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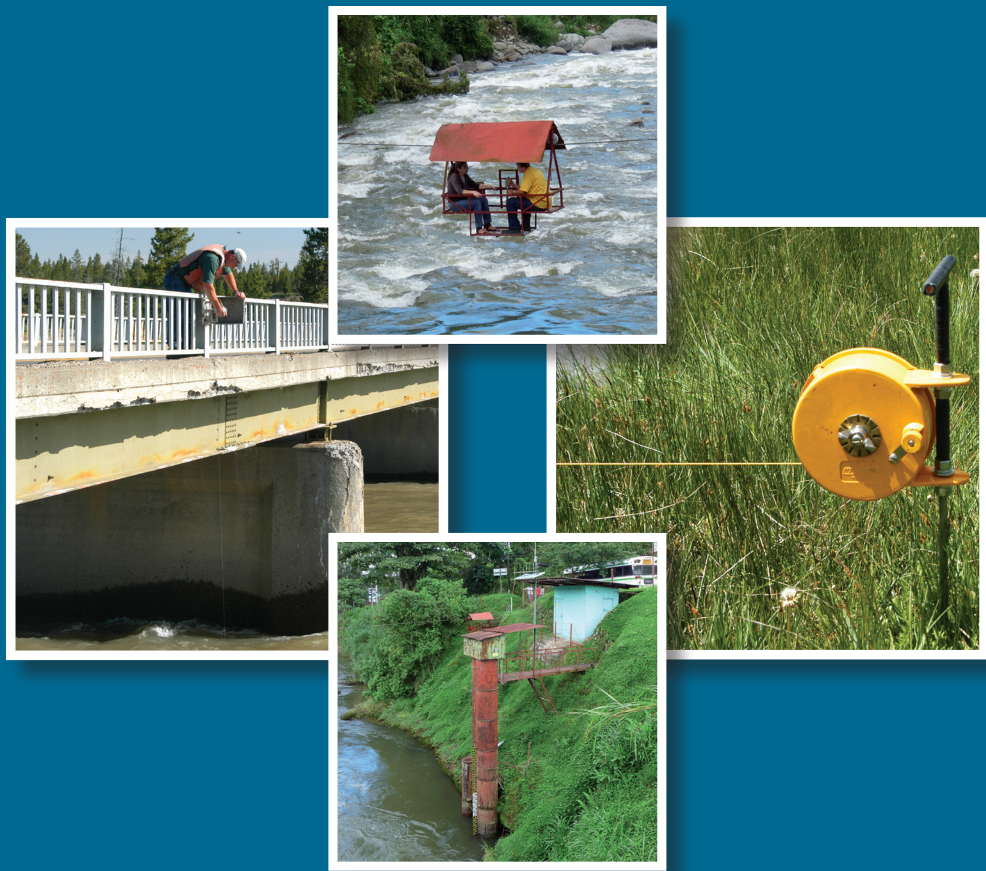

MANUAL ON STREAM GAUGING

VOLUME I – FIELDWORK

World
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Weather • Climate • Water

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Volume I – Fieldwork

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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.

CHAPTER 1

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 2

SELECTION OF GAUGING STATION SITES

2.1 INTRODUCTION

The selection of gauging sites is dictated by the needs of water management or the requirements of the hydrologic network. Those needs and requirements dictate locations of the gauging sites, but the process of general site selection is dependent on the specific purpose of the streamflow record. For example, the general location of a gauging station is immediately fixed if the streamflow record is needed for the design of a dam and reservoir at some specific site, or if the record is needed for operating or managing a completed project. However, for a regional water resources inventory and for the formulation of a long-range water development plan, a network of gauging stations must be established to study the general hydrology of the region. Judicious thought is then required in selecting the general location of gauging stations to ensure optimum information is obtained for the money spent in data collection. To augment the network of continuous record gauging stations to study regional flood-frequency a network of crest stage gauges (see Chapter 4, section 4.7) may be established. Judicious thought is required in selecting the general location of crest stage gauges to ensure an adequate sampling of the many possible combinations of climatic and physiographic characteristics in the region. The general aspects of network design are discussed in section 2.2.

As mentioned earlier, once the general location of a gauging station has been determined, its precise location is selected to obtain the best locally available conditions for stage and discharge measurement and for developing a stable discharge rating. The hydraulic considerations that enter into specific site selection are discussed in section 2.3.

2.2 GAUGING STATION NETWORK DESIGN

The procedures for network design are described in the following sections. Additional information on network design can be found in the *Guide to Hydrological Practices (*WMO-No. 168), sixth edition (2008) and in Rantz (1982).

2.2.1 Network design for water resources development

Two types of gauging stations are usually needed in a stream gauging network designed for a regional study of water resources development, namely, hydrologic and special stations*.

(a) Hydrologic stations are those established to determine the basic streamflow characteristics of the region. They are established only on streams that have natural flow, or on developed streams whose records can be adjusted, by the use of auxiliary records, for such manmade effects as diversion, storage and import of water. There are two types of hydrologic stations – principal and secondary:
    (i) Principal hydrologic stations (called primary stations in some countries) are intended to be operated permanently; in showing time trends they furnish data of importance to regional hydrologic studies;
    (ii) Secondary hydrologic stations are intended to be operated only long enough to establish the flow characteristics of their watersheds, relative to those of a watershed gauged by a principal, or primary, hydrologic station. The length of time that a secondary hydrologic station is operated is dependent on how well its records correlate with those of a principal station. The better the correlation, the shorter the required period of operation. It should be noted that where full range measuring structures are employed in the network they are generally, by definition and cost, regarded as permanent stations.
(b) Special stations are those established to provide specific information at a site for one or more of the following purposes:
    (i) Inventory the outflow from a basin, either developed or undeveloped, to show the quantity of water potentially available for later development. If the station measures natural flow, it may be operated for a definite time period, the length of which is dependent on the degree of correlation with a principal hydrologic station;

* The networks in many countries include principal and special stations only.

(ii) Management and operation of an existing project, A station to be operated during the entire life of the project;
(iii) Fulfilment of legal requirements, agreement, or basin compact. A station to be operated during the entire life of the legal requirement;
(iv) Design of a proposed project. If the station measures natural flow, it may be operated for a definite term that is dependent on the degree of correlation with a principal hydrologic station.

In designing the network, all types of stations should be considered simultaneously, because both principal and secondary hydrologic stations may often be strategically located to also serve as a special station. For example, a hydrologic station, by virtue of its location, may provide the data for one or more of the special purposes listed above.

A minimum network should include at least one principal hydrologic station in each climatologic and physiographic province in a region, because the runoff characteristics of streams are related to the climatologic, topographic and geologic characteristics of the basins they drain and can be highly variable. The annual volume of runoff of a stream may be large or small and may be derived from storm runoff, or snowmelt runoff, or a combination of both. Daily discharge rates may be highly variable or relatively steady. The streamflow may be ephemeral, intermittent, or perennial, and if perennial, base flow may be well or poorly sustained. A stream that flows through more than one province should be gauged at the boundary between provinces. An obvious example is a stream that flows through a mountain province and enters an alluvial valley province; commonly, the runoff of the two provinces will differ greatly.

