open-water discharge measurements. All of the same arithmetic checking, logic and consistency checking and shift analysis should be performed on ice measurements. Differences between computations for a standard discharge measurement and an ice measurement are listed below:

(a) Computation of effective depth – The inside body of the discharge measurement notes for ice measurements contain two additional columns of data and information. One of the extra data columns is a field measurement of the vertical distance between the free water surface and the bottom of the ice (solid or slush ice). These measurements should be compared to the total depth for each vertical, and if in any given vertical the depth from the water surface to the bottom of the ice is found to be greater than the total depth, a warning message should be issued by the electronic processing system to the hydrographer. The second additional column is effective depth, d_e, for each vertical and is computed as the difference between the total depth, D, and the vertical distance, d_i, between the free water surface and the bottom of the ice. The equation is:

$$d_e = D - d_i \tag{6.1}$$

(b) Computation of subsection area – The area of each subsection is computed by multiplying the subsection width times the effective depth, d_e, of the vertical;

(c) Velocity coefficient – For verticals where the 0.6 depth method is used to observe velocity, it is frequently necessary to apply a velocity coefficient to correct for the ice effect on the vertical velocity distribution. This velocity coefficient is similar to the use of a method coefficient for computing the mean velocity in a vertical, as described in a previous section on arithmetic checking. The mean velocity in the vertical is computed by multiplying the velocity coefficient times the point velocity observed at the 0.6 depth. If a velocity coefficient is not given, then it should default to 1.00. If the two-point method (0.2 depth/0.8 depth) is used to observe velocity, then no velocity coefficient is necessary;

(d) Shift computations – Shifts are not usually computed for ice measurements, but in some cases may be desired. The hydrographer should have the option to specify if shifts should be computed, and if so, they should be computed just as they are for a regular open-water measurement;

(e) Percent difference from rating curve – The difference, in percent, between the measured discharge and the rating curve should be computed for all ice measurements, based on the same method as described for standard discharge measurements;

(f) Discharge ratio – For some gauging stations, the discharge-ratio method is used for computing ice records. The hydrographer should have the option to specify the computation of the ratio if it is used. The electronic processing system then should compute the ratio, K_i, for each ice measurement as the ratio of the measured discharge, Q_m, to the open-water rating discharge, Q_r, that corresponds to the mean gauge height of the measurement as:

$$K_i = Q_m / Q_r \tag{6.2}$$

Measurements with vertical angles

Depth measurements of deep, swift streams that are made with cable suspension equipment from bridges, cableways and boats cannot always be made directly. Frequently, the sounding weight is carried downstream by the current and consequently the observed depth is greater than the true vertical depth. In such cases corrections must be made to the observed depth in the field at the time the measurement is made. The body of the field notes for these measurements contains additional columns for recording air-line vertical distance, observed depth, vertical angle and computed vertical depth. The corrections, which usually are not recorded in the field notes, account for an air-line correction and a wet-line correction of the sounding cable. In some cases, such as when sounding line tags are used, the air-line correction may be eliminated or reduced to a negligible amount.

The electronic processing system should contain the air-line correction table and the wet-line table so that the computed vertical depth can be checked. These tables are given in Chapter 5 of Volume I, which also provides a detailed description of the computation methods. A brief summary of the procedure is listed below:

(a) Determine the air-line correction based on the observed air-line vertical distance between the sounding equipment and the water surface, the observed vertical angle and the air-line correction table;
(b) Subtract the air-line correction from the uncorrected observed depth of water. This subtraction must be made before determining the wet-line correction;
(c) Determine the wet-line correction based on the air-line corrected observed depth, the observed vertical angle and the table of wet-line corrections;
(d) Compute the true vertical depth by subtracting the wet-line correction from the air-line corrected observed depth;
(e) Air-line and wet-line corrections should be interpolated from their respective tables to the nearest tenth of a foot.

All other computations and checking are essentially the same for measurements with vertical angles as they are for standard discharge measurements, including the computation of measurement standard error.

Acoustic Doppler Current Profiler moving boat measurements

The Acoustic Doppler Current Profiler (ADCP) is used to define the velocity profile in a stream vertical, as well as depth measurements across the stream. The velocity profile provides a much more accurate measure of the mean stream velocity than other techniques where only one or two measuring points are used, and sometimes adjusted by velocity coefficients. The ADCP moving boat method of measurement has replaced other moving boat methods and has provided a fast, accurate type of discharge measurement for wide and deep streams. This type of measurement is fully computerized with all data collected and computed automatically. Data and information from the ADCP measurement should be transferred to the electronic processing system through an interface. These data and information become the archivable record. Summary information for the measurement is much the same as for a regular discharge measurement.

Special considerations for checking ADCP measurements (Oberg and others, 2005) are as follows:

(a) The discharge-measurement note sheets should be complete, clear and legible;
(b) All electronic data files associated with the measurement should be backed up in the field and archived on an office server;
(c) The number of transects collected should be appropriate for the flow conditions. Transects should be collected in reciprocal pairs;
(d) Configuration files should be checked for errors, appropriateness for the hydrologic conditions and for consistency with field notes. ADCP depth, salinity, edge distances, edge shapes, extrapolation methods and ADCP configuration parameters shown on the field notes should match those in the configuration file;
(e) A moving-bed test should be performed prior to the discharge measurement, recorded, archived and noted on the ADCP measurement note sheets. If a moving bed was detected,

Differential Global Positioning System (DGPS) should be utilized. If DGPS was not used, the measured discharges should be adjusted for the moving bed and the measurement quality should be downgraded;

(f) The average boat speed for the measurement should not exceed the average water speed unless it was impractical or unsafe to do so, and the reason documented in the field notes or station file. Boat pitch-and-roll should not be excessive. Excessive boat speed or pitch-and-roll may justify downgrading the measurement quality;

(g) The measured edge distances recorded on the ADCP measurement note sheet should match those electronically logged with each transect. The correct edge shape should be selected and 5-10 seconds of data collected at transect stop/start points while the boat was held stationary. If sub-sectioning was used to correct problems with edges, then the sub-sectioning should be clearly documented on the note sheets. If a vertical wall(s) was present, then the start and/or end points of the transect should be located such that the distance from the wall(s) is equivalent to the water depth at the wall or greater;

(h) There should not be excessive loss of profiles. The loss of more than 10 per cent of profiles in one or more transects may necessitate downgrading the measurement quality, especially if the missing data are concentrated in one part of the measured cross section. When the missing profiles always occur in the same part of the cross section, the measurement quality should be downgraded, even when less than 10 per cent of the profiles are missing;

(i) When more than 25 per cent of the depth cells in one or more transects are marked invalid or missing, the quality of the measurement may need to be downgraded. This downgrading is not necessary, however, if the distribution of the missing depth cells is more or less uniform throughout the water column and/or the cross section measured;

(j) The extrapolation method for the top and bottom discharges should be reviewed. If review of the data shows the need for a different extrapolation method than that chosen for use in the field, the extrapolation method should be corrected and the reasons documented on or attached to the measurement note sheet. Wind and horizontally stratified density currents are common causes for profiles that do not fit well by means of the 1/6th power-law extrapolation method. In these situations it is usually necessary to use different extrapolation techniques for the top and bottom areas and/or limit the portion of the profile used for the selected method;

(k) Measurement computations, including mean discharge and measurement gage height, must be correct.

Indirect measurements

Indirect discharge measurements include slope area, contracted opening, critical depth, culvert, step backwater and flow over dams and embankments. These types of measurements are almost always made after a flood event rather than during the flood. Data collection, recording of field notes and computation procedures are appreciably different than standard measurements made during a flood event. For most indirect measurements, computer programs are used for the computations and detailed reports are prepared. Entry of information from indirect measurements to the electronic processing system should include only the summary information. The same entry form can be used as for a standard discharge measurement.

Portable weir and flume measurements

Measurements of low discharge can be made using a portable weir or flume. Various types of weirs and flumes are available for these measurements and usually are rated in the laboratory so that coefficients and discharge ratings are defined for each. Field setup and measuring methods are described in Chapter 7 of Volume I. After the weir or flume is installed and a sufficient period of time is allowed for streamflow to stabilize, a series of upstream head measurements are taken for a period of about three minutes. The average of these head measurements is used to determine the discharge, either from a rating table (flume measurement) or from an equation (weir measurement). Downstream head measurements usually are not taken because the flume or weir is installed so free fall or minimum backwater conditions exist.

Entry of the inside body of the discharge measurement is relatively simple and includes only the weir or flume head data and the determined discharge. Some hydrographers enter this information on the front sheet of the measurement, rather than in the inside body. Regardless of where these notes are recorded, the electronic processing system should provide a form for entering the basic data and computations and should check the computations. The data and information required are as follows:

(a) Head measurements – These are the individual observations of head. The recommended number of observations is about seven, one observation every 30 seconds for a period of three minutes. However, this number can vary and in some cases only one observation will be recorded. The electronic processing system should allow for at least 10 entries;

(b) Average head, h – This is an unweighted average of the individual head observations. The electronic processing system should calculate the average head, h, and compare it to the entered value. If the two values are different, a message to this effect should be noted. The hydrographer should select the average head value for use in computing discharge;

(c) Discharge, Q – The discharge should be calculated, either from a rating table or from an equation. Rating tables and/or equations for standard weirs and flumes should be included in the electronic processing system. However, if they are not directly available, the hydrographer should enter one.

Entry of front-sheet information for weir and flume measurements greatly is abbreviated from that of a standard discharge measurement.

Tracer-dilution measurements

Tracer-dilution discharge measurements are highly specialized techniques that utilize one of a number of different tracers, different types of measurement equipment and different measurement methods. Data collection, recording and calculation of measurement information vary depending on the method and tracer used. Details of each type of tracer-dilution measurement are described in Chapter 8, Volume I. The methods are standardized so details of tracer-dilution measurements do not need to be entered in the electronic processing system. Only summary information is required.

Volumetric measurements

Low flows sometimes are measured by diverting the flow into a calibrated container, and measuring the time required to fill, or partially fill, the container, as described in Chapter 8, Volume I. If the container is filled completely, the flow volume equals the container volume. If the container is partially filled, the flow volume equals the difference of the ending and starting volume. This procedure usually is repeated three to four times to improve accuracy of the measurement. The discharge is computed by dividing the total volume (sum of the volume measurements from each repetitive run) by the total time of diversion (sum of the time measurements from each repetitive run), in seconds.

Data entry from the inside field notes to the electronic processing system include the following:

(a) Total container volume;

(b) Starting volume for each repetitive run – This value should be equal to or greater than zero, but less than the total container volume;

(c) Ending volume for each repetitive run – This value should be greater than the starting volume, and equal to or less than the total container volume;

(d) Flow volume for each repetitive run – This is the difference between the ending volume and the starting volume, and must be equal to or less than the total container volume;

(e) Fill time for each repetitive run;

(f) Total volume – This is a summation of the individual flow volumes of each run;

(g) Total time – This is a summation of the individual fill times of each run;

(h) Discharge – This is the total volume divided by the total time.

The electronic processing system should make the checks and computations indicated above and report any discrepancies.

The procedure described above is used where the total flow can be easily collected in a container. In some cases, such as at a broad-crested weir or dam, the depth may be too shallow to measure using conventional methods, but volumetric measurements may be applicable to small segments of the flow. This is the volumetric-incremental sampling method. In this method, volumetric flow measurements are made as described in the preceding paragraphs at five to 10 subsections along the weir or dam. The flow rate of each sample is increased by the ratio of the subsection width to the sampled width to obtain a flow rate for each subsection. The total flow of the stream is the summation of the discharge rates of each subsection. The electronic processing system should perform these computations from the input data and report any discrepancies. Front sheet information is an abbreviated version of the standard discharge measurement.

Discharge estimates

Low flows sometimes are estimated when no suitable measuring method is applicable. Various techniques for estimating the flow are used which should be described in the field notes. It is not recommended that the details of making the estimate be entered into the electronic processing system, because they generally cannot be checked or verified, and the paper notes are considered the original archivable record. A summary of the measurement can be entered using the standard discharge measurement entry form, but abbreviated considerably to accommodate only the pertinent information.

6.6.3 Rounding and significant figures

All field data for discharge measurements should be entered to the electronic processing system with the same precision and significant figures as recorded in the field notes. Table look-up values and calculated values should be rounded to standard significant figures, unless specified otherwise by the hydrographer. Exceptions to the standard significant figures are required for calculations of the subsection values of width, area and discharge in the inside body of the field notes, as follows:

(a) Subsection width – The width of each subsection should be used and displayed as an unrounded value;
(b) Subsection area – Each subsection area should be rounded and displayed with one additional significant figure from that of the expected total area. For instance, if the total area is expected to be between 10.0 and 99.9 ft^2, the individual subsection areas should be rounded and displayed to hundredths of a square foot;
(c) Subsection discharge – Each subsection discharge should be rounded and displayed with one additional significant figure over that of the expected total discharge, similar to that described above for subsection area. For instance, if the total discharge is expected to be between 100 and 999 ft^3/s, then each subsection discharge should be rounded and displayed to the nearest 0.1 ft^3/s.

All summary information for discharge measurements should be rounded and displayed with standard significant figures unless specified otherwise by the hydrographer.

6.6.4 Summary of discharge measurements

Discharge measurement information and data from all types of discharge measurements should be summarized in chronological order and grouped by water year to provide a history of the measurements. In addition to the summary of discharge measurements, an output format that includes all of the data and information entered for each measurement should be available to the hydrographer. The hydrographer also should be able to define a custom output format that only would include selected items.

6.7 ENTRY OF RATING CURVES TO THE ELECTRONIC PROCESSING SYSTEM

Rating curves are relations between dependent and independent variables, and the development of rating curves has been described in previous sections of both Volumes I and II. Rating curves are an integral part of the computation of most streamflow records and should be made a part of the permanent records for each station. However, the electronic processing system also should allow the entry, development and display of rating curves independent of computing streamflow records for specific gauging stations. That is, the hydrographer should use the rating curve aspects of the electronic processing system for entering, editing, developing, refining and experimenting with the rating curves.

Rating curve information required for defining the relation between the independent and dependent variables, such as gauge heights and discharges, can be entered into the electronic processing system using various methods, including tabular, equation and graphical methods. Tabular entry is the use of a table of descriptor data pairs, each representing a specific location of the rating curve. Equation entry is the use of a mathematical expression to define the rating curve. Graphical entry is a method whereby a series of points are entered directly on a rating curve plot displayed on the computer monitor. The electronic processing system automatically evaluates the points and connects them to display the rating curve.

Tabular entry and graphical entry are similar in that both utilize hydrographer-defined descriptor points. The primary difference is that tabular entry is based on descriptor points that are hand picked from a paper rating curve plot, whereas graphical entry is based on descriptor points defined on the computer monitor, thus, negating the need for a paper plot.

6.7.1 Tabular entry

Rating curves may be entered to the electronic processing system by keyboard as a series of descriptor points, sometimes referred to as point pairs. Each point pair contains the independent variable and the corresponding dependent variable for one position on the rating curve. The electronic processing system should not limit the number of point pairs that can be entered. Point pairs always should be entered in ascending order of the independent variable, starting with the lowest point on the rating curve. If the hydrographer incorrectly enters a point pair in which the independent variable is not ascending the electronic processing system immediately should issue a warning message to alert the hydrographer that an entry error was made. This checking method also should be used for the dependent variable for those ratings where the dependent variable is not allowed to decrease. A similar warning message also should be given if negative values are entered for ratings where they are not allowed.

Rating curves that are entered as linear scale ratings will require only the table of point pairs. No other descriptive information is needed for either plotting or expanding a linear scale rating.