It is also desirable that there be one or more principal hydrologic stations in a basin. The total number of stations depends on basin size, the number of key sub-basins, and the potential for future land use change. Basin size is a factor because principal hydrologic stations will be correlated with secondary hydrologic stations, and as a rule-of-thumb, it is generally unsatisfactory to correlate streamflow records for two stations where one station has more than ten times the discharge of the other. Consequently, if the drainage pattern is such that streams draining areas as small as 50 km^2 have a potential for development, there should be some principal hydrologic stations with drainage areas smaller than 50 km^2.

Secondary hydrologic stations should be established and operated for a limited number of years at study sites in each of the climatologic and physiographic provinces in the region. They should function just long enough to establish a satisfactory correlation between their records and those of principal hydrologic stations that have similar runoff regimes, or between their records and the climatological and physical characteristics of their watersheds. By moving the secondary hydrologic stations to new sites after satisfactory correlations have been established, the entire region can eventually be covered with a dense network based on the principal hydrologic stations that are to be operated permanently. Generally speaking, the results of correlations between station runoff and basin characteristics will be adequate only for academic study of regional runoff characteristics or for gross generalizations about runoff at a site. If the site of a secondary hydrologic station is also the site of a proposed project, its correlation with a principal hydrologic station is of more than academic importance. Accordingly, such a station should generally be operated for a period longer than that of a secondary hydrologic station whose streamflow record is primarily for a special hydrologic study.

A minimum network will also include some special stations. To obtain information on the water resources potential of the region, special stations should be established to measure the outflow of major basins or sub-basins. Where a proposed project is of significant economic importance to the region, high priority should be given to the establishment of a special station at the project site for project design. The general location of a special station is fixed if it is to be established for the operation of an existing project or if it is to be established to fulfil a legal requirement. If any of the above special stations gauge natural flow, that special station also functions as a hydrologic station.

The design of the gauging station network must be tailored to the drainage characteristics of the region or its sub-regions. For example, the foregoing discussion was principally applicable to a region or sub-region having large well-developed stream systems. The design technique requires some modification in dealing with a region or sub-region where all streams are relatively small and do not combine to form large rivers. A stream pattern of that type occurs, for example, where many relatively small streams drain the seaward flank of a coastal mountain. In that situation principal hydrologic stations would be established on streams that are typical of an area and secondary hydrologic stations would be added to give wide geographic coverage on each of the hydrologic areas. It is important not to neglect the watersheds that have low runoff; having all gauges on the more productive streams can give a false impression of the water resources potential of the region. For that reason it would be prudent, before deciding on the network pattern, to make a reconnaissance of the region, using discharge measurements, precipitation maps, remote sensing, and other methods to define hydrologic boundaries and to characterize streams with respect to their runoff productivity. An attempt should be made to predict the sites where streamflow data will be most urgently needed, and to endeavour, where possible, to have hydrologic stations double as special stations, thereby reducing the number of gauging stations needed.

The gauging station network, once designed, requires evaluation from time to time. It has already been mentioned that secondary hydrologic stations are intended to be moved to new sites when statistical analysis indicates that they have fulfilled their purpose at their present sites. Project development in a basin brings changes to the network as hydrologic stations that served as special stations for the design of a proposed project, lose their utility as hydrologic stations after the project is completed. Change in location may or may not be necessary for those stations if they are to be used for the operation of the completed project. Additional special stations will often be required for the unanticipated monitoring of salinity in tidal reaches, or the monitoring of pollution at sites of intensive basin development.

2.2.2 Network design for flood frequency study

Regional flood frequency studies are used for the design of dam spillways, bridges, and culverts, and for the delineation of flood plains. The basic data for such study are records of annual peak discharge at stream sites throughout the region. Peak discharge data will be available for the principal streams at sites in the network of continuous record gauging stations that were established for the study of regional water resources. In addition, such data will usually be available for a number of smaller streams. However, a myriad of small streams is usually found in a region, and because of the high number of such streams, it is not economically feasible to gauge more than a small percentage of them. The network of standard gauging stations should therefore be augmented by a network of relatively inexpensive crest stage gauges (see Chapter 4, section 4.7) to provide more adequate coverage. Crest stage gauges are sometimes referred to as partial record gauges because they obtain peak flow data only. Other types of gauges, such as low cost pressure transducers and ultrasonic gauges, coupled with inexpensive data loggers, may also be used to expand a data network.

Even though the network of gauges is expanded by the addition of the partial record crest stage gauges or other inexpensive gauges, it is still essentially a sampling of a large number of streams in a given region. In analysing the data collected, peak discharges will be related to climatic and basin characteristics. The resulting relations will be used to estimate peak discharge at un-gauged sites. A stratified sampling technique should therefore be used in selecting the general location of crest stage gauges to ensure an adequate sampling of the many possible combinations of climatic and basin characteristics.

The design of a stratified sampling procedure depends on the climatic and physical characteristics of the region. As an example, the procedure used in a current flood frequency study will be described.

Three hundred auxiliary crest stage gauges were to be installed on watersheds ranging in size from 0.5 to 40 km^2, in a region of rugged relief encompassing about 400 000 km^2. A generalized soils map and a generalized isohyetal map of 100-year intensities of 1-hour storms were available for defining sub-regions of geologic and climatologic homogeneity. To delineate the boundaries of such sub-regions one map was superimposed upon the other; 23 sub-regions of apparent homogeneity were defined. It was expected that the peak discharge data eventually collected would show those boundaries to be in error; that is, it was expected that many of the sub-regions could eventually be combined and that some would require splitting. However, that preliminary classification of sub-regions gave a starting point for the distribution of the gauges – an average of 13 gauges per sub-region.

The next step was to distribute the 13 gauges in each sub-region. The climatology and physiography of the region is such that over large areas altitude tends to correlate with precipitation, basin slope, depth of soil mantle, and basin shape. Basin altitude was therefore selected as the single parameter to represent all parameters other than basin size, in each of the 23 sub-regions. A matrix was set up, as shown below, to indicate the sampling design for each sub-region. Each of the 12 boxes in the matrix represents a crest stage gauge, and the 13th gauge was arbitrarily added to one of the boxes so that a total 13 gauges were provided in the sub-region. In each sub-region the total range of altitude was divided into thirds. A, B, and C in the matrix represent altitude ranges, where A represents the highest third in the sub-region, B the middle third, and C the lowest third. Four ranges of drainage area size, as shown in the matrix, were used in each sub-region.

Table I.2.1. Matrix for sampling design

	Drainage area in square kilometres			
	0.5–4.0	*4–10*	*10–20*	*20–40*
	A	A	A	A
Altitude	B	B	B	B
	C	C	C	C

The purpose of the above matrix was to ensure a relatively unbiased sampling of the basins which would include most combinations of physical and climatic variables.

2.2.3 Network design for low flow frequency studies

Regional studies of low flow magnitude and frequency are useful in the planning, design, and management of water supply facilities. Low flow data provided by the network of continuous record gauges in a region can be supplemented by establishing a more comprehensive network of low flow partial record sites. These low flow sites should be distributed to measure natural flow streams in the region, including ephemeral, intermittent, and perennial streams. They should include a wide range of drainage, physiographic, and climatological characteristics. A low flow partial record network can be designed in a similar manner to a crest stage gauge network, as described in the preceding section.

Low flow magnitude and frequency can be defined at each low flow partial record site by statistical regression methods. A minimum of 8 to 10 low flow discharge measurements spanning a period of 3 to 4 years are recommended at the low flow partial record site. The corresponding (same date and time) discharge at a nearby long term hydrologic station in the same region is also determined. A statistical regression of the corresponding discharges is used to define the low flow magnitude and frequency relation for the partial record low flow site, based on the low flow magnitude and frequency relation at the long term hydrologic station. After sufficient low flow measurements are obtained to define a frequency relation at the low flow site, that site can be discontinued and a new site established on another stream in the region. In this way, low flow magnitude and frequency can be defined for a region at a much reduced cost.

Low flow partial record gauges usually do not require permanent field equipment. They are established for the purpose of making occasional discharge measurements of low flow, and do not require the measurement of stream stage. For this reason, low flow stations are inexpensive to operate and are virtually maintenance free.

2.3 CONSIDERATIONS IN SPECIFIC SITE SELECTION

After the general location of a gauging station has been determined, a specific site for its installation must be selected. For example, let us suppose that the outflow from a reservoir is to be gauged to provide the streamflow data needed for managing reservoir releases. The general location of the gauging station will be along the stretch of stream channel between the dam and the first stream confluence of significant size downstream from the dam. From the standpoint of convenience alone, the station should be established close to the dam, but it should be far enough downstream from the outlet gates and spillway outlet so that the flow is fairly uniformly established across the entire width of the stream. On the other hand the gauge should not be located so far downstream that the stage of the gauged stream may be affected by the stage of the confluent stream. Between those upstream and downstream limits for locating the gauge, the hydraulic features should be investigated to obtain a site that presents the best possible conditions for stage and discharge measurement and for developing a stable stage-discharge relation if the velocity area method is to be used (see Section 5.6.1).

If the proposed gauging station is to be established for purely hydrologic purposes, unconnected with the design or operation of a project, the general location for the gauge will be the reach of channel between two large tributary or confluent streams. The gauge should be far enough downstream from the upper tributary so that flow is fairly uniformly established across the entire width of stream, and far enough upstream from the lower stream confluence to avoid variable backwater effect. Those limits often provide a reach of channel of several kilometres whose hydraulic features must be considered in selecting a specific site for the gauge installation.

The ideal gauge site satisfies the following criteria, many of which are defined in ISO 1100-1:

(a) The general course of the stream is straight for about 10 times the stream width, upstream and downstream from the gauge site if the control is a river reach (channel control). If the control is a section control, the downstream conditions must be such that the control is not drowned. The water entering a section control should have low velocity (see (f) below);

(b) The total flow is confined to one channel at all stages and no flow bypasses the site as subsurface flow;
(c) The stream-bed is not subject to scour and fill and is relatively free of aquatic vegetation;
(d) Banks are permanent, high enough to contain floods, and are free of brush;
(e) Unchanging natural controls are present in the form of a bedrock outcrop or other stable riffle for low flow and a channel constriction for high flow – or a waterfall or cascade that is unsubmerged at all stages. If a natural control is not available, then channel conditions should allow for the construction of an artificial control such as a weir or flume (see Chapter 3);
(f) A pool is present upstream from the control at extremely low stages to ensure a recording of stage at extremely low flow, and to avoid high velocities at the streamward end of stage recorder intakes, transducers, or manometer orifice during periods of high flow. The sensitivity of the control should be such that any significant change in discharge should result in a measurable change in stage;
(g) The gauge site is far enough upstream from the confluence with another stream or from tidal effect to avoid any variable influence the other stream or the tide may have on the stage at the gauge site;
(h) A satisfactory reach for measuring discharge at all stages is available within reasonable proximity of the gauge site. It is not necessary for low and high flows to be measured at the same stream cross-section;
(i) The site is readily accessible for ease in installation and operation of the gauging station;
(j) Within reach of a suitable telemetry system;
(k) Good conditions for discharge measurements at all stages;
(l) Instruments, shelter, and housing above all flood levels. Sensors with a range to measure floods and drought.

Rarely will an ideal site be found for a gauging station and judgment must be exercised in choosing between adequate sites, each of which may have some shortcomings. Often adverse conditions may exist at all possible sites for installing a required gauging station and a poor site must be accepted. For example, all streams in a given region may have unstable beds and banks, which result in continually changing stage-discharge relations.

The reconnaissance for a gauging site essentially starts in the office where the general area for the gauge site is examined on topographic, geologic, and other maps. Reaches having the following pertinent characteristics should be noted: straight alignment, exposed consolidated rock as opposed to alluvium, banks subject to overflow, steep banks for confined flow, divided channels, possible variable backwater effect from a tributary or confluent stream or from a reservoir, and potential sites for discharge measurement by current meter. The more favourable sites will be given critical field examination and they should be marked on the map, access roads should be noted, and an overall route for field reconnaissance should be selected.

In the field reconnaissance the features discussed earlier are investigated. With regard to low flow, a stable well defined low water control section is sought. In the absence of such a control, the feasibility of building an artificial low water control is investigated. If a site on a stream with a movable bed must be accepted – for example, a sand channel stream – it is best to locate the gauge in as uniform a reach as possible, away from obstructions in the channel, such as bridges, which tend to intensify scour and fill. Possible backwater resulting from weeds in the channel should also be investigated. If the gauge is to be located at the mouth of a gorge where the stream leaves the mountains or foothills to flow onto an alluvial plain or fan, reconnaissance current meter measurements of discharge should be made during a low flow period to determine where the seepage of water into the alluvium becomes significant. The station should be located upstream from the area of water seepage in order to gauge as much of the surface flow as possible; the sub-surface flow or underflow that results from channel seepage is not "lost" water, but is part of the total water resource.

With regard to high stages, high water marks from major floods of the past are sought and local residents are questioned concerning historic flood heights. Such information is used by the engineer in making a judgment decision on the elevation at which the stage recorder must be placed to be above any floods that are likely to occur in the future. The recorder shelter should be so located as to be protected from water-borne debris during major floods. Evidence is also sought concerning major channel changes, including scour and deposition at stream banks that occurred during notable floods of the past. That evidence, if found, gives some indication of changes that might be expected from major floods of the future.

The availability of adequate cross-sections for velocity area measurement of discharge should also be investigated. Ideally, the measurement cross-section should be of fairly uniform depth, and flow lines should be parallel and fairly uniform in velocity throughout the cross-section. The cross section should be free of large bed forms such as boulders and sand dunes. The measurement section should be in reasonable proximity to the gauge to avoid the need for adjusting measured discharge for change in storage, if the stage should change rapidly during a discharge measurement. However, a distance of as much as a kilometre between gauge and measuring section is acceptable if such distance is necessary to provide both a good stage measurement site and a good discharge measurement site. Low flow discharge measurements of all but the very large streams are made by wading. For flows that cannot be safely waded, the current meter is operated from a bridge, cableway, or boat. It is most economical to use an existing bridge for that purpose, but in the absence of a bridge, or if the measuring section at a bridge site is poor, a suitable site should be selected for constructing a cableway. If construction of a cableway is not feasible because of excessive width of the river, high water measurements may be made by boat. The cross-section used for measuring high flows may not be suitable for measuring low flows, and wading measurements are therefore made wherever measuring conditions are most favourable.