Rating curves entered as logarithmic scale ratings will require entry of scale offset information in addition to the table of point pairs. A scale offset is a value that is subtracted from the independent variable before interpolating between point pairs of the rating. It is important that the scale offset entered at this point is the same as the one used for the plotted rating curve. If the hydrographer does not enter a scale offset for a logarithmic rating, the electronic processing system should not accept the rating and should prompt the hydrographer that an offset is required.

The electronic processing system should allow one, two or three scale offset values for each logarithmic rating curve, with each respective offset applicable to a designated range of the rating. The offsets should be entered starting with the lowest rating curve segment and progressing upward, with a defined breakpoint between successive offsets. The breakpoint is the value (usually gauge height) of the independent variable above which the succeeding offset should be used. The following combinations of offsets and breakpoints are allowable:

(a) One offset, no breakpoints – In this case, a single offset is used throughout the range of the rating;
(b) Two offsets, one breakpoint – In this case, the first offset is used for all values of the independent variable that are less than or equal to the breakpoint value. The second offset is used for all values of the independent variable that are equal to or greater than the breakpoint value;
(c) Three offsets, two breakpoints – In this case, the first offset is used for all values of the independent variable that are less than or equal to the first breakpoint value. The second offset is used for all values of the independent variable that are equal to or between the first and the second breakpoints. The third offset is used for all values of the independent variable that are equal to or greater than the second breakpoint value.

A point pair entry in the table of point pairs is required at each breakpoint of the rating. If the hydrographer omits the point pair corresponding to a breakpoint, the electronic processing system should issue a warning message and should not accept the rating unless this requirement is met. The point pair at each breakpoint is used as the ending point for the rating-curve segment below the breakpoint, and the beginning point for the rating-curve segment above the breakpoint. This process insures continuity of the rating-curve segments.

6.7.2 Equation entry

Some ratings may be easily expressed in equation form, and if so, they may be entered to the electronic processing system as a mathematical expression. Such ratings usually are of simple form, consisting of a smooth curve or straight line, with no unusual shapes or sharp bends. For all equation ratings, a basic format as given in equation 6.3 should be used:

$$Y = a + b(X - e)^c \tag{6.3}$$

where Y = dependent variable (usually discharge); X = independent variable (usually gauge height); a = equation constant (default value is zero); b = multiplier (default value is 1); e = scale offset (default value is zero) and c = exponent (default is 1).

Equation 6.3 can be used for rating curves interpolated either linearly or logarithmically. Other types of equations are not recommended for surface-water rating curves.

Upper and lower equation limits also should be required as part of the input for equation ratings. These limits should, by default, be in terms of the independent variable. However, the hydrographer should have the option to specify the limits in terms of the dependent variable. When extrapolation of equation ratings is needed, and can be justified, a modification of the approved limits should be allowed. The electronic processing system automatically should not extrapolate the equation beyond the approved specified limits.

The electronic processing system should allow up to three equations for the definition of a rating curve. Breakpoints, in terms of the independent variable, between two consecutive equations are required to define the exact point of the ending of one equation and the beginning of the next equation. Consecutive equations must intersect at the given breakpoint. The electronic processing system should calculate the dependent variable at the breakpoint by using each equation, and if the two calculated values of the dependent variable are not identical the electronic processing system should alert the hydrographer and not accept the equations until appropriate changes are made. These checks and modifications should be made at the time of equation entry and before application of the equations.

When multiple equations are used to define a rating curve, a lower limit should be specified for the lower equation, and an upper limit should be specified for the upper equation. The same rules and guidelines apply to these limits as stated previously for single equation limits.

6.7.3 Graphical entry

Graphical input of rating curves is presently the most automated and preferred method of entering a rating curve to the electronic processing system. Historically, rating curves have been drawn manually and descriptor points read from the plot. The electronic processing system should provide a method whereby the hydrographer can automatically plot selected discharge measurements and other rating curve information on the computer monitor, and then fit a rating curve to the plotted points directly on the monitor. The fitting process will be done by specifying a series of descriptor points, either directly on the computer monitor or in a table displayed on the monitor. After the hydrographer is satisfied with the accuracy and smoothness of the rating curve, the electronic processing system should automatically transform the plotted rating curve into a rating table.

6.8 RATING TABLES

The rating table is primarily for the purpose of displaying values of the dependent variable for the complete range of the independent variable. Rating tables should be generated with the electronic processing system for all rating curves. The tables are populated by interpolating values of the dependent variable for the complete range of the independent variable, at intervals equal to the stated precision of the independent variable or other user-defined interval. For instance, if the independent variable is gauge height, and its stated precision is hundredths of a foot, then values of the discharge would be computed for every hundredth of a foot for the range of gauge height defined by the limits of the rating.

6.8.1 Interpolation methods

The method used to interpolate between rating input points should be based on the method used to develop the rating. Rating curves defined as linear scale ratings should be interpolated between input points using simple linear interpolation.

Rating curves defined as logarithmic scale ratings should be interpolated between log-transformed input points using linear interpolation. The applicable scale offset must be subtracted from all input values of the independent variable before making the logarithmic transformations. If the rating is defined with two or three scale offsets, then each offset should be applied within the range defined by the respective breakpoints.

It is very important that the interpolation process use the same offset(s) that are used for the development of the rating curve plot so that the resulting rating table precisely duplicates the plotted curve. If the rating is plotted on the electronic processing system monitor, the rating curve automatically is converted to a rating table, and the offset will automatically be the same for both the plotted curve and the resulting table. If the rating curve is entered as a table of descriptor points, then the interpolation method must use the scale offset(s) entered with the descriptor points. The hydrographer is responsible for insuring that the offsets are identical.

Note that the subtraction of the scale offset from the independent variable is made only for the purpose of transformation and interpolation. The subtraction should not alter the original values of the independent variable that are displayed in the rating table or plotted on rating curve plots.

The dependent variable (discharge) for many rating curves has a minimum value of zero, which cannot be transformed to a logarithm. A simple linear interpolation between the zero point and the next larger input value of the dependent variable should be used for logarithmic ratings beginning with zero. To avoid appreciable distortion of the low end of the rating it is recommended that the input value of the dependent variable that follows the zero input value be equal to or less than 0.1. The electronic processing system should issue a warning message to the hydrographer if 0.1 is exceeded and provide an opportunity to make changes.

The independent variable (gauge height) can sometimes be zero or negative at the low end of a rating curve. This value is permissible only when subtraction of the scale offset from the independent variable results in a positive number.

Rating curves defined by one or more equations also should be transformed into rating tables. This is a simple method of computing the dependent variable for the entire range of each equation, as defined by the breakpoints and input limits.

6.8.2 **Rating table smoothness analysis**

One method of analyzing the smoothness of a rating curve and/or rating table can be done by studying the differences between successive values of the dependent variable. To make this task easy for the hydrographer, the rating table should display the computed differences (traditionally referred to as first differences) of the dependent variable between every tenth value of the independent variable displayed in the rating table. For instance, if gauge height is incremented every 0.01 ft in the rating table, then the difference between discharges corresponding to gauge heights at 0.1 ft intervals should be computed and displayed. For metric rating tables the differences should be displayed for every 0.01 m.

6.8.3 **Other rating table information**

The rating table should include descriptive information that identifies the gauging station, type of rating, period of use and other items that are unique for that rating. An example of an expanded rating table (English units) for a logarithmic stage-discharge rating curve is shown in Figure II.6.2. This sample rating table illustrates the header information and a typical arrangement of table information.

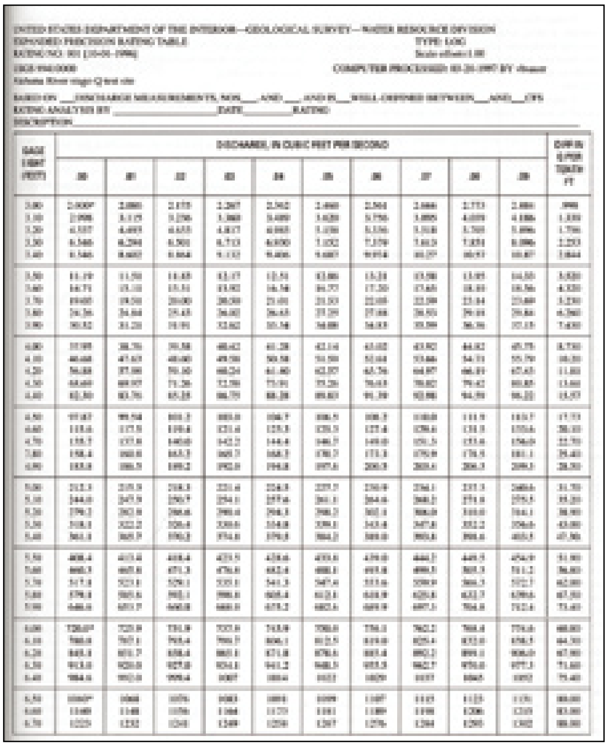

Figure II.6.2. Example of expanded rating table, in English units

6.9 **RATING CURVE PLOTS**

The electronic processing system should plot the rating curves on the computer monitor with interaction by the hydrographer to manipulate, draw and define ratings electronically. The requirements for monitor plots are essentially the same as for paper plots as described in Chapter 1, Volume II of this Manual. These monitor plots should be a highly flexible part of the electronic processing system and also should provide the capability to produce a paper plot of the same rating, if required. The electronic processing system should develop and print the entire plotting form for a paper plot. It should print the grid as well as the rating curve and other rating curve information.

6.9.1 **Linear scale plots**

An arithmetically divided, linear, plotting scale is the simplest type of rating curve plot. Linear scale plots are convenient to use and easy to read. Zero values can be plotted on the arithmetic scale, whereas these values cannot be plotted on logarithmic scales. For this reason linear scale plots frequently are used for analyzing the low end of stage-discharge ratings. However, for detailed hydraulic analysis linear scale plots have little or no advantage over logarithmic scale plots. A stage-discharge relation plotted to a linear scale is almost always a curved line, concave downward, which can be difficult to shape correctly if only a few discharge measurements are available. Logarithmic scale plots, on the other hand, have a number of analytical advantages as described in Chapter 1, Volume II of the Manual.

Linear scale plots are excellent for displaying a rating curve. Usually, a rating curve is first drawn on a logarithmic scale plot for shaping and analysis and then transferred to a linear scale plot for display. The electronic processing system should make this process simple and easy.

Linear scale subdivisions should be established to cover the complete range of the independent and dependent variables, or a selected range. If only part of the rating is to be plotted, the hydrographer should specify the range of either the independent variable or the dependent variable for the desired plot. The electronic processing system should make an initial determination of scales, subdivided in uniform increments that are easy to read and interpolate. The scales also should be chosen so that the plotted rating curve is neither very steep nor very flat. Usually, the curve should be a slope between 30 and 50 degrees. The hydrographer should be able to change the scales easily and quickly so that various plots can be viewed. The electronic processing system should replot all measurements and rating curve information each time a scale change is made.

If the range of the variables is large, it may be necessary to break the plotting scale and plot the rating curve in two or more segments to provide scales that are easily read with the necessary precision. This method may result in separate curves for low water, medium water and high water. Although two or three separate curves are plotted, they should be plotted within the same plotting form, if possible. The electronic processing system should arrange the individual plots on the form so that they are separate and distinct, properly scaled and not overlapping. Optionally, the separate curves could be plotted on separate forms.

6.9.2 **Logarithmic scale plots**

Many rating curves can be analyzed best by plotting the rating on logarithmic scale plots. Hydraulic characteristics that are evident in logarithmic plots relate to the type of control, the stream cross section, cross-section shape changes and shifting control patterns as described by Rantz and others (1982), by Kennedy (1984) and in Chapter I, Volume II of this Manual.

Logarithmic scale selection procedure

The electronic processing system should plot rating curves and rating curve information to logarithmic scales, by default, if the rating is defined as a logarithmic rating. Ratings defined as linear ratings, or equation ratings, may be plotted to logarithmic scales at the hydrographers option. The initial plot should cover the full range of the rating or a selected range if defined. A normal logarithmic scale (no offset) always should be used for the abscissa, or dependent variable. However, the ordinate scale should be adjusted, by default, by an amount equal to the offset defined for the primary rating being plotted. If multiple offsets are defined for the rating, and the hydrographer chooses to plot a continuous rating for the complete range of all segments then the electronic processing system should default to the offset corresponding to the lowest segment of the rating to make the initial plot. If this is a plot for a new rating, where no other rating is to be plotted, then the electronic processing system should define the ordinate scale as a normal log scale (no offset), or use an offset selected by the hydrographer. Although default scale selections and offsets are prescribed, the hydrographer should be allowed to over-ride the defaults and provide his/her own selections.

Generally, it is advised that full log cycles be used for logarithmic scale plots. However, the hydrographer should have the option to set lower and/or upper limits so that only partial log cycles are used at each end of the scales. The setting of scales should be highly flexible and easily changed so the hydrographer can plot and position the rating to the best advantage.

The linear measurement of a log cycle, horizontally and vertically, must be equal. Unless this requirement is met it is impossible to hydraulically analyze the resulting plot of the rating.

Scale offsets

Many rating curves, especially stage-discharge rating curves, are analyzed and drawn on logarithmic scale plots, using a scale offset for the ordinate or gauge-height scale, as described in Chapter 1, Volume II of this Manual. The electronic processing system should allow up to three scale offsets for each rating curve. This procedure conforms to many stage-discharge rating curves where three major segments are present: (a) the extreme low water segment that usually is controlled by a section control; (b) the within bank segment that can be either a section control or channel control and (c) the overbank segment that usually is channel control. Short transition curves that join major rating segments usually are curved lines that will not plot as a straight line, regardless of the scale offset.

Scale offsets must be limited to values that are less than the lowest value of the independent variable for the rating curve, or segment of a curve, being defined. Otherwise, the mathematics would produce zero or negative results, for which logarithms cannot be determined. The electronic processing system should not accept scale offsets that are equal to or greater than the lowest value of the independent variable for the range in which the offset applies.

Negative scale offsets are acceptable. A negative offset for the low segment of a stage-discharge relation would indicate that the gauge height of zero flow is negative. Although such a condition usually is not advised, this condition can result at some gauging stations.

6.9.3 **Rating curve shaping**

Stage-discharge rating curves usually are shaped by fitting a curve or straight line to a series of plotted discharge measurements. For paper plots, this fitting is easily performed by hand with straight edges and plastic curves. For electronic processing system monitor plots, a method(s) should be provided whereby the hydrographer similarly can fit a smooth curve or straight line to points plotted on the electronic processing system monitor. This should be a highly interactive process between the electronic processing system and the hydrographer.

Certain aids should be made available for electronic processing system plots to ensure that stage-discharge ratings are hydraulically correct. One is to plot a theoretical rating based on the control properties and the governing hydraulic equations. The computations and plotting of theoretical ratings should be performed with the electronic processing system, but will require interaction with the hydrographer. Methods of computing theoretical ratings will be described in a subsequent section. The theoretical ratings are used primarily for defining the rating shape, and not necessarily for locating the rating position. The hydrographer must use such ratings with caution, and should make discharge measurements to verify these ratings whenever possible.