Without knowledge of stage and discharge at a potential gauge site and of concurrent stage at the stream confluence or reservoir, an engineer or hydrologist can only conjecture concerning the location on the stream where backwater effect disappears for various combinations of discharge and stage. A safe rule is the following: given a choice of several acceptable gauging sites on a stream, the gauging site selected should be the one farthest upstream from the possible source of variable backwater. If it is necessary to accept a site where variable backwater occurs, there are basically two choices for defining a method to compute discharge. One is to use a measurement of water surface slope, and the other is to use a measurement of stream velocity, in addition to the measurement of stream stage. These measurements are used to define a multi-variable relationship to compute discharge. Slope is sometimes used where variable backwater is infrequent, or affects discharge computations only during high stages. A uniform reach for measurement of slope should be sought, along with a site for the installation of an auxiliary gauge. If an auxiliary gauge is used to measure the water surface fall between it and the principal gauge, the site of the auxiliary gauge should preferably be downstream from the principal gauge, and it should be far enough downstream to provide a fall of at least 0.1 metre. For gauge sites that have frequent, or constant, backwater it is preferable to use some type of velocity index measurement instead of water surface slope. Velocity index gauges, such as electromagnetic or acoustic, can usually be installed at the same location as the stage gauge.

In cold regions the formation of ice always presents a problem in obtaining reliable winter records of streamflow. However in regions that are only moderately cold, and therefore subject to only moderate ice “build-up”, forethought in the selection of gauge sites may result in streamflow records that are free of ice effect. Gauge sites that are desirable from that stand-point are as follows:

(a) Below an industrial plant, such as a paper mill, steel mill, thermal power plant, or coal mine. Waste heat may warm the water sufficiently or impurities in the water may lower the freezing point to the extent that open water conditions always prevail;
(b) Immediately downstream from a dam with outlet gates. Because the density of water is maximum at a temperature of 4°C, the water at the bottom of a reservoir is commonly at or near that temperature in winter. Most outlet gates are placed near the bottom of the dam, and the water released is therefore approximately 4°C above freezing. It would take some time for that water to lose enough heat to freeze;
(c) On a long fairly deep pool just upstream from a riffle. A deep pool will be a tranquil one. Sheet ice will form readily over a still pool, but the weather must be extremely cold to give complete cover on the riffle. At the first cold snap, ice will form over the pool and act as an insulating blanket between water and air. Under ice cover the temperature of the streambed is generally slightly above the freezing point, and may, by conduction and convection, raise the water temperature at the base of the riffle to slightly above freezing, even though water enters the pool at 0°C. That rise in temperature will often be sufficient to prevent ice formation on the riffle;
(d) The station must be placed so that it is not destroyed by moving ice.

After the many considerations discussed on the preceding pages have been evaluated, the precise sites for the water level recorder and for the cableway for discharge measurements (if needed) are selected. Their locations in the field are clearly marked and referenced. The maximum stage at which the low water control will be effective should be estimated; the intakes, transducers or orifices should be located upstream from the low water control, a distance equal to at least three times the depth of water on the control at that estimated maximum stage. If the water level sensors are located any closer to the control, they may lie in a region where the streamlines have vertical curvature. Water level determination in that region is hydraulically undesirable. If an auxiliary gauge such as a downstream stage gauge, or a velocity index gauge, is required, then similar field information should be marked and referenced. If a section control exists, then a cross section of the control should be surveyed.

The gauging stations shown in Figures I.2.1 and I.2.2, satisfies most of the requirements discussed in this chapter. Low flow measurements are made by wading in the vicinity of the gauge. The bridge site in Figure I.2.1 provides accessibility, convenience to power lines, and a good location for an outside reference gauge and high flow discharge measurements. Figure I.2.2 shows another example with a cableway, solar panel, and satellite transmission antenna.

Up to this point there has been no discussion of specific site location for crest stage gauges. Those gauges provide peak stage data only and should be installed in a straight reach of channel that can be utilized in computing peak discharge by defining a stage-discharge relationship, or by measuring peak discharge indirectly by using one of the indirect methods such as the slope-area method, the

Figure I.2.1. Gauge with walkin shelter, solar panels, satellite antenna, and wire-weight gauge on bridge

Figure I.2.2. Gauge with walk-in shelter, solar panels, satellite antenna, and cableway

contracted opening method, or the culvert method (see Chapter 9).

Site section for low flow partial record gauges requires a good location, easily accessible, for making current meter measurements of low discharges. There usually are no requirements for measuring stream stage.

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CHAPTER 3

GAUGING STATION CONTROLS

3.1 TYPES OF CONTROL

The conversion of a record of stage to a record of discharge is made by the use of a stage-discharge relation. The physical element or combination of elements that maintains the relation is known as a control. One major classification of controls differentiates between section controls and channel controls. Another classification differentiates between natural and artificial controls. Artificial controls are structures built for the specific purpose of controlling the stage-discharge relation, such as a weir, flume, or small dam. A third classification differentiates between complete, partial, and compound controls.

Additional information about controls can be found in Carter and Davidian (1968), Rantz and others (1982), *Manual on Stream Gauging* (WMO-No. 519) and *Guide on Hydrological Practices* (WMO-No. 168) and ISO (1983, 1995, 1996, 1998).

Section control exists when the geometry of a single cross-section is such as to constrict the channel, or when a downward break in bed slope occurs at a cross-section. The constriction may result from a local rise in the streambed, as at a natural riffle or rock ledge outcrop, or at a constructed weir or dam; or it may result from a local constriction in width, which may occur naturally or be caused by some man-made channel encroachment, such as a bridge whose waterway opening is considerably narrower than the width of the natural channel. Examples of a downward break in bed slope are the head of a cascade or the brink of a waterfall.

Channel control exists when the geometry and roughness of a long reach of channel downstream from the gauging station are the elements that control the relation between stage and discharge. The length of channel that is effective as a control increases with discharge. Generally speaking, the flatter the stream gradient, the longer is the reach of channel control.

A *complete control* is one that governs the stage-discharge relation throughout the entire range of stage experienced at the gauging station. More commonly, however, no single control is effective for the entire range of stage and thus a *compound control* will exist for the gauging station. A common example of a compound control is the situation where a section control is the control for low stages and channel control is effective at high stages. The compound control sometimes includes two section controls, as well as channel control. In that situation the upstream section control is effective for the very low stages, a section control farther downstream is effective for intermediate stages, and channel control is effective at the high stages. At very high stages the control may be a combination of channel control and overbank conveyance where overbank flow is a significant part of the total flow.

With regard to complete controls, a section control may be a complete control if the section control is a weir, dam, cascade, or waterfall of such height that it does not become submerged at high discharges. A channel control may be a complete control if a section control is absent, as in a sand channel that is free of riffles or bars, or in an artificial channel such as a concrete lined floodway.

A *partial control* is a control that acts in concert with another control in governing the stage-discharge relation. That situation exists over a limited range in stage whenever a compound control is present. As an example, consider the common situation where a section control is the sole control for low stages and channel control is solely operative at high stages. At intermediate stages there is a transition from one control to the other, during which time submergence is "drowning out" the section control. During this transition period the two controls act in concert, each as a partial control. Where the compound control includes two section controls, the degree of submergence of the upstream section control will be governed by the downstream section control, during a limited range in stage. When that occurs, each of the section controls is acting as a partial control. A constriction in channel width, unless unusually severe, usually acts as a partial control, the upstream stage being affected also by the stage downstream from the constriction.

3.2 ATTRIBUTES OF A SATISFACTORY CONTROL

The two attributes of a satisfactory control are stability and sensitivity. If the control is stable the stage-discharge relation will be stable. If the control is subject to change, the stage-discharge relation is likewise subject to change, and frequent discharge measurements are required for the continual recalibration of the stage-discharge relation. This increases the operating cost of the gauging station and likely reduces the accuracy of the streamflow record.

The primary cause of changes in natural controls is the high velocity associated with high discharge. Of the natural section 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. Of the natural channel controls those with unstable bed and banks, as found in sand channel streams, are the most likely to change as a result of velocity induced scour and deposition.

Another cause of changes in natural controls is aquatic vegetation. The growth of aquatic vegetation on section controls increases the stage for a given discharge, particularly in the low flow range. Vegetation on the bed and banks of channel controls also affects the stage-discharge relation by reducing velocity and the effective cross sectional area. In the temperate climates, accumulations of water-logged fallen leaves on section controls each autumn clog the interstices of alluvial riffles and raise the effective elevation of all natural section controls. The first ensuing stream rise of any significance usually clears the control of fallen leaves.

Controls, particularly those for low flows, should be sensitive. Sensitivity may be defined as an indication of the quickness and extent of response to an increase in discharge by an increase in stage. Generally a low water control is considered to be sensitive if a change of no more than 2 per cent in discharge is represented by a change of one unit of recorded stage. This unit is usually taken to be 3 mm in the United Kingdom of Great Britain and Northern Ireland and the United States of America. The following tables show typical computation of sensitivity for Crump and Flat-V weirs based on this concept.

It will be noted from the tables that the percentage change in discharge is referred to the lower of the two discharges considered and that percentage sensitivity decreases as stage increases. The fact that sensitivity cannot be defined by a single value has probably resulted in its lack of general usage as a quality indicator. This is unfortunate because it

Table I.3.1. Crump weir

Stage, m	*Discharge/metre width m^3/s*	*Difference m^3/s*	*Sensitivity percentage*
0.050-0.053	0.022-0.024	0.002	9.1
0.250-0.253	0.245-0.249	0.004	1.6
0.500-0.503	0.693-0.699	0.006	0.7
0.750-0.753	1.273-1.281	0.008	0.6
1.000-1.003	1.960-1.969	0.009	0.5
1.500-1.503	3.601-3.611	0.010	0.3
2.000-2.003	5.544-5.556	0.012	0.2

Table I.3.2. Flat-V weir

Stage, m	*Discharge/metre width m^3/s*	*Difference m^3/s*	*Sensitivity percentage*
0.050-0.053	0.017-0.020	0.003	15.5
0.250-0.253	0.969-1.000	0.031	3.0
0.500-0.503	5.495-5.558	0.063	1.0
0.750-0.753	15.121-15.276	0.155	1.0
1.000-1.003	31.050-31.283	0.233	0.8
1.500-1.503	85.573-85.853	0.280	0.3
2.000-2.003	175.650-176.302	0.652	0.3

affords a means of comparing the quality of gauging stations. To overcome this difficulty, sensitivity may be calculated at some previously determined probability level of discharge, for example at the 95 percentile, the flow equalled or exceeded for 95 per cent of the time. In this context sensitivity may be defined as the increase in stage in millimetres caused by an increase in discharge of say one per cent at the stage corresponding to the long-term 95 percentile level of discharge. This means that on average the sensitivity of the control, and therefore of the stage-discharge relation, will be better than the adopted value for 95 per cent of the time.

In practice the importance of sensitivity is reflected in the reading of stage to a specified uncertainty. For instance if the stage is read with an uncertainty of 5 mm at a station having a sensitivity of 5 mm then the uncertainty in the computed discharge is one per cent which is regarded as acceptable. However, if at a station with a sensitivity of 0.5 mm the stage is read with an uncertainty of 5 mm, then the uncertainty in the computed discharge would be 10 per cent which may be regarded as unacceptable. In this latter case it may be considered desirable to operate the station to a standard that would ensure a more accurate reading of stage.

Controls are therefore designed to ensure a satisfactory sensitivity requirement. To meet this requirement it is necessary that the width of flow at the control be greatly constricted at low stages. In a natural low water control such constriction occurs if the control is in effect, notched, or if the controlling cross-section roughly has a flat V shape or a flat parabolic shape. These shapes will ensure that the width of flow over the control decreases as discharge decreases.

In the interest of economy a gauging station should be located upstream from a suitable natural control (Figure I.3.1). However, where natural conditions do not provide the stability or the sensitivity required, artificial controls should be built if they are economically feasible.

3.3 ARTIFICIAL CONTROLS

An artificial control for a velocity area station is a structure built in a stream channel to stabilize and constrict the channel at a section, and thereby simplify the procedure of obtaining accurate records of discharge. The artificial controls built in natural streams are usually broad-crested weirs that conform to the general shape and height of the streambed. (The term "broad-crested weir", as used in this manual, refers to any type of weir other than a thin-plate weir.) In small canals and drains, where the range of discharge is limited, thin-plate weirs and flumes are the controls commonly built. Thin-plate weirs are built in those channels whose flow is relatively sediment free and whose banks are high enough to accommodate the increase in stage (backwater) caused by the installation of a weir. A suppressed rectangular weir is shown in Figure I.3.2. Flumes are largely self-cleaning and can therefore be used in channels whose flow is sediment laden, but their principal advantage is that they cause relatively

Figure I.3.1. Gauging stations with natural section controls

Figure I.3.2. A suppressed rectangular weir

Figure I.3.3. A Parshall measuring flume

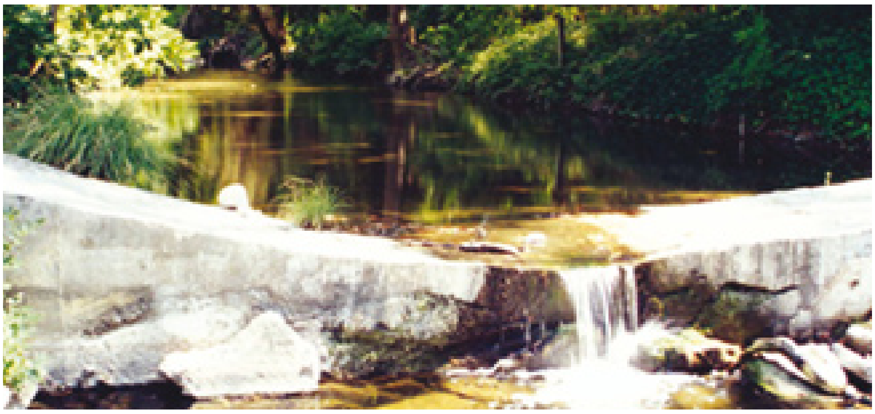

Figure I.3.4. A concrete, broad-crested artificial control

little backwater (head loss) and can therefore be used in channels whose banks are relatively low. Flumes are generally more costly to build than weirs. A Parshall flume (Figure I.3.3) is an example of such a flume.