Another aid when working with logarithmic scale ratings for stage-discharge stations is to measure the slope of straight line rating segments for comparison to theoretical slopes that correspond to various control conditions. Rating slope computations should be done automatically with the electronic processing system on command. The hydrographer first should designate the end points of the segment of rating where the slope is to measured. The electronic processing system should check to be sure the selected rating curve segment is reasonably close to a straight-line segment. This checking can be done by computing percentage differences of discharge between the actual rating and the straight line defined by the selected end points, at intermediate points along the rating segment. If any difference exceeds ±1 per cent (default value), the rating segment should be considered curvilinear and the slope should not be computed. The electronic processing system should issue a statement to the hydrographer to this effect and simultaneously provide an opportunity for the hydrographer to select a different percentage to use for checking the differences, or to select a different rating segment to check. On the other hand, if the rating segment is found to be a straight line (within the default, or selected, percentage difference), then the slope should be computed and displayed. When displaying a computed slope, the electronic processing system also should include the statement section control for slopes greater than 2.0 and channel control for slopes less than 2.0.

The slope of a logarithmic rating is computed as the horizontal distance divided by the vertical distance. These are measured as linear distances on logarithmic plotting scales. They should not be measured in terms of the independent and dependent variables, but rather in terms of the logarithms of these variables. For a straight-line segment, two points $[(Q_1, G_1)$ *and* $(Q_2, G_2)]$ on the segment can be used to compute the slope using:

$$c = \frac{\log Q_2 - \log Q_1}{\log(G_2 - e) - \log(G_1 - e)} \qquad (6.4)$$

where c = the rating curve slope and e = the scale offset for the independent variable, G.

6.9.4 **Computer development of rating curves**

Rating curve analysis and development are described in detail in previous chapters of Volumes I and II. All rating curves, simple and complex, traditionally have been developed by hand plotting of measurements and manually drawing curves of best fit. Complex ratings, such as slope ratings and ratings, have been developed through a combination of hand calculations and plotting methods. All of these methods are time-consuming and tedious. The computer development methods that can assist the hydrographer in rating curve shaping and definition are given in the following sections.

Stage-discharge ratings

Stage-discharge ratings are graphical relations between stream stage and discharge. These ratings can be developed within the electronic processing system using various plotting and curve drawing functions. However, the hydrographer should use care in ensuring that the ratings are hydraulically correct. The electronic processing system can be used in providing computations that aid in the correct hydraulic shaping of the rating curves. Three such methods, section control, channel control and step backwater, are useful for this purpose.

Section control methods

Rating segments that are controlled by a specific cross section of the stream, such as a sand bar, rock outcropping, manmade weir or other stream feature, can be approximated by flow computations based on a surveyed cross section of the control and the weir equation. The input of cross-section data and the computation of cross-section properties should be a part of the electronic processing system as described in previous sections of this Manual.

Flow computations can be made for the section control by using the cross-section properties, a coefficient of discharge, C and the weir equation. For purposes of defining the theoretical rating shape the method defined here is simplified and some of the more detailed intricacies of weir computations are not accounted for in the method.

The general form for the weir equation to be used for section control computations is as follows:

$$Q = CLh^{1.5} \tag{6.5}$$

where Q = discharge, in cubic metres per second; C = the discharge coefficient; L = the top width, in metres, of the water surface at the control section and for the gauge height of interest and h = the head, in metres (difference between the gauge height and lowest point of the control section).

The discharge coefficient, C, used in the weir equation may be input directly by the hydrographer at the time the cross-section data are entered. A value of C should be required for the lower limit of gauge height for the computations, and for the upper limit of computations. Optionally, C values may be specified for intermediate gauge heights. The electronic processing system should use linear interpolation, based on gauge height, for intermediate values of C.

For control sections where C is not known, the hydrographer may choose to obtain estimates of the C values computed from discharge measurement data. The electronic processing system should allow the hydrographer to designate specific discharge measurements for which a C value would be computed, based on the gauge height and discharge of the measurement, the cross section and the weir equation. The computation of C would be based on the weir equation.

The electronic processing system should display the computed values of C in tabular format for each of the discharge measurements. The hydrographer can use this information to choose values of C to input as described above. The electronic processing system should allow the hydrographer the option to plot gauge height and C and draw a smooth curve of relation. This curve could be used for defining C for the range of theoretical rating curve computations.

The range of theoretical computations for a given cross section should be specified by defining the lower and upper limit gauge height. Intermediate computations should be spaced at 0.1 intervals of gauge height. The theoretical rating curve should be plotted on the rating curve plot and clearly identified as theoretical.

Channel control methods

Rating curve segments that are controlled by channel conditions such as cross-section area, channel slope, channel shape and roughness of the bed and banks can be defined by theoretical computations using the Manning or Chezy equation and a typical cross section near the gauge. Such computations can define the correct hydraulic shape of the rating but not necessarily the correct position of the rating. Computations of this type have been historically referred to as the conveyance-slope method as described by Rantz and others (1982). It is described also in previous chapters of this Manual so the details of these computations will not be repeated here.

The energy slope, S, as used in the Manning and Chezy equations, can be estimated from various sources such as topographic maps and high-water marks. It also can be computed from the Manning or Chezy equations, the surveyed cross section and discharge measurements. The electronic processing system should allow the hydrographer to designate specific discharge measurements for which slope, S, is computed. These computed values of S should be displayed in tabular format, from which the hydrographer can choose values to input at the lower and upper limits of the conveyance-slope computations. The electronic processing system should use linear interpolation to determine intermediate values of slope.

The electronic processing system should provide an option for the hydrographer to plot the computed values of slope and gauge height so that a curve of relation can be drawn. This curve then would be used to determine values of *S* for the conveyance-slope computations.

The range of theoretical computations for a given cross section using the conveyance-slope method should be specified by defining the lower and upper limit gauge height. Intermediate computations should be spaced at selected intervals of gauge height. The theoretical rating curve should be plotted on the rating curve plot and clearly identified as theoretical.

Step backwater method

Step backwater is a water-surface profile computation method that requires a minimum of two cross sections, but generally four or more cross sections are required to produce accurate results. The details of the method are described by Shearman (1990) and were described in Chapter 1, Volume II of this Manual. It is an excellent method to define the shape, and position of the rating curve, and sometimes is used instead of discharge measurements when they are difficult to obtain. Cross-section data and other information necessary for step backwater computation are entered in the step backwater program.

The step backwater method computes water-surface elevations at each cross section in the stream reach downstream from the gauge. The computation depends on a given discharge in the reach and on an assumed water-surface elevation at the downstream end of the reach. Two or more downstream elevations are used to verify that the results at the gauge will define a unique stage-discharge relation. The electronic processing system should provide an option to plot the profiles of water-surface elevations for the various starting elevations for each selected discharge. This type of plot is referred to as a convergence plot that is useful in evaluating the accuracy of the step backwater results.

The electronic processing system should have a direct link to the step backwater software so that results can be transferred easily to the rating analysis for a gauging station. Generally, a series of discharges is selected and for each discharge in the series the step backwater method will compute a gauge height at each cross section used in the computations. The parameters to be transferred are the discharges and the corresponding computed gauge heights for the cross section at the gauge. Each transferred pair (gauge height at the gauge and corresponding discharge) should be plotted on the rating curve and identified as a step backwater computation.

The step backwater program also computes the water-surface elevation for critical depth of flow for each discharge at each cross section. The hydrographer should have the option to select a cross section and plot the critical water-surface elevation computed for that section and the corresponding discharge on the rating plot. This is an additional method to define the shape of a rating where section control is in effect.

Slope ratings

Slope ratings are used for stations with channel controls where variable stream slope downstream from the base gauge affects the position of the stage-discharge relation. Variable stream slope usually is caused by a downstream condition, such as a reservoir, tributary stream or overbank storage. In reality, the term slope rating is a misnomer, because these ratings do not use actual stream slope as a rating parameter. Instead, an index of stream slope is used, which usually is the water-surface fall measured between the base gauge and an auxiliary gauge downstream from the base gauge. For some slope stations, the auxiliary gauge may be located upstream from the base gauge, but a better index of stream slope can be obtained if the auxiliary gauge is located downstream from the base gauge.

The rating method for slope stations involves a complex relation of three separate rating curves: (a) stage-discharge, (b) stage-fall and (c) fall ratio-discharge ratio. These ratings are described in Chapter 2 of this Volume and detailed descriptions of slope ratings can be found in Kennedy (1984) and Rantz and others (1982). Slope ratings usually are classified into three specific types: (a) unit fall ratings, (b) constant fall ratings and (c) limiting fall ratings. Although these different fall ratings are sometimes treated separately in the literature, they can be treated as one rating for computational purposes. This treatment is accomplished by defining the stage-fall rating to fit the specific fall rating type. For instance, if a unit fall rating is desired, then the fall rating is defined so that fall equals 1 ft for all gauge heights. If a constant fall rating is desired, for a fall other than unity, then the fall rating is defined so that the desired constant fall is computed for all gauge heights. Finally, if a limiting fall rating is desired, then the stage-fall rating is defined so that a variable fall is computed, which is dependent on gauge height.

The development of slope ratings must be defined empirically, using discharge measurements, simultaneous measurements of fall and a trial-and-error method to position and shape the individual rating curves. This procedure traditionally has been done by hand plotting and hand computing methods, which is a slow and tedious process. The electronic processing system should provide an interactive process, whereby the hydrographer makes the decisions regarding the curve positions and shape and the system makes the routine computations and plots.

Velocity-index ratings

Velocity-index ratings, like slope ratings, can be used for gauging stations where variable backwater precludes the use of a stage-discharge rating. For velocity-index stations, some method of recording a point or line velocity is required. This recording normally is accomplished with separate electromagnetic or acoustic gauges. Instrumentation and rating development procedures for this purpose have been described in several chapters of Volumes I and II of this Manual.

A stage-discharge rating is not used at gauging stations where velocity-index ratings are used. Instead, ratings are developed for velocity-index versus mean stream velocity and gauge height versus cross-section area. For some streams, gauge height may also be significant independent variable in the velocity-index rating. When this is the case, multiple regression analysis can be used to develop a velocity-index rating. Each of these ratings is developed for a standard cross section of the stream. Details for developing velocity-index ratings are given in Chapter 2 of this Volume. The electronic processing system should provide an interactive process that allows the hydrographer to fit and test the ratings so that the best combination of ratings can be attained.

The methods described in Chapter 2, Volume II of this Manual for velocity-index ratings usually refer to a single channel rating situation. However, these methods can be used where the stream is subdivided into two or more subsections, either horizontally or vertically. In such cases each subsection has its own set of ratings and is computed separately. The total discharge is the sum of the subsection discharges.

Rate-of-change-in-stage ratings

Rate-of-change-in-stage ratings sometimes are used at gauging stations where changing discharge causes a variable stream slope. These ratings are used for stations with a condition frequently referred to as loop ratings. One of four methods: Jones, Boyer, Lewis or Wiggin's, usually is used to determine the rating at a station with this condition. These methods are described in Chapter 2 of this Manual. Empirical, trial-and-error methods are used to develop these ratings and require a number of discharge measurements. Like other complex ratings, these ratings traditionally have been done using hand computations and hand-plotting methods. The electronic processing system should provide an interactive method so the hydrographer can quickly and easily develop a rate-of-change in stage rating. The hydrographer should be allowed to fit and test trial ratings until the best combination is attained.

6.10 **Discharge measurement shift adjustments**

Discharge measurements are used primarily to check rating curves to insure that currently used rating curves are still valid. The electronic processing system should automatically compute the shift information for each discharge measurement. The shift information should, by default, be computed on the basis of the rating curve applicable for the time and date of the discharge measurement; however, the hydrographer should be allowed to specify a different rating curve for which the shift information is computed. If, at a later date, a new rating curve is prepared, then the shift information should be automatically updated for all measurements that fall within the period of time that the new rating is applicable. Shift information should be displayed as part of the output for each discharge measurement.

The following sections describe the methods of computing shift information for individual discharge measurements made at stage-discharge stations, slope stations, rate-of-change in stage stations and stations. Shifts are not computed or used for structure stations and BRANCH model stations. Definition of shift curves, use of partial or average shifts and other aspects of shift application are described in a later section of this Manual.

6.10.1 **Shifts for stage-discharge ratings**

The shift information that should be computed for discharge measurements applicable to stage-discharge rating curves is as follows:

(a) Rating shift, S_r – This shift is the numerical difference between the gauge height, G_r, which corresponds with the rating curve discharge for the measurement, and the gauge height, G_m, of the discharge measurement. The resulting algebraic sign should be observed. The equation is:

$$S_r = G_r - G_m \tag{6.6}$$

(b) Measurement percent difference, D – This is the percent difference between the measured discharge, Q_m, and the rating curve discharge, Q_r, that corresponds to the gauge height of the discharge measurement. This represents the difference between the measured discharge and rating discharge if no shift is applied. The equation is:

$$D = 100\left(Q_m - Q_r\right)/Q_r \tag{6.7}$$

(c) Shifts for the gauge height of zero flow, S_0 – If the gauge height of zero flow, G_0, is determined either when a regular discharge measurement is made, or independently during a visit to the gauging station, then it is possible to compute a shift for that gauge height if the rating curve is defined down to zero flow. This information can be very useful as an aid in defining the low end of a shift curve. The equation for computing the shift for the gauge height of zero flow is similar to equation 5.6 for computing the rating shift, and is:

$$S_0 = G_r - G_0 \tag{6.8}$$

Because the discharge corresponding to G_0 is by definition zero, it is not possible to compute a measurement percent difference.

6.10.2 **Shifts for slope ratings**

Slope ratings usually are referred to as complex ratings because they involve two sites for measuring gauge height (a base and auxiliary gauge) and three individual ratings of different parameters. The required ratings are (a) a stage-discharge rating, (b) a stage-fall rating and (c) a fall ratio-discharge ratio rating. The following paragraphs describe how shift information is computed for individual discharge measurements at stations with slope ratings.

The stage-discharge rating is the only rating of the three slope station ratings that is allowed to be shifted, and shift information is referenced to this rating. If either the fall rating or the ratio rating changes then new ratings should be prepared. It also should be noted that slope ratings only may apply to certain ranges of stage and in some cases only when the fall is less than a specified amount.

For slope ratings, the measured discharge, Q_m, is considered the true discharge. The adjusted discharge, Q_{adj}, is an adjustment of the measured discharge that is computed by using the observed stages at the base gauge and the auxiliary gauge, the observed fall, which is the difference between the two observed stages, and the defined rating curves. This adjusted discharge is used for comparison to the rating discharge, Q_r, to determine shift information. If no shift is present, then Q_{adj} and Q_r will be equal. The method for computing Q_{adj} and shift information is as follows:

(a) Adjusted discharge, Q_{adj} – First, compute the measured fall, F_m, as the difference between the observed mean gauge height for the measurement at the base gauge, G_b, and the auxiliary gauge, G_a. The equation is:

$$F_m = G_b - G_a \tag{6.9}$$

Second, if the auxiliary gauge is upstream from the base gauge, reverse the order of G_b and G_a in equation 6.9;
Third, determine the rating fall, F_r, that corresponds to the base gauge height, G_b, from the stage-fall rating;
Fourth, compute the fall ratio, R_f, of the measured fall to the rating fall. The equation is:

$$R_f = F_m / F_r \tag{6.10}$$

Fifth, determine the discharge ratio, R_q, corresponding to R_f from the ratio rating;
Finally, compute the adjusted discharge, Q_{adj}, based on the measured discharge, Q_m, and the discharge ratio, R_q. The equation is:

$$Q_{adj} = Q_m / R_q \tag{6.11}$$

(b) Stage-discharge rating shift, S_r – Determine the gauge height, G_r, corresponding to the adjusted discharge, Q_{adj}, from the stage-discharge rating. Compute the shift, S_r, based on the observed gauge height, G_b, for the base gauge and the rating gauge height, G_r. The equation is:

$$S_r = G_r - G_b \tag{6.12}$$

(c) Measurement percent difference, D – The percent difference, D, between the adjusted discharge, Q_{adj}, and the rating discharge, Q_r, also should be computed. This percentage represents the error of the adjusted discharge from the rating discharge if no shift is applied.