Artificial controls eliminate or alleviate many of the undesirable characteristics of natural section controls. Not only are they physically stable, but they are not subject to the cyclic or progressive growth of aquatic vegetation. Algal slimes that sometimes form on artificial controls can be removed with a wire brush, although care should be taken so that the surface of the weir is not damaged. The controls are usually self-cleaning with regard to fallen leaves. In moderately cold climates artificial controls are less likely to be affected by the formation of winter ice than are natural controls. The artificial control can, of course, be designed to attain the degree of sensitivity required for the gauging station. In addition, an artificial control may often provide an improved discharge measurement section upstream from the control by straightening the original angularity of flow lines in that cross-section.

In canals or drains, where the range of discharge is limited, artificial controls are usually built to function as complete controls throughout the entire range in stage. In natural channels it is generally impractical to build the control high enough to avoid submergence at high discharges, and the broad-crested weirs that are usually built are effective only for low, or for low and medium, discharges. Figures I.3.4 and I.3.5 illustrate broad-crested weirs of two different shapes; the crests of both have a flat upward slope from the centre of the stream to the banks, but in addition, the weir in Figure I.3.5 has a shallow V-notch in the centre for greater sensitivity.

Figure I.3.5. A concrete, broad-crested artificial control with a shallow V notch in the centre

Attributes desired in an artificial control include:

(a) The control should have structural stability and should be permanent. The possibility of excessive seepage under and around the control should be considered, and the necessary precautions should be prevented by means of sheet-piling or concrete cut-off walls and adequate abutments;
(b) The crest of the control should be as high as practicable to eliminate, if possible, the effects of variable downstream conditions or to limit those effects to high stages only;
(c) The profile of the crest of the control should be designed so that a small change in discharge at low stages will cause a measurable change in stage. If the control is intended to be effective at all stages, the profile of the crest should be designed to give a stage-discharge relation of such shape that it can be extrapolated to peak stages without serious error;
(d) The shape of the control structure should be such that the passage of water creates

no undesirable disturbances in the channel upstream or downstream from the control;

(e) Artificial controls are often built to conform to the dimensions of laboratory-rated or field-rated weirs or flumes. The question arises whether to use the pre-calibrated rating or to calibrate each new installation in place. There are two schools of thought on the subject. In many countries, the pre-calibrated rating is accepted and discharge measurements by current meter or by other means are made only periodically to determine if any statistically significant changes in the rating have occurred. If a change is detected, the new rating is defined by as many discharge measurements as are deemed necessary. The most serious error where a calibration rating or formula is used may occur in the measurement of stage over the control. This is especially so at low flows. Care should always be exercised in setting the "zero" of the reference gauge and the recorder to the crest of the control. In other countries, most notably in the United States of America, the position is taken that it is seldom desirable to accept the rating curve prepared for the model structure without checking the entire rating of the prototype structure in the field by current meter measurements, or by other methods of measuring discharge. The experience in those countries has been that there are invariably sufficient differences between model and prototype to require complete in-place calibration of the prototype structure. The most notable exceptions are standard thin-plate weirs with negligible velocity of approach. The relative merits of the two schools of thought will not be debated here.

If the stream carries a heavy sediment load, the artificial control should be designed to be self-cleaning. Flumes have that attribute; broad-crested weirs can often be made self-cleaning by a design modification in which the vertical upstream face of the weir is replaced by an upstream apron that slopes gently from the stream bed to the weir crest.

3.4 SELECTION AND DESIGN OF AN ARTIFICIAL CONTROL

Cost is usually the major factor in deciding whether or not an artificial control is to be built to replace an inferior natural control. The cost of the structure is affected most by the width of the stream and the type or condition of bed and bank material. Stream width governs the size of the structure, and bed and bank material govern the type of construction that must be used to minimize leakage under and around the structure.

If an artificial control is to be used, the type and shape of the structure to be built is dependent upon channel characteristics, flow conditions, range of discharge to be gauged, sensitivity desired, and the maximum allowable head loss (backwater).

Channel characteristics and flow conditions govern the general choice to be made among the various types of control structure. The standard types of weir and flume are usually not suitable for steep channels where the Froude number (*Fr*)* is greater than about 0.5. Best results are obtained for the type of structure where the static head is the major part of the total head, which may not be the case in steep channels where the velocity head becomes excessively large. Furthermore, structures act as sediment traps on steep streams. A flume is superior to a weir for use in sediment-laden streams, but even flumes are unable to pass the larger bed material that is part of the moving bed load in steep channels.

The three factors to be considered, namely, range of discharge to be gauged, sensitivity desired, and the maximum allowable head loss, must be treated together in the precise determination of the most suitable type of control structure, its shape, and crest elevation. However, two preliminary steps are first necessary.

First, the head-discharge relations for various artificial controls of standard shape are assembled. These are found in Technical Note No. 117, (WMO-No. 280), *Use of Weirs and Flumes in Stream Gauging*. Head discharge relations for additional artificial controls that were field-calibrated will usually be available in the files of water resources agencies in the area – some are given in Volume II, Chapter 1 – Discharge ratings using simple stage-discharge relations. The head discharge relations that are assembled need only be approximately correct, because channel conditions at the site of the proposed control will seldom match those for the model control.

The second preliminary step is to determine an approximate stage-discharge relation for the anticipated range in stage in the unobstructed channel at the site of the proposed control. This may be done by a few current meter measurements and/or by the use of an open-channel discharge

* $Fr = \bar{V}^2 / g\bar{d}$, where $\bar{V}$ and $\bar{d}$ are the mean velocity and mean depth in the cross section, respectively, and g is the acceleration of gravity.

equation, such as the Manning equation, in which uniform flow is assumed for the site and a value of the roughness coefficient is estimated. The reliability of the computed stage-discharge relation will be improved if one or more discharge measurements are made to verify the value of the roughness coefficient used in the computations. The purpose of the computations is to determine the tailwater elevation that is applicable to any given discharge after an artificial control is installed.

The next step is to consider the lower discharges that will be gauged. The tail-water elevations corresponding to those discharges are used to determine the minimum crest elevations permissible for the proposed artificial control under conditions of free flow at the lower discharges. If a flume is to be installed, the throat section should be narrow enough to ensure sensitivity at low discharges. If a weir is to be installed and if the stream is of such width that a weir with a horizontal crest will be insensitive at the low discharges, the use of a flat V crest is recommended — for example, one whose sides have a slope of between 1 vertical to 10 horizontal, and 1 vertical to 20 horizontal. The sensitivity desired is usually such that the discharge increases no more than 2 to 5 per cent for each increase in stage of 0.003 m. It is also desirable that the crest of the weir or critical-flow flume (Parshall flume) be so shaped that the minimum discharge to be gauged has a head of at least 0.06 m to eliminate the effects of surface tension and viscosity.

Head discharge relations for weirs of various types and shapes are computed for the anticipated range in discharge, and a structure is selected that best meets the demands of the site in acting as a control for as much of the range as possible, without exceeding the maximum allowable backwater effect (head loss) at the higher stages and with minor submergence effect at lower stages. In other words, a high crest elevation minimizes submergence but maximizes backwater effect which may cause or aggravate flooding; a low crest elevation maximizes submergence but minimizes backwater effect. The engineer must select a control design that is optimum for the local condition.

It was stated earlier that standard types of weirs and flumes are usually not suitable for steep channels. While this is true, it should also be noted that where accurate discharge data are required for steep sediment-laden streams whose unstable beds cause unstable stage-discharge relations, the discharge relations are sometimes stabilized by the construction of specially designed weirs or flumes. In general, this has been done only for research watersheds. The specially designed crest of the broad crested weir or the floor of the flume is often given a supercritical slope in the direction of flow to prevent the deposition of sediment on the structure. The intakes for recording stage are located in the supercritical flow portion of the structure. Because of the relative instability of supercritical flow it is usually necessary to construct a laboratory model of the reach of channel for use in designing a control structure whose operation will be compatible with channel conditions. The location and design of the intakes for recording the stage of supercritical flow is of prime importance in the model study. The cost of designing and building a control of this kind is usually prohibitive for routine stream gauging but this of course depends upon the need for the data.

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CHAPTER 4

MEASUREMENT OF STAGE

4.1 INTRODUCTION AND PURPOSE

The *stage* of a stream or lake is the height of the water surface above an established datum plane. The water-surface elevation for most rivers and streams is measured above an arbitrary or predetermined gauge datum and is called the gauge height of the river or stream. For lakes, reservoirs, coastal streams, and estuaries the height of the water surface is usually referred to as an *elevation* measured above sea level or above an established geodetic vertical datum. Gauge height is often used interchangeably with the more general term stage, although the term gauge height is more appropriate when used with a specific reading on a gauge. Stage or gauge height is usually in metres and hundredths or thousandths of a metre, or in feet and hundredths of a foot.

A record of stream stage is useful in itself, as in designing bridges, embankments, levees, and other structures affected by stream elevations, or in planning for the use of flood plains. In stream gauging, gauge heights are used as the independent variable in a stage-discharge relation to compute discharges. Reliability of the discharge record is therefore dependent on the accuracy and precision of the gauge-height record as well as the stage-discharge relation. Elevation records of lakes and reservoirs provide an index of lake surface area and volume, as well as the elevation of the lake or reservoir.

Gauge-height records may be obtained by systematic observation of a non-recording gauge, or with automatic water level sensors and recorders. Various types of transmitting systems are frequently used to automatically relay gauge-height information from remote gauging stations to office based computer systems.

New technology, especially in the field of electronics, has led to a number of innovations in sensing, recording, and transmitting gauge height data. In the past the majority of gauging stations used floats in stilling wells as the primary method of sensing gauge height. Floats and stilling wells are still in common use today. However the current trend is toward the use of submersible or non-submersible pressure transducers which do not require a stilling well. Electronic data recorders and various transmission systems are now being used extensively. Transmission systems may include telephone, cellular phone, radio, or satellite methods. These new methods and equipment generally require only an instrument shelter with a pressure measurement device in the stream connected by tubing or cable to a data logger and/or transmitter in the instrument shelter.

This chapter describes the instrumentation and methods currently being used to obtain gauge height data. This includes the traditional float/stilling-well method and the newer methods utilizing pressure transducers and gas-purge systems. This chapter first describes basic requirements necessary to all gauges, such as gauge datum and stage accuracy standards. Gauge structures and the various types of instrumentation are described, such as reference and auxiliary gauges, crest-stage gauges, float gauges, shaft encoders, bubble gauges, recorders, telemetry systems, power supplies, and other new instruments that are still in the development and testing phase. This chapter also describes various instrumentation configurations, methods of data retrieval, and gauging station design and operation criteria.

A large amount of information in this chapter is based on *Guide to Hydrological Practices* (WMO-No. 168), ISO (1983), Rantz (1982) and ISO (1983).

4.2 BASIC REQUIREMENTS FOR COLLECTING STAGE DATA

The collection of stage data, either manually or automatically, requires that various instruments be installed at a gauging site. For stage data to be useful for their intended purposes, requirements for maintaining a permanent gauge datum and meeting specified accuracy limits are important. This section of the report provides definitions of the components, as well as the basic accuracy requirements. Descriptions of specific instruments are given in subsequent parts of this report.

4.2.1 Gauge component definitions

The *reference gauge* for an automatically recording gauging station is a non-recording gauge used to set recorders, data loggers, or transmitters from which the primary gauge height record is obtained. The reference gauge designated for this purpose is sometimes called the base gauge, but this Manual will use the term “reference gauge”, as defined by Rantz (1982). For gauging stations utilizing a stilling well, the reference gauge is usually a staff gauge, float-tape gauge, or electric-tape gauge located inside the stilling well. These gauges are generally referred to as inside gauges. For gauges without stilling wells, such as a bubble gauge, the reference gauge is usually a gauge mounted directly in or over the stream, designated specifically to be the reference gauge, and generally referred to as an outside gauge. This may be a staff gauge, wire-weight gauge, chain gauge, or other type of gauge. For a non-recording station, the reference gauge is usually an outside gauge, such as a staff gauge, wire-weight gauge, chain gauge or other type of gauge read by an observer.

An *auxiliary gauge* is any non-recording gauge other than the reference gauge, and is used primarily for comparison and checking of the reference gauge. For instance, where the inside gauge is the reference gauge, an outside gauge is considered an auxiliary gauge.

A *stage sensor* is a device that automatically determines (senses) the vertical position of the water surface. This may be a float riding on the water surface inside a stilling well. It may be a non-submerged pressure transducer coupled with a gas-purge bubbler orifice. It may be a submerged pressure transducer coupled with an electronic cable to transmit the hydrostatic pressure for determining the water level. Or it may be an acoustic, laser, or optical pulse that reflects from the water surface to other instruments designed and calibrated for measuring gauge height.

A *stilling well and intake system* consists of a closed well, connected to the stream by intake pipes. It is designed so that an accurate water level of the stream can be determined inside the stilling well, even during rapidly rising or falling conditions, but without any appreciable surge or wave action. Stilling wells are generally used where a float sensor is used, but can be used with gas-purge bubbler orifices or submersible pressure sensors as well.

A *gas-purge bubbler orifice* is a device placed in the stream, or sometimes near the bottom of a stilling well, to emit bubbles at a set rate. This device, when connected by a gas feed tube to a gas-purge system and pressure transducer, becomes part of a bubble-gauge stage-sensing system.

A *stage recorder* is a graphical, digital, or electronic device that automatically records and stores gauge height readings sensed by a stage sensor. Graphical (analog) recorders produce a continuous chart of gauge height. Digital and electronic recorders generally store gauge heights at pre-determined time intervals, such as every 5-minutes, 15-minutes, or 1-hour. However, other uniform time intervals are sometimes used, as well as non-uniform time intervals based on pre-programmed conditions.

Gauge height retrieval is the means by which gauge height data are extracted from the recorder. This may be simply by manually removing a chart or paper punch tape, by downloading the data from the recorder to a field computer, or by removing an electronic memory device from the recorder.