The equation is:

$$D = 100\left(Q_{adj} - Q_r\right)/Q_r \qquad (6.13)$$

6.10.3 Shifts for rate-of-change-in-stage ratings

Rate-of-change-in-stage ratings are complex ratings apply to streams where rapid changes in stage affect the stage-discharge rating. The most commonly used rating of this type is the Boyer method which is described herein. The Boyer method includes a stage-discharge rating and a rating of stage versus the factor, $1/US_c$. The term $1/US_c$ is a measure of flood-wave velocity, U, and the constant discharge stream slope, S_c. This term usually is defined empirically from the discharge measurements. The greatest effect of changing stage occurs on streams having relatively mild slopes and rapid changes in discharges. Frequently, this effect will happen when the flow regime of a stream has been changed artificially, such as below a dam when releases are made quickly or in urban areas where basin development causes rapid increases in flow rates for a stream that was previously sluggish.

Shift information for Boyer ratings should be computed only for the stage-discharge rating. The rating of stage versus $1/US_c$ should not be shifted. If this rating changes a new rating should be prepared. The shift and percent difference should be based on the rating discharge, Q_r, and the adjusted discharge, Q_{adj}.

The method for computing the adjusted discharge and the shift information for Boyer ratings is as follows:

(a) Adjusted discharge, Q_{adj} – First, compute the change in stage, dG, for the discharge measurement as the difference between the ending gauge height, G_e, and the starting gauge height, G_s. For rising stages the difference is positive and for falling stages the difference is negative. The equation is:

$$dG = G_e - G_s \qquad (6.14)$$

Second, compute the elapsed time, dt, for the discharge measurement as the difference between the ending time, t_e and the starting time, t_s. The equation is:

$$dt = t_e - t_s \qquad (6.15)$$

Third, compute the rate-of-change in stage, dG/dt, for the discharge measurement;

Fourth, determine the factor, $1/US_c$, for the mean gauge height of the discharge measurement, from the stage-$1/US_c$ rating;

Fifth, compute the adjustment factor, F_{adj}, using the following equation:

$$F_{adj} = 1 + \sqrt{\left(\frac{1}{US_c}\right)\left(\frac{dG}{dt}\right)} \qquad (6.16)$$

Finally, compute the adjusted discharge, Q_{adj}, as:

$$Q_{adj} = Q_m / F_{adj} \qquad (6.17)$$

The adjusted discharge, Q_{adj}, represents the discharge that would be computed from the two ratings and the observed gauge height if no shift is applied;

(b) Rating shift, S_r – Determine the rating gauge height, G_r, corresponding to the adjusted discharge, Q_{adj}, from the stage-discharge rating. Compute the shift, S_r, as the difference between the rating gauge height, G_r, and the measured gauge height, G_m, as:

$$S = G_r - G_m \qquad (6.18)$$

(c) Measurement percent difference, D – Determine the rating discharge, Q_r, from the stage-discharge rating using the measured mean gauge height, G_m. Compute the percent difference, D, between the adjusted discharged, Q_{adj}, and the rating discharge, Q_r, as:

$$D = 100\left(Q_{adj} - Q_r\right)/Q_r \qquad (6.19)$$

This percent difference represents the error between the Boyer adjusted discharge and the rating discharge if no shift adjustment is applied.

6.10.4 Shifts for stations

Ratings at gauging stations with velocity index as part of the rating system are considered complex ratings and in some cases can be extremely complex if two or more velocity meters or index velocity measurement points along a line/plane are in use. Stream channels may be subdivided either vertically or horizontally, with each subdivision having a specific set of ratings, or in some cases the individual meters may be averaged for use with one set of ratings. Also, for some stations discharge measurements may be made so that only the total discharge is computed, with no accurate method of subdividing the measured discharge into the various rating components. Because of this variability it is not possible to describe all of the ways that rating shifts are computed. The electronic processing system should provide an interactive mode that allows the hydrographer to define the shifts and the shifting method.

Shift information for a basic rating is described in the following paragraphs. A basic rating includes a single rating of stage and cross-section area, a single rating of velocity-index and mean velocity and in some cases an optional rating of stage and a velocity correction factor. The rating discharge, Q_r, is computed by multiplying the cross-section area, A_r, from the area rating, times the mean velocity, V_r, from the velocity rating, and times the velocity correction factor, K_r, from the stage-factor rating. If the velocity correction factor is not used it is set to a default value of 1.00. The basic equation for discharge is:

$$Q_r = A_r V_r K_r \tag{6.20}$$

Shifts are allowed only for computation of V_r from the velocity rating. The stage-area and stage-factor ratings should not be adjusted through the use of shifts. If either the stage-area or the stage-factor ratings change, then new ratings should be prepared.

It also should be noted that a standard cross section must be used for the ratings and for computing shifts. That is, a specific cross section in the stream channel should be designated as the rating section. This cross section may be the same section as used for making discharge measurements or it may be a different section. All computations should be related to and based on the standard cross section. For instance, the mean stream velocity, as used for rating purposes, should be computed by dividing the measured discharge by the cross-section area determined from the stage-area rating of the standard cross section. This mean stream velocity is the velocity that should be used to check or define the velocity rating and the one to be used for plotting purposes on the velocity rating for those sites where a stage-factor rating is not used. If a stage-factor rating is used then this velocity should be adjusted by dividing it by the applicable factor before using it to check or define the velocity rating.

The order of computations for shift determinations is important because two, and in some cases three, ratings are involved. The following step-by-step procedure should be used:

(a) Standard cross-section area, A_r – Determine the cross-sectional area, A_r, of the standard cross section from the stage-area rating, using the mean gauge height, G_m, of the discharge measurement;

(b) Velocity correction factor, K_r – Determine the velocity correction factor, K_r, from the rating of stage and velocity correction factor, using the mean gauge height, G_m, of the discharge measurement. If this rating is not used, then set the velocity correction factor to a default value of 1.00;

(c) Adjusted mean stream velocity, V_m – Compute the mean stream velocity, adjusted for the velocity correction factor, for the standard cross section using:

$$V_m = \frac{Q_m}{A_r K_r} \tag{6.21}$$

where Q_m is the measured discharge, and the other variables are as previously defined;

(d) Rating velocity-index, V_{ir} – Determine the rating velocity-index from the rating of velocity-index and mean stream velocity, by entering the rating with the adjusted mean stream velocity, V_m, as computed in equation 6.21;

(e) Velocity-index shift, S_v – Compute the velocity-index shift as the difference between the rating velocity-index, V_{ir}, and the mean measured velocity-index, V_{im}, for the discharge measurement. The shift, S_v, is defined by:

$$S_v = V_{ir} - V_{im} \tag{6.22}$$

S_v should retain the resulting algebraic sign (+ or –) for application purposes. When the computed shift is applied to the measured velocity-index, V_{im}, it will yield a corrected velocity-index to use for entry to the velocity rating when determining the rating mean velocity, V_r;

(f) Measurement percent difference, D – The measurement percent difference is the percentage of error between the measured discharge, Q_m, and the unshifted discharge, Q_r. To compute the unshifted rating discharge, Q_r, use equation 6.20 as described in previous paragraphs. The measurement percent difference is computed as:

$$D = 100\left(Q_m - Q_r\right)/Q_r \tag{6.23}$$

6.11 APPLICATION OF SHIFT ADJUSTMENTS

Shifts are gauge-height adjustments used to account for temporary changes to rating curves without having to re-define the rating curve. The methods for computing shift information for the various types of discharge measurements are described in the previous section, Discharge measurement shift adjustments. For surface-water computations, shift adjustments are added to unit values of the input parameter to yield temporary unit values that are applied to the rating curve for computation of the output dependent variable. The algebraic sign of the shift must be maintained correctly. When measurements plot above a rating curve, that is, when the actual gauge height for a given discharge is higher than indicated by the rating curve, the sign of the shift is negative. When measurements plot below a rating curve the sign of the shift is positive. Also, it is important to note that a shift is a temporary correction, used only for computational purposes. It does not permanently alter the input unit value.

Although most shifts will apply to stage-discharge ratings they also may be defined and applied to the velocity-index versus mean velocity rating for velocity-index stations. Shifts should not be allowed for any other types of rating curves except stage-discharge ratings and velocity-index and mean velocity ratings. Because shifts are predominantly used for stage-discharge ratings, the shift discussions in this section will relate to that type of rating. Much of the following discussion regarding application of shifts is based on Kennedy (1983).

Shifts usually are applied only when discharge measurements deviate from a rating curve by more than a specified percentage. The specified percentage frequently is based on the accuracy of discharge measurements that can be made at the gauging station. For instance, if discharge measurements can be made with 5 per cent or better accuracy then shifts will be used only when measurements deviate more than 5 per cent from the rating. Otherwise, if more than two or three consecutive discharge measurements consistently plot on one side of the rating a shift curve may be used for these measurements even though they are within the specified shift percentage.

The shift adjustments that apply during the periods between discharge measurements must be interpolated by an appropriate method before the unit and daily discharge records can be computed. The method used will depend on the hydrographer's judgment considering the nature of the shifting, the frequency of measurements and the type of channel and control.

Small shifts that change gradually may be distributed satisfactorily by inspection using mental interpolation. Larger shifts, whose variations are adequately defined by discharge measurements, warrant a more rigorous analysis with some form of graphic shift-adjustment-variation diagram. The accuracy of discharge records computed from a rating with large and erratic shifts depends to a great extent on the frequency of discharge measurements, and particularly unstable streams may need weekly or even daily measurements to define the day-to-day shift-adjustment variation.

6.11.1 Shift-adjustment variation diagrams

A shift-adjustment variation diagram (sometimes called a V-diagram), a graph of the relation between shift adjustment and either time or stage, is commonly used to interpolate shift adjustments between measurement-defined values. The V-diagram shifts can be graduated with time, stage or time and stage simultaneously either manually or as part of an electronic processing system.

When a low-water control is scoured or filled or affected by backwater from leaves, debris or aqueous growth, the corresponding rating shift is greatest at low water and normally tapers to zero at some higher stage. This is called a stage-variable shift. If the channel is alluvial and its bed is raised and lowered by sediment being picked up or deposited, the shift variation with stage may be negligible compared to its variation with time, and the shifts are called time variable. Most streams have shifts that must be graduated with stage while the stage graduation is changing with time.

Time-variable-shift distribution can be made manually or by an electronic processing system using interpolation between discharge measurement-defined shifts. The hydrographer may introduce arbitrary data points based on judgment and knowledge of stream conditions.

Stage-variable-shift distribution can be made by using V-diagrams similar to those in Figure II.6.3. Each diagram involves a base-rating curve (the numbered rating in effect at the time) a shift-rating curve (a rating curve drawn to fit the measurements that define the shift, usually only on the rating work-curve sheet) and the V-diagram (the gauge-height differences between the base curve and shift curve, plotted against stage). The shift curve should normally be drawn first with the same consideration given to its shape as would be given to a numbered rating. The V-diagram is best defined by drawing its corresponding shift curve first. The V-diagrams of the type in Figure II.6.3(a), for relatively small shifts, may be defined directly from measurement-defined shifts without drawing a shift curve. This process is not recommended with other V-diagram types or for large shifts where it could lead to grossly misshapen equivalent ratings and dubious discharge records.

The use of a stage-varied-shift adjustment is equivalent to drawing a new numbered rating curve and may be preferable for temporary rating changes. The principal use for stage-shift diagrams is one step in the process used for varying shifts with both stage and time as explained in subsequent paragraphs of this section.

Figure II.6.3 illustrates typical stage-shift V-diagrams and the relations between their corresponding base rating curves and shift curves. The V-diagrams for manual application are usually curved and shifts are determined by direct readings from the curve. The V-diagram must be approximated by two or more straight lines for application by an electronic processing system. Selected coordinates that define the V-diagram are entered to the electronic processing system and applied by interpolation between the entered points.

Shift adjustments varied by time only

The simplest way to vary shift adjustments between discharge measurements is by time interpolation. Time-varied shifts are usually used for periods when stage does not change very much, and the shifting control is affected by a gradual change due to scour or fill. For example, such a condition might be caused by gradual accumulation of falling leaves on a section control. Time interpolation of shifts is sometimes more convenient when computing discharge records by hand methods. For automatic data processing, time interpolation of shifts can be accomplished through the use of two or more constant (vertical) shift-variation diagrams with linear interpolation between successive diagrams.

Shift adjustments varied by stage only

The use of a stage-varied-shift adjustment is equivalent to drawing a new numbered rating curve and may be preferable for temporary rating changes. The shift-variation diagrams shown in Figure II.6.3 are typical stage-only diagrams, as described in a previous section. They are applied over a period of time by manually or automatically determining the shift for each stage value for which discharge is computed during the specified time period. Stage-only shift-variation diagrams are an integral part of the more typical situation, where shift application is varied by both stage and time, as described in the following section.

Shift adjustments varied by time and stage

Two or more shift curves can be used in combination to apply shifts to unit values so that the shifts are varied either by time only, or both stage and time. Varying the shift in this way is accomplished by defining a shift curve and assigning it a starting date and time, but no ending date and time. A second shift curve is defined with a subsequent starting date and time. If the two shift curves are defined so that each one has a different constant shift (not varied with stage), then the electronic processing system will interpolate between these two shifts based on time only. This procedure commonly is referred to as time interpolation of shifts as described previously. If two consecutive shift curves are entered so that one or both of them have shifts that vary by stage, then the electronic processing system will interpolate shifts based on both stage and time for all unit values between the two assigned shift curves.

Shift curves should be defined and numbered as a means of describing and tracking specific shifting characteristics at specific points in time. Each shift curve usually is based on one or more discharge measurement and other field observations that define a change in the position of the rating curve, and this change usually is considered a temporary change. To estimate shifts at other times, intermediate to the defined shift curves, a linear-interpolation procedure is used.

Individual shifts, and not entire shift curves, should be interpolated. That is, only those shifts needed to adjust unit values should be determined by interpolation, and not those outside the range of recorded unit values. Likewise, the interpolation process should be continuous in time, so that a shift interpolation is performed for each unit value to which shifts are to be applied.

The interpolation procedure is described in the following step-by-step example:

(a) Two shift curves, numbered 001 and 002 for example, are defined graphically for use at dates and times, t_1 and t_2, respectively;
(b) An interpolated shift, S_n, is required for unit value, G_n, at an intermediate date and time, t_n;
(c) The electronic processing system computes the shifts, S_1 and S_2, corresponding to the unit value, G_n, from each of the shift curves, 001 and 002, respectively;
(d) The electronic processing system performs an un-weighted, linear time interpolation of shifts S_1 at time t_1, and S_2 at time t_2, to obtain the shift, S_n, at time t_n;
(e) The same interpolation procedure is used to estimate shifts for all other unit values resulting between times, t_1 and t_2.

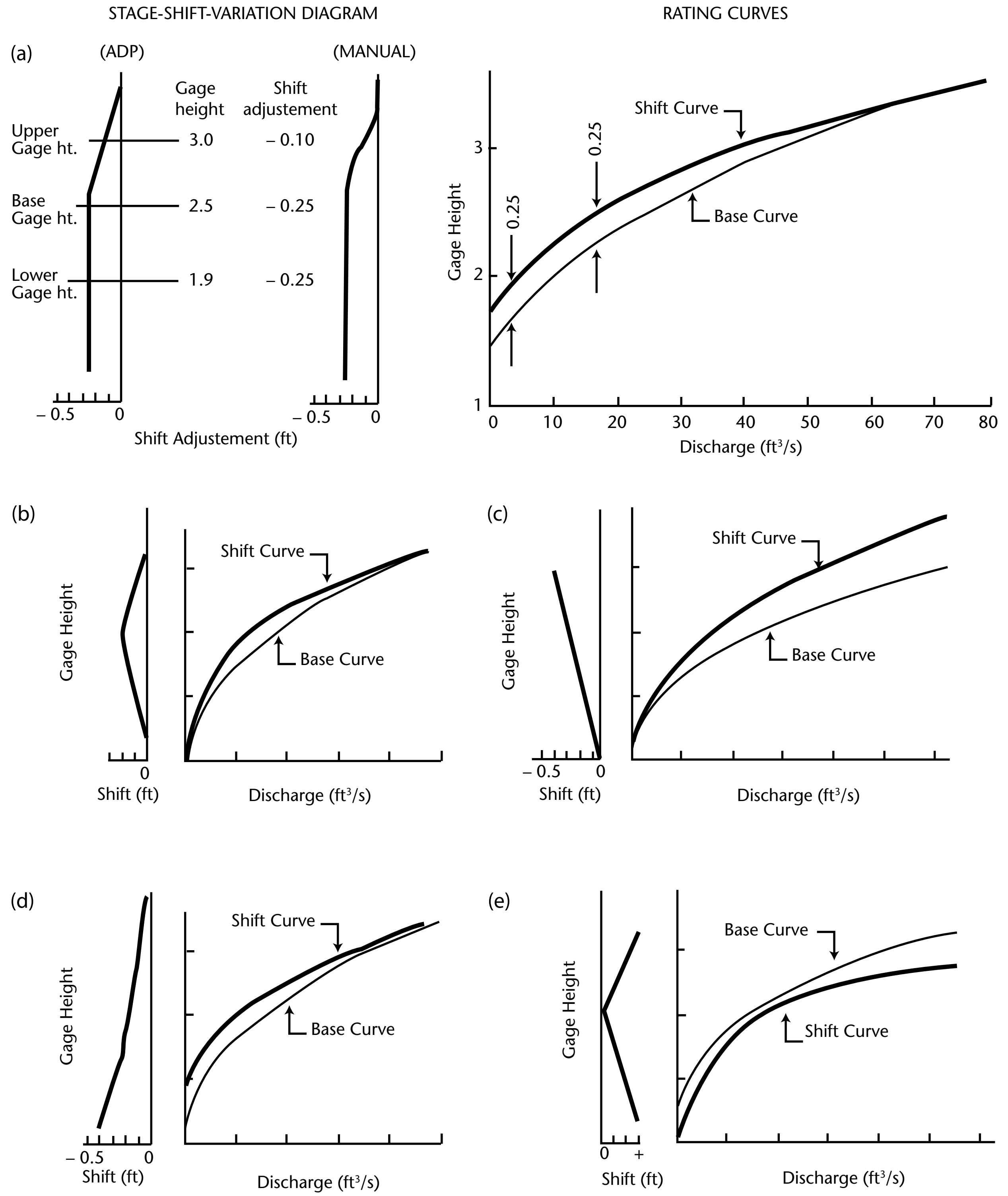

Figure II.6.3. Effect of various stage-shift-variation diagram shapes on shift-curve shape

All computed and interpolated shifts should be rounded to the same number of significant figures as used for the gauge height or other unit value to which the shift is to be applied. Rounding should be performed before any application process.

6.11.2 Unit value graphical comparison of shifts

Shifts that are applied to a time series of unit values should be displayed with the electronic processing system in a graphical plot. The graphical comparison should show a time-series plot of the unit values of gauge height (or other independent variable) and a superimposed plot of the unit values of shifts. Scales for the two plots should be used so that each plot is easily discernible and readable. The hydrographer should have the option to change either or both of the scales. An example plot is shown in Figure II.6.4.

6.12 PRIMARY COMPUTATIONS

Primary computations are the functions that convert input data, such as gauge height, velocity index and other auxiliary data into time series of unit values, daily values, monthly values and annual values of discharge, mean velocity, reservoir contents and other output parameters. In the past primary computations were generally performed by hand. Today, almost exclusively, primary computations are performed by automated data processing systems. Therefore, the subsequent discussions of primary computations are described in terms of automated data processing methods.

The primary computation process is dependent on the type of gauging station and, except for stage-only stations, always will require the use of at least one rating curve. To carry out the conversion process previously developed data and information will be required, such as time series of input variables, correction diagrams, shift curves and rating tables. The conversion should be carried out with minimal interaction from the hydrographer and should produce files of information that can be used to produce tables and graphs that commonly are referred to as primary output.

6.12.1 Unit value computations

Unit value files of uncorrected input parameters, such as gauge height and velocity index, are entered to the electronic processing system as described in previous sections of this chapter. Also, specific information such as parameter correction diagrams, shift curves and rating curves are entered as described previously. The primary computations should produce additional unit values files of specific output parameters, dependent on the station type. These unit values and their associated time tags are saved for the purpose of computing daily mean values, various statistics and for archiving. The unit values files that should be computed for each type of station are described in the following sections.

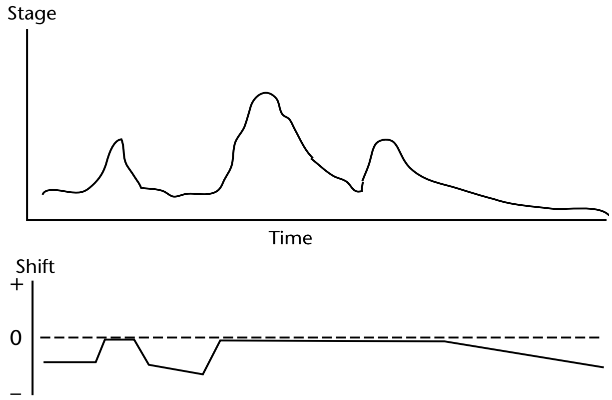

Figure II.6.4. Example plot of unit values of stage and shifts

Stage-only stations

Stage-only stations are those stations where unit and daily mean values of gauge height and associated statistics are required. For this type of station only the unit values files of gauge-height data and the gauge- height correction information are needed. Primary computations should create the following unit values files. Unless otherwise noted, each unit value file should be saved for further use and for archiving:

(a) Gauge-height corrections – The electronic processing system should evaluate and compute the gauge- height correction that corresponds to each input value of gauge height. Gauge-height corrections include instrument errors, gauge datum errors and gauge datum conversions (for example, conversion to a mean sea level datum). The computations should use each correction and correction diagram as defined by the hydrographer. The corrections and correction diagrams should be interpolated by time and stage, as required. If two or more corrections or correction diagrams apply to the same time period, the gauge-height correction should be determined from each one independently for each time step, and summed to produce the cumulative correction for each time step. All gauge-height corrections should be rounded to standard gauge-height precision before using them in further calculations. The resulting time series of cumulative gauge-height correction values should be saved as a working file, and for later archiving;
(b) Corrected gauge heights – A unit values file of corrected gauge heights should be computed by adding the cumulative gauge-height correction (see above) to the input unit values of gauge height for each time step. This file of corrected

gauge heights is considered the final, and most accurate, gauge-height record for the gauging station. The file also should be saved for further computations and archiving.

Stage-discharge stations

Stage-discharge stations are those stations where unit and daily values of discharge are computed based on unit values of gauge height and a stage-discharge rating curve. This station is the most common type of gauging station and requires unit values files of gauge height and information defining gauge-height corrections and shift adjustments. Unless otherwise noted each unit value file should be saved for further use and archiving:

(a) Gauge-height corrections – A file of unit values of cumulative gauge-height corrections should be computed and saved for each unit value of gauge height, as described for stage-only stations in the preceding section;

(b) Corrected gauge Heights – A file of unit values of corrected gauge heights should be computed and saved, as described for stage-only stations in the preceding section;

(c) Shift adjustments – Unit values of shifts should be computed for each unit value of corrected gauge height. These shifts should be based on the shift curves defined by the hydrographer and for the applicable time period. Interpolation of shifts by time and stage should be performed with the electronic processing system according to the methods described in previous sections. All unit values of shifts should be rounded to standard gauge-height precision before using them in further computations. The computed unit value shifts for each gauge height and time step should be saved in a unit values file for further use and for archiving;

(d) Discharge – Unit values of discharge should be computed by temporarily adding the shift adjustment to the corrected gauge height for each time step. The corrected and shifted gauge height then should be used to determine the corresponding discharge from the applicable rating curve. The shift-adjusted gauge height is a working value only and should not permanently alter the gauge height. It is not required that the shift-adjusted gauge heights be saved. The computed unit values of discharge, however, should be saved for later use, and archiving.

For the low end of the rating, and if the rating is defined to zero discharge, all shift-adjusted gauge heights that are lower than the gauge height of zero flow will be assigned a unit value discharge of zero. If the rating is not defined to zero flow, and a shift adjusted gauge height is below the lowest gauge height of the rating, a flag should be set indicating the rating was exceeded on the low end. Rating extrapolations can be made by the hydrographer at a later point in the processing.

Velocity index stations

Velocity index stations are those stations where unit values of discharge are computed on the basis of unit values of gauge height, cross-section area, an velocity-index, mean stream velocity and a velocity adjustment factor (optional). At least two rating curves are required: (a) a stage-area rating and (b) an index velocity versus mean stream velocity rating, or an index velocity and stage versus mean stream velocity in the case of a multiple parameter rating based on velocity and stage. A third rating sometimes is used relating stage to a velocity adjustment factor. Information defining gauge-height corrections, velocity-index corrections and index- velocity shift adjustments also are required.

Two unit value input files are used for velocity index stations: (a) an input file of unit values of gauge height and (b) an input file of unit values of velocity-index. For various reasons these files may not have corresponding and simultaneous time steps which is required for the unit value computations of discharge. If the time steps for the two files do not correspond the electronic processing system should automatically interpolate each file to provide estimated unit values corresponding to all recorded times of both files. That is, the gauge-height file should be interpolated so that an estimated gauge height is available for all time steps of the velocity-index file, and conversely, the velocity-index file should be interpolated so that a velocity-index is available for all time steps of the gauge-height file. Therefore, this method doubles the size of each of the input unit values files. The electronic processing system should flag, save and archive all estimated unit values, together with the recorded unit values.

Unit value files should be computed with the electronic processing system for the following parameters. Unless otherwise noted, each unit value file should be saved for further use, and for archiving:

(a) Gauge-height correction – A file of unit values of cumulative gauge-height corrections should be computed and saved for each unit value of gauge height (including estimated values) as described previously for stage-only stations;

(b) Corrected gauge heights – A file of unit values of corrected gauge heights should be computed by adding the gauge-height corrections to the corresponding unit values of gauge heights;

(c) Velocity adjustment factor – If a rating of gauge height versus velocity adjustment factor is used for the gauging station a velocity adjustment factor should be computed from that rating for each unit value of corrected gauge height. Shift adjustments are not applied to gauge height for use with the velocity adjustment factor rating. If a velocity factor rating is not used for the station, then the velocity adjustment factor of 1.00 is used for all gauge heights. Velocity adjustment factors should be rounded to two decimal places for application purposes;

(d) Cross-section area – The cross-sectional area should be computed for each unit value of gauge height using the stage-area rating;

(e) Velocity-index correction – Correction values should be computed for each input value of velocity-index (including estimated values), based on the velocity-index correction value diagrams, and the methods of interpolation as previously described. All velocity-index correction values should be rounded to standard velocity precision;

(f) Corrected velocity-index – Each input value of velocity-index should be corrected by adding the velocity-index correction value to the corresponding value of the input velocity-index;

(g) Velocity-index shifts – Shifts for each value of the corrected velocity-index should be computed based on the velocity shift curves. All velocity shifts should be rounded to standard velocity precision before applying to further computations;

(h) Mean rating velocity – The mean rating velocity should be computed for each shift adjusted value of the corrected velocity-index by using the rating of velocity-index versus mean velocity;

(i) Mean stream velocity – The mean stream velocity should be computed for each time step by multiplying the mean rating velocity times the velocity adjustment factor;

(j) Discharge – The unit values of discharge should be computed by multiplying each unit value of cross-sectional area times the corresponding value of mean stream velocity.

For some velocity index stations, two or more horizontal subsections may be present, each of which has its own set of unit values. For these stations unit values files are computed for each subsection as described above. Unit values of the total discharge for the stream for each time step is computed as a summation of the corresponding unit values of the subsection discharges. If time steps for the subsections do not correspond, interpolation of unit values will be required.

For streams where two or more velocity-index meters are positioned to measure velocity at different vertical positions a velocity averaging procedure should be used to compute an average velocity-index for the stream. Various averaging procedures can be used depending on the gauge configuration and the number of velocity-index gauges. The electronic processing system should provide for hydrographer-defined equations to compute average velocity-index. The ratings for such a station are based on the average velocity-index. All other aspects of computing unit values of discharge for the stream are the same as described above.

Slope stations

Slope stations are those stations where discharge is computed on the basis of a stage-discharge relation that is adjusted for variable water-surface slope. Water-surface slope cannot be measured directly so the water-surface fall between the base gauge and an auxiliary gauge is used as an indicator of slope. The two gauges must be set to the same datum to determine fall between the gauges. The auxiliary gauge preferably is located downstream from the base gauge at a distance that provides a measurable fall but does not introduce hydraulically appreciable channel changes or tributary inflow. For some sites the auxiliary gauge may be located upstream, but this is not advised because the water-surface slope in the upstream reach is not as representative of backwater conditions as it is in the downstream reach.

Computation of discharge at a slope station requires unit values of gauge height at the base gauge and the auxiliary gauge. Three ratings are required: (a) stage-discharge, (b) stage-fall and (c) fall ratio versus discharge ratio. Information defining gauge-height corrections for the base gauge and the auxiliary gauge, and shift adjustments for the base gauge are required.

Timing accuracy of unit-value data is very important at each gauge because water-surface fall computations require that time synchronous stage data be available for the base gauge and the auxiliary gauge. Even with the best timers and time-correction methods it is not always possible to obtain this kind of accuracy, and stage data will sometimes be recorded and/or time corrected to different time steps for the two gauges. For such situations, the stage data for the base gauge should be interpolated so that estimated stage values are available for each corresponding stage value at the auxiliary gauge. Likewise, the stage data at the auxiliary gauge should be interpolated so that estimated stage values are available for each corresponding stage value at the base gauge. This procedure effectively doubles the number of stage values at each gauge, half of which are measured values and half are estimated values. The electronic processing system should flag, save and archive all estimated unit values, together with the recorded unit values.

Computations of discharge using the slope method are subject to constraints that should be checked and applied for each unit value computation. These constraints are:

(a) Slope ratings should not be used if the measured fall values are negative. In these cases, discharges should not be computed and the electronic processing system should issue a warning that negative fall values have been encountered;
(b) Slope affected ratings may apply throughout the range in stage measured at a station or they may apply only for a specific range in stage. The hydrographer should designate the lower and upper limits of the slope rating by entering a minimum gauge height and a maximum gauge height, below and above which the slope rating procedures should not be used. Discharge should be computed directly from the stage-discharge rating for gauge heights that are outside these limits;
(c) Slope ratings may, in some situations, have maximum fall constraints. That is, for measured fall values exceeding a designated amount, or for measured fall exceeding the fall from the stage-fall rating, no slope adjustments should be applied. The hydrographer should enter a maximum fall so that when measured falls exceed this value, slope adjustments will not be made. Likewise, the hydrographer should designate that when measured fall exceeds the rating fall slope adjusted computations will not be made. For both of these situations, unit values of discharge should be computed by direct application of the stage-discharge rating;
(d) For some slope stations, constraints 2 and 3 both may apply, and should be checked.

Unit value files should be computed with the electronic processing system for the following parameters, subject to the above constraints. Unless otherwise noted, each unit value file should be saved for further use and archiving:

(a) Gauge-height corrections, base gauge – A file of unit values of cumulative gauge-height corrections for the base gauge should be computed and saved for each corresponding unit value of gauge height (including estimated values), as described previously for stage-only stations;
(b) Corrected gauge heights, base gauge – A file of unit values of corrected gauge heights for the base gauge should be computed by adding the gauge-height corrections for the base gauge to the corresponding unit values of gauge heights;
(c) Gauge-height corrections, auxiliary gauge – A file of unit values of cumulative gauge-height corrections for the auxiliary gauge should be computed and saved for each corresponding unit value of gauge height (including estimated values), as described previously for stage-only stations;
(d) Corrected gauge heights, auxiliary gauge – A file of unit values of corrected gauge heights for the auxiliary gauge should be computed by adding the gauge-height corrections for the auxiliary gauge to the corresponding unit values of gauge heights;
(e) Measured water surface fall – A file of unit values of measured water-surface fall should be computed by subtracting each unit value of gauge height at the auxiliary gauge from the corresponding gauge height at the base gauge. If the auxiliary gauge is located upstream from the base gauge, fall should be computed by subtracting the base gauge height from the auxiliary gauge height;
(f) Shift adjustments – For slope stations shift adjustments are used only for the stage-discharge rating for the base gauge. A unit values file of shift adjustments should be computed for each base gauge height, including estimated values, by using the defined shift curves and the time/stage interpolation procedures described previously. If shift curves are not applicable for specific time periods, shifts should default to zero for that time period;
(g) Rating discharge – Unit values of rating discharge are computed for each unit value of shift adjusted gauge height for the base gauge, using the stage-discharge rating for the base gauge. The rating discharge is an unadjusted discharge value, and does not represent the true discharge of the stream;
(h) Rating fall – Unit values of rating fall are computed for each unit value of gauge height (not shift adjusted) for the base gauge, using the stage-fall rating for the base gauge;
(i) Fall ratio – Unit values of fall ratio are computed by dividing the measured water-surface fall by the rating fall;

(j) Discharge ratio – Unit values of the discharge ratio are computed using the rating curve of fall ratio versus discharge ratio;
(k) Discharge – Unit values of discharge are computed by multiplying the rating discharge times the discharge ratio. The resulting discharge represents the true discharge of the stream.

Rate-of-change-in-stage stations

Rate-of-change-in-stage stations are those stations where discharge is computed on the basis of a stage-discharge relation that is adjusted for variable rates of change in stage. Computation of discharge, as based on the Boyer Method, requires unit values of gauge height. Two ratings are required: (a) a stage-discharge rating and (b) a stage versus $1/US_c$ rating. Information defining gauge-height corrections and shift adjustments also are required.

Computation of discharge using the Boyer Method is subject to constraints that should be checked and applied for each unit value computation. These constraints are:
(a) Rate-of-change-in-stage ratings apply only to high discharges where channel control conditions are effective. The hydrographer should specify a minimum gauge height and a maximum gauge height, below and above which the rate-of-change-in-stage computations should not be applied. Discharge should be computed directly from the stage-discharge relation when the stage is outside these limits;
(b) Rate-of-change-in-stage computations are frequently not made when the Boyer adjustment factor results in only a small change of the rating discharge. The electronic processing system should use default values of 0.96 to 1.04 as the range of Boyer adjustment factors for which adjustments would not be made. The hydrographer should be allowed to change these values, if necessary (for example, to achieve smoothness of the computed unit values of discharge).

Unit value files should be computed with the electronic processing system for the parameters listed below, subject to the above constraints. Unless otherwise noted, each unit value file should be saved for further use and archiving:
(a) Gauge-height corrections – A file of unit values of cumulative gauge-height corrections should be computed and saved for each corresponding unit value of gauge height, as described previously for stage-only stations;
(b) Corrected gauge heights – A file of unit values of corrected gauge heights should be computed by adding the gauge-height corrections to the corresponding unit values of gauge heights;
(c) Rate of change in stage – A rate-of-change in stage (dG/dt) should be computed for each unit value of corrected gauge height that is within the range of gauge heights defined by the minimum and maximum constraint. First, the difference in stage is computed by subtracting the previous unit value of corrected gauge height from the next unit value of the corrected gauge height. This difference in gauge height is converted to the rate-of-change in stage, in feet (or mm) per hour, by dividing it by the time difference of the previous and next unit values. This method of computation provides an average rate-of-change-in-stage for the time period extending one time interval before and one time interval after the current unit value of gauge height. The algebraic sign of the computed rate-of-change-in-stage should be retained as computed. A positive sign indicates a rising stage, and a negative sign indicates a falling stage;
(d) Shift adjustment – For rate-of-change-in-stage stations, shift adjustments are used only for the stage- discharge rating. A unit values file of shift adjustments should be computed for each corrected gauge height by using the defined shift curves and the time and stage interpolation procedures described previously. If shift curves are not applicable for specific time periods, shifts should default to zero for that time period;
(e) Rating discharge – Unit values of rating discharge are computed for each unit value of shift adjusted gauge height using the stage-discharge rating. The rating discharge is an unadjusted discharge value, and does not represent the true discharge of the stream for periods when rate-of-change adjustments are applicable;
(f) Boyer factor, $1/US_c$ – The Boyer Factor should be computed for each corrected gauge height (not shift adjusted) that is within the range of gauge heights defined by the minimum and maximum constraint, by application of the stage versus $1/US_c$ rating;
(g) Discharge adjustment factor – Unit values of the discharge adjustment factor, F_{adj}, are computed based on the Boyer Factor and the rate-of-change-in-stage, by using the following equation. Discharge adjustment factors should be computed only for gauge heights that are within the range of gauge heights defined by the minimum and maximum constraint as:

$$F_{adj} = \sqrt{1 + \left(\frac{1}{US_c}\right)\left(\frac{dG}{dt}\right)} \tag{6.24}$$

(h) Discharge – Unit values of discharge are computed by multiplying the rating discharge times the discharge adjustment factor. All unit values of discharge that are based on adjustment factors from 0.96 to 1.04, by default, should not be used unless overridden or otherwise specified by the hydrographer. Instead, the rating discharges based on the shift adjusted gauge heights should be used directly.

Reservoir stations

Reservoir stations are those stations where unit and daily values of reservoir elevation and reservoir contents are required. If only reservoir elevation is required, no rating is needed. However, if reservoir contents are required, then a rating of reservoir elevation versus contents is needed. Input requires unit values of elevation and information defining elevation corrections. Generally, for reservoir stations, the term elevation is used rather than gauge height because the elevation above a National Geodetic Vertical Datum (NGVD), such as mean sea level is used for many reservoir gauges. However, gauge heights are allowed and used at many reservoir stations. Unit values files should be computed with the electronic processing system for the parameters listed below. Unless otherwise noted, each unit value file should be saved for further use and archiving:

(a) Elevation correction – A file of unit values of cumulative elevation corrections should be computed and saved for each corresponding unit value of elevation, as previously described for stage-only stations;
(b) Corrected elevations – A file of unit values of corrected elevations should be computed by adding the elevation corrections to the corresponding unit values of elevations;
(c) Reservoir contents – A file of unit values of reservoir contents should be computed by application of the corrected elevations to the elevation versus contents rating.

Tide stations

Tide stations are those stations located in estuaries and along tidal affected rivers and streams to provide the daily information on diurnal and/or semi-diurnal variations of surface-water levels in those areas. Tide stations may be set to an arbitrary datum or to an elevation based on a NGVD, such as mean sea level. When an arbitrary datum is used, unit values of elevation are determined by adding a constant datum conversion to the unit values of gauge height. No other conversions to other parameters are required, therefore, no ratings are required. Information defining gauge height or elevation corrections also is required. Each unit value file should be saved for further use and archiving:

(a) Gauge-height or elevation correction – A file of unit values of cumulative gauge height or elevation corrections should be computed and saved for each corresponding unit value of gauge height or elevation as described previously for stage-only stations. This correction value is separate from the datum-conversion value used to convert gauge height to NGVD;
(b) Corrected gauge height or elevation – A file of unit values of corrected gauge heights or elevations should be computed by adding the gauge-height or elevation corrections to the unit values of gauge heights or elevations.

Hydraulic structure stations

Hydraulic structure stations are those stations where unit and daily values of discharge are computed using special ratings and equations for spillways, gates, turbines, pumps, siphons and other controlled conveyances. A special software program, such as the program developed by Sanders and Feaster (2004) is available for this purpose. The basic theory and concepts are described by Collins (1977). Input data may include unit values of headwater gauge heights, tailwater gauge heights, individual gate openings for each gated conveyance, turbine pressures and lockage and other variables as required for a specific site. Hydraulic structure gauging stations are extremely complex and may have many sub-units (individual gates, turbines and others) for which unit values of discharge are computed. Unit values of total discharge are computed as a summation of the individual subunits. Because of the complexity and variability of hydraulic structure gauges, a listing of unit values files will not be given here. However, the electronic processing system should save all unit values files for further use and archiving.

BRANCH model stations

A BRANCH model gauging station utilizes a calibrated digital computer model for simulating the unsteady flow in a channel reach which is usually affected by variable backwater. The model calibration requires basic field data, principally cross-section definition at a number of locations in the gauged reach, roughness coefficients, calibration discharge measurements and gauge-height data at the upstream and downstream end of the gauged reach. Details of calibration and computation are given by Schaffrannek and others (1981). Primary computations require unit values of gauge height at the upstream and downstream ends of the reach, as given below. Information defining gauge-height corrections for the upstream and downstream gauges is required:

(a) Gauge-height corrections, upstream gauge – A file of unit values of cumulative gauge-height corrections for the upstream gauge should be computed and saved for each corresponding unit value of gauge height (including estimated values), as described previously for stage-only stations;
(b) Corrected gauge heights, upstream gauge – A file of unit values of corrected gauge heights for the upstream gauge should be computed by adding the gauge-height corrections for the upstream gauge to the corresponding unit values of gauge heights;
(c) Gauge-height corrections, downstream gauge – A file of unit values of cumulative gauge-height corrections for the downstream gauge should be computed and saved for each corresponding unit value of gauge height (including estimated values);
(d) Corrected gauge heights, downstream gauge – A file of unit values of corrected gauge heights for the downstream gauge should be computed by adding the gauge-height corrections for the downstream gauge to the corresponding unit values of gauge heights.

BRANCH model gauges have a unique characteristic, in that the parameters of gauge height, mean stream velocity and discharge are computed for each cross-section location, as well as at the upstream and downstream gauge locations. For this reason, unit values of each of these parameters, for each cross section, can be saved for future use and archiving. The electronic processing system should allow the hydrographer to designate which output parameters, and for which cross sections and gauge sites, should be saved for future use and archiving.

6.12.2 **Daily value computations**

Various kinds of daily values are computed for each station type, and are based on the unit values files described in the previous sections. Daily values for the various parameters consist of mean values, minimum instantaneous values, maximum instantaneous values and instantaneous values at selected times. Daily values for a gauging station usually are computed for the local time zone designation, for the location of the gauging station. This computation includes the use of daylight savings time wherever applicable. However, the electronic processing system should allow computation of daily values for any other time zone, as selected by the hydrographer.

The electronic processing system should allow the hydrographer to compute daily values for temporary use and study, without requiring that they be saved and archived. Such files of daily values could be used for review and comparisons before finalization of the records.

Daily mean values

Daily mean values, frequently referred to as daily values, consist of a time-weighted arithmetic mean of selected parameters, and are computed from the files of unit values. Daily mean values may be computed for the following parameters:

(a) Gauge height;
(b) Discharge;
(c) Cross-section area (velocity-index stations);
(d) Velocity-index;
(e) Mean stream velocity;
(f) Fall (slope stations);
(g) Elevation (reservoir and tide stations);
(h) Contents (reservoir stations).

A file of all computed daily mean values should be saved for future use and archiving.

The time-weighted arithmetic method of computing daily mean values is referred to as the trapezoidal method. The trapezoidal method is a mathematical integration of the unit value hydrograph and provides an accurate computation of the mean parameter value. With a large number of instantaneous values for each day, the trapezoidal method closely approximates actual integration.

The trapezoidal method assumes that all unit values are instantaneous values, and that each unit value has a specific, designated time of occurrence. The time interval between unit values may be constant or variable. The file of unit values used for the computation of the daily mean value by the trapezoidal method must include a unit value at the midnight time for each day. If actual values are not recorded for the midnight time, a unit value should be interpolated based on the recorded unit values on either side of the midnight time. These interpolated midnight values should be flagged as interpolated, and should be retained in the unit values file for future use and archiving. The equation for the trapezoidal method is:

$$Q = \frac{\left(\frac{q_0 + q_1}{2}\right)(t_1 - t_0) + \left(\frac{q_1 + q_2}{2}\right)(t_2 - t_1)}{t_n - t_0}$$

$$\frac{\cdots + \left(\frac{q_{(n-1)} + q_n}{2}\right)\left(t_n - t_{(n-1)}\right)}{} \quad (6.25)$$

where Q = daily mean parameter value (in the above equation; Q represents discharge; however, the same equation can be used for any other parameter, such as gauge height, velocity and others); q_0 = the parameter unit value at the midnight time at the beginning of the day; $q_1, q_2....., q_{(n-1)}$ = consecutive unit values of the parameter during the day; q_n = the parameter unit value at the midnight time at the end of the day; t_0 = midnight time at the beginning of the day, or zero time; $t_1, t_2......, t_{(n-1)}$ = consecutive times corresponding to the parameter unit values during the day and t_n = midnight time at the end of the day, or 24.00 hour time. Note that all times must be expressed in hours and decimal parts of an hour.

Daily values will not be computed for days when time gaps exceed a value specified as the abort interval. The abort interval, by default, is 2 hours; however, the hydrographer should be allowed to change this interval to any other value less than 24 hours.

Daily minimum and maximum values

The minimum and maximum values for some of the parameters are required for each day. These values are determined from the unit value files for the various parameters and the selection process should consider all recorded and interpolated unit values for each day, including the midnight values at the beginning and end of each day. For some parameters corresponding values of other parameters also should be determined.

Daily values at selected times

Some stations require additional daily values at selected times for some parameters. For instance, reservoir stations sometimes require daily elevation and contents at specific times, such as 0800, 1200 or 2400. If unit values are not available at the specified times, interpolated values should be used. The hydrographer should be able to specify, for all station types, the parameter and time for which selected daily values are required.

Daily values for tidal stations

Tidal stations require the determination of the gauge heights or elevations of tidal peaks and troughs for diurnal and semi-diurnal variations of the water-surface level. The unit values file of the corrected gauge height or elevation data are examined sequentially to determine the two high tides and the two low tides for each day for semi-diurnal fluctuations. The procedure for computing daily values for tidal stations also recognizes diurnal and mixed fluctuations when they occur. The following discussion is excerpted from Hutchinson and others (1977):

> "In order to find true tidal peaks and troughs which occur once or twice in relation to the lunar day rather than the solar day, the record is NOT broken up into groups of observations in a calendar day before processing. Instead, the whole record is scanned continuously for successive peaks and troughs within periods of given length following the time of the previous extreme. After each extreme is found, the calendar day in which it occurred and time is determined. This completely eliminates any confusion with inclusion or exclusion of extremes occurring just before or just after midnight.
>
> The method of finding successive tidal peaks and troughs is to look for an opposite extreme in a selected time period (normally 10 1/2 hours) following each recognized peak or trough. That is, when a tidal peak is found (and its date and time are stored) a search is made for the lowest stage in the selected time period following the time of the previous tidal peak. Then having found the time of this tidal trough, a search is made for the highest stage in the selected time period following the time of the previous tidal trough. Comparison of two peaks found within a calendar day and two troughs found within the same calendar day are used to assign each as a HIGH-HIGH, a LOW-HIGH, a HIGH-LOW, or a LOW-LOW for the day.
>
> Although the normal tide on most of the United States coastline is semi-diurnal, at a few places the tides are diurnal or are mixed semi-diurnal. This program tries to give meaningful results in a situation by the following logic. Starting each search for a peak or trough, a normal, semi-diurnal tidal cycle is assumed and the length of the selected time period for the search is set at about 10-1/2 hours (0.44 day). This length of the search period was picked so as to be long enough to include the normal time of occurrence of the next peak or trough for a semi-diurnal tide (which should occur about 6-1/2 hours after the preceding trough or peak) and short enough to avoid confusion with the advance side of the next following tidal wave if the two tidal waves are of greatly different magnitude. (If a 12-hour search period were used, confusion could occur such as when the second tidal wave of the day is so much higher than the first that the water level 12 hours after the previous tidal trough is rapidly rising and already higher than it was at the time of the first real peak which occurred about 6-1/2 hours after the previous tidal trough).

In order to be able to produce meaningful results for sites where the tide is actually diurnal or is a mixture of semi-diurnal and diurnal, an additional test is made after each search for the next apparent extreme. If the next extreme is found to occur in the last hour of the 10-1/2 hour search period, it is assumed that this extreme is not a true tidal peak or trough in a semi-diurnal cycle but is instead falling toward a trough or rising toward a peak in a diurnal tidal cycle. Then in order to find the real tidal peak or trough in this longer cycle, that particular search period is extended by another 12 hours and the new results used as the next peak or trough. However, after finding the next tidal peak or trough, the following search is again made for an initial period of 10-1/2 hours so that a change back from a diurnal tide to a semi-diurnal tide is not missed."

The daily values of HIGH-HIGH, LOW-HIGH, HIGH-LOW, and LOW-LOW determined in the above procedure should be saved for further use, and for archiving. In addition, the cumulative elevation correction values corresponding to each of the peak and trough elevations should be saved and archived.

There are special considerations for computations of mean daily discharge for tidally affected sites. Ruhl and Simpson (2005) discuss these considerations for several stations where the index-velocity method was used to compute discharges at tidally affected gages. Calculating daily discharge in a tidally influenced environment cannot be accomplished simply by averaging all of the values collected during that 24-hour period. Simple averaging causes cyclical variations, or aliasing, in the data that are spurious and are a function of the averaging scheme, not the data. Therefore, a low-pass filter is used to remove frequencies that have periods less than 30 hours. The most energetic variations removed in this process are the astronomical tides (typically with periods at or around 12 and 24 hours); however, other variations (meteorological, hydrologic, or operational) that have periods less than 30 hours also are removed. These considerations are discussed in more detail in Chapter 2 of this Volume and will not be repeated here.

6.12.3 **Summary of primary computations**

Primary computations include the determination of unit values and daily values for numerous parameters. It is important and necessary to summarize these results in tables that can be used for review, analysis and publication. Standard formatted tables include unit values, primary computations and daily value tables. The electronic processing system should allow for the design of other summary tables, as needed, and as specified by the hydrographer.

Unit values tables

The electronic processing system should provide a flexible array of unit values tables to allow for the analysis and review of individual parameters, or selected groups of parameters. For instance, a unit values table may show only the final, corrected values of gauge height for a selected period of time; or the unit values table may show the final gauge-height values and the corresponding discharge values. The hydrographer should select the input parameters needed in a unit values table. The unit values should be displayed in chronological order and generally grouped by day, month and year. The hydrographer also should specify selected time intervals for a unit values table. For instance, an hourly table may be selected, even though 15-minute unit values are available or, even-hour unit values may be selected that require interpolation of unit values that are not recorded on the even-hour.

Primary computations tables

Primary computations involve the application of various instructions to derive the final discharge record (or other parameter such as reservoir contents, tide, and others) for a gauging station. These instructions include gauge-height corrections, shifts and rating curves. The computations should be displayed in a table that shows input data and computed information so that they can be easily reviewed. Each gauging station type, such as stage-discharge, slope, velocity-index and others, will have primary output formats specifically designed for the station type. A listing of items, by station type, recommended for inclusion in a primary computation form is shown in Table II.6.1. Arrangement of the information is not critical.

Daily values tables

A daily values table is a listing of the daily values for each day of the year for selected parameters at a gauging station. Generally, daily values are the daily mean discharges for a gauging station, but other parameters such as stage, elevation, reservoir contents or other statistics such as daily maximum, daily minimum and daily unit value at a specific time may compose a daily values table. The hydrographer should specify time periods in the daily values table and include multiple parameters in one table. In addition, the daily values table should show monthly

Table II.6.1. Items recommended for inclusion in primary output tables for various gauging station types

Item	Station Type								
	1	2	3	4	5	6	7	8	9
HEADER INFORMATION									
Station ID number	X	X	X	X	X	X	X	X	X
Station name	X	X	X	X	X	X	X	X	X
Water year information	X	X	X	X	X	X	X	X	X
Date of primary processing	X	X	X	X	X	X	X	X	X
Name of responsible hydrographer	X	X	X	X	X	X	X	X	X
List of ratings used		X	X	X	X	X			
Unit value recording interval	X	X	X	X	X	X	X	X	X
Station type (i.e., processing method)	X	X	X	X	X	X	X	X	X
Datum adjustment (if applicable)	X	X	X	X	X	X	X	X	X
TABULAR INFORMATION									
Date	X	X	X	X	X	X	X	X	X
Hourly gauge heights for base gauge	X	X			X	X	X		
Maximum daily gauge height	X	X	X	X	X	X			X
Time of maximum gauge height	X	X	X	X	X	X			X
Shift corresponding to maximum gauge height		X		X	X				
Gauge height correction corresponding to max. ght	X	X	X	X	X	X			X
Minimum daily gauge height	X	X	X	X	X	X			X
Time of minimum gauge height	X	X	X	X	X	X			X
Shift corresponding to minimum gauge height		X		X	X				
Gauge height correction corresponding to min. ght	X	X	X	X	X	X			X
Mean daily gauge height	X	X	X	X	X	X			X
Maximum daily discharge		X	X	X	X			X	X
Time of maximum discharge		X	X	X	X			X	X
Minimum daily discharge		X	X	X	X			X	X
Time of minimum discharge		X	X	X	X			X	X
Mean daily discharge		X	X	X	X			X	X
Hourly discharges			X	X				X	X
Maximum daily index velocity			X						
Time of maximum index velocity			X						
Shift corresponding to max. index velocity			X						
Index velocity correction for max. index velocity			X						
Minimum daily index velocity			X						
Time of minimum index velocity			X						
Shift corresponding to min. index velocity			X						
Index velocity correction for min. index velocity			X						
Mean daily index velocity			X						
Maximum daily cross section area			X						
Time of maximum cross section area			X						
Minimum daily cross section area			X						
Time of minimum cross section area			X						

Gauging station type:
1 - Stage only; 2 - Stage-discharge; 3 - Velocity index; 4 - Slope; 5 - Rate-of-change in stage; 6 - Reservoir.

Table II.6.1 (*continued*)

Item	Station Type								
	1	2	3	4	5	6	7	8	9
Mean daily cross section area			X						
Maximum daily stream velocity			X						
Time of maximum stream velocity			X						
Minimum daily stream velocity			X						
Time of minimum stream velocity			X						
Mean daily stream velocity			X						
Maximum daily reservoir contents						X			
Time of maximum reservoir contents						X			
Minimum daily reservoir contents						X			
Time of minimum reservoir contents						X			
Mean daily reservoir contents						X			
Reservoir gauge height at specified time						X			
Gauge height correction at specified time						X			
Reservoir contents at specified time						X			
High-high daily gauge height w/o datum adj.							X		
High-high gauge height w/datum adj.							X		
Time of high-high							X		
Gauge height correction for high-high							X		
Low-high daily gauge height w/o datum adj.							X		
Low-high gauge height w/datum adj.							X		
Gauge height correction for low-high							X		
High-low daily gauge height w/o datum adj.							X		
High-low gauge height w/datum adj.							X		
Time of high-low							X		
Gauge height correction of high-low							X		
Low-low daily gauge height w/o datum adj.							X		
Low-low gauge height w/datum adj.							X		
Time of low-low							X		
Gauge height correction for low-low							X		
Mean daily tide gauge height w/o datum adj.							X		
Mean daily tide gauge height w/datum adj.							X		
Maximum daily gauge height at auxiliary gauge				X					
Time of max. daily ght				X					
Ght correction corresponding to max. aux ght				X					
Minimum daily gauge height at auxiliary gauge				X					
Time of min. daily ght				X					
Mean daily ght at auxiliary gauge				X					
Maximum daily fall				X					
Time of max. daily fall				X					
Minimum daily fall				X					
Time of min. daily fall				X					
Maximum rate of change in stage					X				
Time of max. rate of change in stage					X				
Maximum adjustment factor					X				
Time of max adjustment factor					X				
Minimum adjustment factor					X				
Time of min. adjustment factor					X				

Gauging station type:
1 - Stage only; 2 - Stage-discharge; 3 - Velocity index; 4 - Slope; 5 - Rate-of-change in stage; 6 - Reservoir.

and annual totals, means and extremes, as appropriate.

6.13 HYDROGRAPH PLOTS

Hydrographs are useful for graphical viewing, verification, editing and comparisons of streamflow information, including most of the basic information that contributes to the primary computation of streamflow records. Hydrograph plots of unit values of discharge along with comparative plots of other parameters such as gauge height, velocity and shifts, and supplementary data such as peak discharge, peak stage and discharge measurements, provide an excellent means of reviewing and editing primary computations. Likewise, hydrograph plots of daily discharge records can be combined with hydrograph plots of other station records, precipitation records and temperature records for estimating missing records. Hydrograph plots provide a graphical summary of the records for visual presentation and publication.

All hydrograph plots, both unit value and daily value, should be viewable on the computer monitor. In addition, the hydrographer should have the option to plot all hydrographs on paper plots. All scales and grid lines should be generated by the electronic processing system. Preprinted plotting forms are not advised.

6.13.1 Unit value hydrographs

The electronic processing system should allow the hydrographer to choose any of the unit values files for hydrograph plotting. Generally, hydrographs showing unit values of discharge will be of most interest, but other unit values hydrographs, such as gauge height, elevation and reservoir contents also may be required. Other unit values files of supplementary information, such as for shifts, gauge-height corrections, auxiliary gauge information and others should be superimposed on the same plot if these additional parameter plots are specified. Also, unit value information from other gauging stations, precipitation stations and temperature stations should be superimposed on the same plot, as specified by the hydrographer.

When more than one unit values file is shown on a unit values hydrograph plot, each should be clearly identified by a distinctive plotting symbol. Individual scales should be shown for each parameter, labeled with the correct parameter name and units of measurement.

The abscissa scale for a unit values hydrograph plot is time, with hours being the primary unit of subdivision. Each day, month and year are shown as secondary subdivisions. The ordinate scale should conform to the parameter being plotted. Discharge scales should default to logarithmic but should be changeable to linear if specified. All other scales, such as for gauge height, elevation, shifts, rainfall, temperature and others, should default to linear scales. The range of the ordinate scale should default to one that will include the full range of the plotted unit values file but should be changeable to any specified range.

6.13.2 Daily value hydrographs

A daily values hydrograph is one of the most common methods for displaying the results of streamflow computations for a gauging station. This hydrograph usually is an annual plot showing the daily values for a water year but can be for any other period of time. Daily value hydrographs usually are plots of daily mean discharge for a gauging station, with comparative hydrograph plots of daily mean discharge for one or more nearby gauging stations. For some stations, the daily values hydrograph also may include daily values of precipitation and/or temperature. Daily values hydrographs also can be used to display other parameters, such as gauge height, elevation and reservoir contents.

When more than one daily values file is shown on a daily values hydrograph plot, each should be clearly identified by a distinctive plotting symbol. Individual scales should be shown for each parameter, labeled with the correct parameter name and units of measurement.

The abscissa for daily values hydrographs is a time scale, with days being the primary subdivision. Months and years are secondary subdivisions. The ordinate should be logarithmic for discharge plots, unless otherwise specified by the hydrographer. Other daily values parameters should be plotted using linear scales. The range of the ordinate scale for the primary parameter should default to one that will include the full range of the daily values for the time period being plotted.

6.14 COMPUTATION OF EXTREMES

For most discharge gauging stations it is required that the maximum peak stage and discharge, the secondary peak stages and discharges and the minimum discharge be computed for each water year. The maximum peak stage and discharge, and the minimum discharge are referred to as the annual peak and annual minimum. Guidelines for these computations are given in the following sections.

6.14.1 Annual peak stage and discharge

The annual peak stage and discharge are defined as the highest instantaneous (unit value) gauge height and discharge associated with the highest flood peak that occurred during the water year. The annual peak stage and discharge and the associated date and time, should be determined with the electronic processing system. If the highest gauge height and discharge was at the beginning or end of the water year as a result of a recession from or rise to a peak that occurred in the previous or following water year, they should not be included as an extreme. For some gauging stations, the hydrographer may designate that the maximum daily discharge be used rather than the maximum instantaneous discharge.

The annual instantaneous maximum gauge height may sometimes occur at a different time than the annual instantaneous maximum discharge. In these cases, the annual maximum instantaneous discharge should be determined and the gauge height corresponding (at the same date and time) to this discharge. In addition, the annual maximum instantaneous gauge height should be determined and also the discharge corresponding (at the same date and time) to this gauge height. Dates and times for both pairs of values should be determined.

6.14.2 Secondary peak stages and discharges

Secondary peak stages and discharges are those peaks that are less than the annual peak stage and discharge, but greater than a specified base discharge. Furthermore, the secondary peaks must conform to guidelines that insure their independence. That is, to provide reasonable certainty that a peak has not been influenced, or affected, by another peak. These guidelines are described by Novak (1985) and are given as follows:

> "Two peaks are considered independent if the hydrograph recedes to a well-defined trough between the peaks. Publish both peaks if the instantaneous discharge of the trough is equal to or less than 75 per cent of the instantaneous discharge of the lower peak; otherwise publish only the higher peak.
>
> For small, highly responsive watersheds, only the highest peak discharge resulting from an obvious single storm event should be reported regardless of the trough configuration or magnitude between peaks.
>
> For periods of diurnal peaks caused by snowmelt, report only the highest peak during each distinct period of melting, if such periods can be identified, even though other peaks may meet the preceding criteria. Identification of each distinct period of melting is largely a matter of individual judgment, but the principle as explained in paragraph 1 above for instantaneous discharges can be applied to daily discharges as an identification guide."

All secondary peak stages and discharges should be determined with the electronic processing system. In addition, the date and time for each secondary peak should be determined.

6.14.3 Annual minimum discharge

The annual minimum discharge is defined as the lowest instantaneous (unit value) discharge that results during the water year. For some gauging stations, the hydrographer may specify that the lowest daily discharge be determined as the annual minimum discharge. In either case, the electronic processing system should determine the annual minimum discharge and the associated date and time (if applicable) for the water year.

6.15 ESTIMATING MISSING RECORDS

Complete records of daily discharges, and other parameters, are necessary in order to compute monthly and annual totals and other statistics. Complete records also are needed to compute total runoff from a drainage basin, to calibrate runoff models and to compute chemical and sediment loads. Data sometimes are missing because of instrument failures and other reasons, thus not permitting the normal computation of daily records. Also, normal computation methods may not be applicable at times such as during backwater from ice, debris or other abnormal stream conditions. Therefore, it is necessary to make estimates of discharge or other hydrologic parameters for these periods of missing record.

The electronic processing system should allow the hydrographer to estimate both unit values and daily values. However, estimation of missing records should be kept to a minimum, and usually should be limited to those parameters that will be published and to those parameters that may be required for the purpose of computing a published parameter.

For example, in some cases it may be reasonable to estimate unit values of gauge height for the purpose of computing daily values of discharge, provided the gauge heights can be estimated with reasonable accuracy. The electronic processing system should provide estimating methods that commonly are accepted but the hydrographer must be able to interact and apply unique site specific information and procedures in order to make the best estimate of missing records. Several estimating techniques are described in the following sections.

6.15.1 Hydrographic and climatic comparison method

The hydrographic and climatic comparison method, as described in Chapter 1 of this Volume, is the most common method used to estimate discharge during periods of missing record and ice-affected periods. A semi-logarithmic hydrograph of daily discharge is plotted, encompassing the period of missing record, and valid records for periods prior to and after the missing record period. Other data and information, as shown below, may be superimposed on this plot to aid in the estimation procedure:

(a) Hydrographs of nearby stations (reference sites);
(b) Hydrographs based on the direct application of ice-affected gauge heights to the rating (without correction for ice-induced backwater);
(c) Daily or hourly precipitation;
(d) Daily temperature, and/or daily maximum and minimum temperatures;
(e) Discharge measurements;
(f) Recession curves for the station being estimated;
(g) Notes and observations (for example, observed ice conditions).

The electronic processing system should allow vertical and horizontal repositioning of the hydrograph of the reference site (or sites) until it corresponds as closely as possible to the available good record of the site to be estimated. When long periods of missing record must be estimated, this repositioning process may need to be performed various times, each time for a different segment of the missing period. Values of daily mean discharge are then estimated by using the reference site as a guide and drawing a hydrograph for the missing period, taking into account all of the other available data and information, such as the discharge measurements, climatic data and notes. This estimation process is performed by the hydrographer on the electronic processing system monitor, After the estimated hydrograph segment is completed and accepted by the hydrographer, the electronic processing system automatically should determine the daily values of discharge, flag the values as estimated, and insert them into the daily values file.

A period of missing record resulting during an unbroken recession can be estimated by connecting the adjacent periods of good record with a straight line or a smooth recession curve on a semi-logarithmic plot. This procedure is improved if recession curves, within the range of discharge to be estimated, are available for the station in question to superimpose on the plot. Recessions also may vary by season; therefore, it is useful to categorize the recession curves by season of the year. The hydrographer should be able to re-position the recession curves vertically and horizontally to obtain the best fit of the recession curves. The electronic processing system should allow for the storage, and later recall, of recession curve data for this purpose.

6.15.2 Discharge ratio method

The discharge ratio method is used for estimating discharge during ice-affected periods and is described in Chapter 1 of this Volume. The electronic processing system should automatically display computed correction factors, K, for each discharge measurement on a semi-logarithmic plot, along with the equivalent open-water daily discharge hydrograph and the climatic data. The hydrographer should define the interpolation between computed K values, and the electronic processing system should then compute daily values of discharge based on the interpolated K values and the open water discharges. Daily values files of the open-water discharge and the corresponding correction factors, K, should be saved and archived for all ice-affected periods.

6.15.3 Regression method

Multiple, stepwise, regression is a useful method of relating time series discharge data of one gauging station to concurrent time series discharge data of a nearby reference gauge(s). Regression equations can be developed for specific ranges of discharge, for instance, low flows, medium flows and/or high flows. They also can be developed for seasonal periods and for ice-affected periods. The electronic processing system should provide a flexible method of developing regression equations, allowing the hydrographer to specify reference gauge records, time periods and discharge ranges. The regression equations should include the ability to time-lag reference gauge records, and to use transformations of discharges (for example, logarithmic). Also, developed regression equations and their associated limitations should be documented and archived for later use.

A useful addition to the regression method is to plot the 95 per cent confidence limits about the predicted discharges. When the observed discharges fall outside the confidence limits for several days or more, there is an indication of error in the discharge record, such as an incorrect shift analysis. This assumes, of course, that there is a strong correlation between the base gauge and the reference gauge.

A regression equation can be applied to provide estimated discharges for periods of missing record. In addition, the same regression equation should be used to compute discharge values for short time periods adjacent to the estimated period where discharges are known. These adjacent periods sometimes can be used for verifying the accuracy of the regression results, and for adjusting the estimated discharges during the period of missing record to more closely fit the adjacent known records.

6.15.4 **Water-budget method**

Records missing for a gauging station just upstream from a reservoir, for the purpose of measuring inflow to the reservoir, can be estimated using the water-budget method if accurate records are available for the outflow from the reservoir and the change in contents of the reservoir. The daily inflow to the reservoir is equal to the daily outflow plus or minus the change in reservoir contents. In some cases, where the flow at the inflow station may not represent the total inflow to the reservoir, an adjustment may be required. The adjustment may be simply the application of a drainage area ratio, or other multiplication factor supplied by the hydrographer. The adjustment factor can also be estimated by applying the water-budget equation during periods when inflow, outflow and storage records are all available. The water-budget method is:

$$Q_i = K(Q_o + \Delta C) \qquad (6.26)$$

where Q_i = flow at inflow gauge; Q_o = outflow from reservoir; K = inflow adjustment factor and ΔC = change in contents of reservoir, computed as midnight contents on current day minus midnight content on previous day.

The same principle can be used to estimate missing outflow records for gauging stations located just downstream from a reservoir. Equation 6.26 simply is rearranged to solve for outflow, Q_o.

6.15.5 **Mathematical translation method**

The mathematical translation method is a set of various mathematical functions that can be used to translate streamflow records for other gauging stations (referred to as reference gauges) into estimates of streamflow for the gauge site where missing records result. Some of these functions are similar to the regression method described previously but are defined independently from regression methods. The selection of reference gauges to use for making an estimate is important because the reference stations should be hydrologically related to the station for which estimates are made. For this reason reference stations usually are nearby stations, have similar runoff characteristics and are sometimes stations on the same stream. The hydrographer should use considerable care and judgment in selecting stations to use with the mathematical translation method. This method includes the following mathematical functions:

(a) Combining two streamflow records by addition, subtraction, multiplication or division;
(b) Transforming a streamflow record into a different record using:

$$Q_e = a + b\left(Q_r + c\right)^d \qquad (6.27)$$

where Q_e = estimated discharge; Q_r = discharge at reference gauge and *a, b, c*, and *d*, are constants defined by the hydrographer;
(c) Offsetting a reference gauge record by a specified time period. The offset record can be mathematically combined with another reference record or can be transformed by an equation. Two or more reference records can be offset with the same, or different, offsets;
(d) Transformation of reference gauge records into log10 and inverse log10. These transformations can be made prior to performing any of the above mathematical functions.

6.15.6 **Flow routing methods**

Various flow routing models can be used to route a streamflow record from a reference gauge to a downstream location on the same stream, thereby providing an estimate of the flow at a downstream gauge site. However, these models are used external to the electronic processing system, and the results must be imported to the streamflow data base. Generally, it is not expected that such models will be used very often for estimating streamflow records because of the complex and intense efforts needed for calibration and application.

6.16 MONTHLY AND ANNUAL VALUE COMPUTATIONS

Monthly and annual values of stage, elevation, discharge, runoff, reservoir contents and tidal lows and highs should be computed for each station as required or designated. The required and designated monthly and annual values will vary with station type and with specific stations. All computations of monthly values should be based on the rounded results of daily values and all computations of annual values should be based on rounded results of either daily or monthly values, as indicated. Therefore, consistent agreement results amount the daily, monthly and annual values.

At least two sets of annual values should be computed for each gauging station: (a) for the calendar year, January through December and (b) for the water year, October through September. In special cases the hydrographer may designate additional or alternative types of years, such as the climatic year, April through March.

6.16.1 Monthly and annual values of stage

Monthly and annual values of stage should be computed for those stations where stage routinely is measured for defining the gauge-height fluctuations of a stream. For some stations, the stage may be the primary end product, such as for a stage-only station. In other instances the stage may be measured for the purpose of computing other parameters, such as discharge.

The monthly stage values that should be computed are the following:
(a) Monthly mean stage – The arithmetic mean of all daily mean stages for each month;
(b) Monthly minimum daily stage – The lowest daily mean stage for each month;
(c) Monthly maximum daily stage – The highest daily mean stage for each month.

The annual stage values that should be computed are the following:
(a) Annual mean stage – The arithmetic mean of all daily mean stages for the water year and calendar year;
(b) Annual minimum daily stage – The lowest daily mean stage for the water year and calendar year;
(c) Annual maximum daily stage – The highest daily mean stage for the water year and calendar year.

6.16.2 Monthly and annual values of discharge

Monthly and annual values of discharge should be computed for gauging stations where daily discharge is routinely computed and where streamflow is the parameter of primary interest. Some of the monthly and annual values are required, whereas others are optional, and are computed only for specific gauging stations. The optional computations generally are designated on the basis of streamflow conditions, drainage basin size, natural runoff conditions, degree of regulation and other factors that may affect the hydrologic value and need for the computed parameters.

The monthly discharge values that are required are the following:
(a) Monthly total discharge – Total of all daily mean discharges for each month;
(b) Monthly mean discharge – The mean of all daily mean discharges for each month, and is computed by dividing the monthly total discharge by the number of days in the month;
(c) Monthly minimum daily discharge – The lowest daily mean discharge for each month;
(d) Monthly maximum daily discharge – The highest daily mean discharge for each month.

The monthly discharge values that are optional are as follows:
(a) Monthly runoff volume – This is the monthly total discharge, converted to a volume;
(b) Monthly runoff depth – The monthly total discharge volume, converted to a depth, in inches or millimetres, that would uniformly cover the drainage basin;
(c) Monthly mean unit runoff – The monthly mean flow that would emanate from 1 mi^2 or 1 km^2 of drainage area, if the flow were uniformly distributed throughout the drainage basin.

The annual discharge values that are required are as follows:
(a) Annual total discharge – The total of all daily mean discharges for the year;
(b) Annual mean discharge – The mean of all daily mean discharges for the year, and is computed by dividing the annual total discharge by 365 or by 366 for leap years;
(c) Annual minimum daily discharge – The lowest daily mean discharge for the year;
(d) Annual maximum daily discharge – The highest daily mean discharge for the year.

The annual discharge values that are optional are as follows:
(a) Annual runoff volume – The annual total runoff volume is computed by summing the monthly values of runoff volume for the year;
(b) Annual runoff depth – The annual total runoff depth is computed by summing the monthly values of runoff depth for the year;
(c) Annual mean unit runoff – The annual mean unit runoff is computed by dividing the annual mean discharge by the drainage area.

6.16.3 Monthly and annual values for reservoirs

The computation of monthly and annual values for reservoir stations is varied and highly dependent on the type of daily values that are used for the station. Reservoir stations may require daily mean elevations, daily mean contents or elevation or contents at a specific time, The choice of daily values that are used and published for a reservoir station is dependent on hydrographer requirements, and consequently, the monthly values that should be computed will be based on these.

6.16.4 Monthly and annual values for tidal stations

Tidal stations require the computation of various monthly and annual values as described below. For tidal stations that use an arbitrary gauge-height datum and a datum-conversion constant to convert the gauge heights to national datum, such as mean sea level, the monthly and annual values should be computed for both datums.

The monthly tide values that may be computed are as follows:
(a) Monthly mean stage and/or elevation – The mean of all daily mean stages and/or elevations for each month;
(b) Monthly mean high tide – The mean of all daily HIGH-HIGH tide stages and/or elevations for each month;
(c) Monthly mean low tide – The mean of all daily LOW-LOW tide stages and/or elevations for each month;
(d) Monthly minimum low tide – The lowest of all daily LOW-LOW tide stages and/or elevations for each month;
(e) Monthly maximum high tide – The highest of all daily HIGH-HIGH tide stages and/or elevations for each month.

The annual tide values that may be computed are as follows:
(a) Annual mean stage and/or elevation – The mean of all daily mean stages and /or elevations for the year;
(b) Annual mean high tide – The mean of all daily HIGH-HIGH tide stages and/or elevations for the year;
(c) Annual mean low tide – The mean of all daily LOW-LOW tide stages and/or elevations for the year;
(d) Annual minimum low tide – The lowest of all daily LOW-LOW tide stages and/or elevations for the year;
(e) Annual maximum high tide – The highest of all daily HIGH-HIGH tide stages and/or elevations for the year.

6.17 STATION ANALYSIS DOCUMENTATION

The station analysis documentation is a narrative description of the methods used to analyze the gauging station records for a water year or other period of analysis. The analysis includes information about station equipment, performance of the gauge and related equipment, the rating, shifting control methods, computation of discharge, accuracy and any other information about how the station records were produced. The station analysis is one of the most important documents produced for each year of gauging station records because it is the primary documentation for quality assurance and quality control of these records.

The station analysis for a gauging station usually is written and finalized at the end of each water year, however, parts of it may be written at any time during the year as information becomes available. The electronic processing system should provide some form of record processing notebook that can be utilized as an aid in writing the station analysis. The electronic processing system automatically should transfer information from the record processing notebook to the appropriate paragraphs of the station analysis.

The electronic processing system also should automatically transfer information from other parts of the electronic processing system to the station analysis to assist the hydrographer. These include the following:
(a) Level and datum information from the most recent level summary. This information should include the date of the latest levels, and information about datum differences of the various gauges at the station;

(b) All periods (dates) of missing record, and the total number of days of missing record from the unit values files;
(c) The minimum and maximum gauge heights recorded during the water year from the unit values files;
(d) The number of discharge measurements made during the water year, and their corresponding sequence numbers, from the measurement file. In addition, the lowest and highest measured gauge height and discharge from the measurement file;
(e) The comparison of measured discharges to computed unit values of discharge from the measurement file and the primary computations;
(f) Methods of estimating missing records and ice records from the electronic processing system documentation of estimating missing records for the water year;
(g) The listing of records used for hydrograph comparisons from the electronic processing system documentation of hydrograph comparisons used for the water year. This listing should include station names, parameters compared and periods of record compared;
(h) The sequence numbers for the rating curves and the shift curves used during the water year from the rating curve file and the shift curve file;
(i) Any information relative to quality control from field notes, record processing notebook and comment files that have been documented in the electronic processing system.

The transferred information, both from the record processing notebook and the various other parts of the electronic processing system, can then be used to write the station analysis. The station analysis should include, at a minimum, the following items and paragraphs. For some gauging stations, other paragraphs may be required in order to adequately describe the computation methods. For example, for velocity-index stations there should be a velocity record section. See Kennedy (1983) for additional details:
(a) Station name;
(b) Station ID;
(c) Water year;
(d) Equipment;
(e) Gauge-height record;
(f) Gauge-height and datum corrections;
(g) Rating;
(h) Discharge;
(i) Quality assurance and control;
(j) Remarks;
(k) Recommendations.

The name of the hydrographer who writes the station analysis and the date of preparation automatically should be attached to the end of the station analysis. Also, the name of the reviewer automatically should be attached, along with the date of review completion.

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