A *telemetry system* is the means by which gauge height data are automatically transmitted from the field recorder to another location. Telephone, cellular phone, radio, or satellite communication may perform this function.

4.2.2 Gauge datum

The datum of the gauge may be either a recognized datum, such as the North American Vertical Datum (NAVD) of 1988 in the United States of America, or an arbitrary datum chosen for convenience. An arbitrary datum plane is usually used for stream-gauging sites where it is desirable for all recorded gauge heights to be relatively low numbers. The arbitrary datum plane for a stream-gauging site should be selected so that negative values of gauge height do not occur. This requires the arbitrary datum plane to be below the lowest expected gauge height, which will be at, or below, the elevation of zero flow on the control for all conditions.

A permanent gauge datum should be maintained, if at all possible, so that only one datum for the gauge-height record is used for the life of the gauging station. To maintain a permanent datum each gauging station requires at least three permanent reference marks that are independent of the gauge structure. Levels should be run periodically to all gauges and reference marks to verify that the reference gauge and the auxiliary gauges have not changed relative to the established datum and to determine the magnitude of any changes that may have occurred. Reference and auxiliary gauges should be corrected as necessary in order to maintain the established datum. Levels should be run at least once a year at new gauge sites and at sites where the datum is not stable. After it is confirmed that the datum is fairly stable, levels can be run every 2 or

3 years, and in some cases an even longer time between levels may be acceptable.

The gauge datum may require a change when excessive channel scour, or a manmade channel change, occurs. It is recommended that such a change be in increments of whole metres (or feet) so the new datum can be easily related to the old datum. In some instances the gauge itself may need to be relocated to another site. The relation between the datum for the new gauge site and the datum for the old gauge site should be defined by leveling; however, it is not usually necessary to use the same datum at both sites. A permanent record of all datum changes should be maintained.

If an arbitrary datum plane is used for a gauging station, it is desirable to establish its relation to the national datum, or mean sea level, by levels in order to maintain a national datum for the gauge height record. In addition it would allow for recovery of the arbitrary datum if the gauge and local reference marks are destroyed.

4.2.3 Stage accuracy requirements

Stage and elevation data are used primarily as an index for computing stream discharge and reservoir contents. The established methods require that stage data should be measured and stored as instantaneous values rather than averaged values. Subsequent data processing and analysis will provide the means for any required averaging. Therefore, the following paragraphs on accuracy requirements and stage measurement error pertain to instantaneous stage values.

A number of factors enter into the determination of stage accuracy requirements. For instance the specific use for which the stage data are collected is a significant factor. Stage data used to compute streamflow records must be significantly more accurate than stage data used for some design applications, or certain flood plain management applications. The primary use of stage data is for computation of streamflow records, consequently stage accuracy requirements are very stringent. In accordance with this primary use and because the use of stage data cannot be predicted, the overall accuracy of stage data established for gauging stations is either 0.003 m (or 0.01 ft), or 0.2 per cent of the effective stage, whichever is greater. For example, the required accuracy would be 0.02 m at a 10 m effective stage, 0.006 m at 3 m, and 0.003 m at all effective stages less than 1.75 m. Effective stage is defined as the height of the water surface above the orifice, intake, or other point of exposure of the sensor to the water body. The instrument should be installed in the field with the orifice or intake only slightly below the zero-flow stage, or other defined low point-of-use.

The accuracy criteria stated above applies to the complete stream gauging station configuration and is a composite of errors, or total error, from all of the components necessary for sensing, recording and retrieving the data. The individual sources of stage measurement errors are described in the next section of this report.

At non-stream gauging station sites, such as reservoirs, lakes and estuaries, the same accuracy requirements apply as those for stream sites. However, in some cases higher accuracy may be required, such as for the computation of storage changes in reservoirs, or for computation of discharge using slope ratings or unsteady-flow models. Where greater accuracy is required instruments should be selected accordingly.

When field conditions such as high velocities, wave action or channel instability make it impossible to collect accurate stage data or to define an accurate stage-discharge relation, stage data should be collected with the greatest accuracy feasible. Appropriate instruments and methods should be selected to fit the field conditions.

Sources of stage measurement errors

The measured stage of a stream or other water body at any given point in time is subject to numerous sources of incremental errors. The combined effect of these errors should be within the accuracy requirements stated in the preceding section. The accuracy requirement for any single component of a stage measuring system will generally be more stringent than the requirement for the system as a whole. However, it is not always possible to isolate, or pinpoint an error and attribute it to one specific component. This part of the report describes the various sources of error in general. For additional descriptions see Rantz (1982).

Datum errors

The gauge datum is described in a previous section of this chapter. Movement of a gauge caused by uplift or settlement of the supporting structure can cause datum errors that can only be detected by running levels. Gauge datum for reference gauges should be maintained to an accuracy of 0.003 m (0.01 ft), which can usually be achieved by running levels to established reference marks every 2 or

3 years. Levels may be required at more frequent intervals where conditions are not stable. Generally, gauges need not be adjusted unless datum discrepancies exceed 0.006 m (0.02 ft).

Gauge reading errors

Errors can result from inaccurate gauge readings where it may be difficult to detect the water line against a staff gauge because of poor lighting or very clear water. In other instances accurate gauge readings may be difficult to make because of water surge. These errors can be reduced or eliminated by careful observation and in the case of surge by averaging several observations. Other errors can be caused by site conditions such as the reading of a wire-weight gauge with the weight lowered from a very high bridge. In almost all cases gauges should be read to the nearest 0.003 m (0.01 ft). If gauges cannot be read to this accuracy, notes should be made and the error magnitude estimated.

Stage sensor errors

Stage sensors, such as floats, pressure transducers and other stage sensing devices may introduce gauge height errors. The float in a stilling well may sometimes leak, the float-tape clamp may have slipped, or small animals may rest on the float. In most cases instances problems with the float will cause it to float lower than originally set, causing gauge readings to be too low. The diameter of the float is also critical to response speed.

Pressure transducers may have or may develop calibration errors. These errors can result in plus or minus deviations from the true gauge height. The standard for acceptable errors in submersible or non-submersible pressure transducers is 0.003 m (0.01 ft) or 0.10 per cent of the effective stage, whichever is greater. In other words the acceptable error is 0.003 m for an effective stage of 3 m, 0.010 m for an effective stage of 10 m, and 0.015 m for an effective stage of 15 m. For Imperial units, the acceptable error is 0.01 ft for an effective stage of 10 ft or less, 0.03 ft for 30 ft and 0.05 ft for 50 ft. Various makes and models of pressure transducers have been tested by the United States Geological Survey (USGS) Hydrologic Instrumentation Facility (HIF) to determine if they meet these standards. Likewise, independent testing is required before use of commercially obtained pressure transducers.

Non-contact stage measuring devices are available that use water surface sensing methods such as ultrasonic acoustic wave transmission, radio wave (radar) transmission and optical (laser) transmission systems. Ultrasonic and radar, although relatively new at this writing, are used extensively and successfully in Europe. Ultrasonic methods can also be used from below the water surface, measuring upwards to detect the water level. Under very controlled conditions, some of these devices are accurate to within 0.003 m (0.01 ft). Sources of error include temperature variations, density variations, and compositional variations in the transmission column. In some cases, obstacles such as snow, rain, or dust can affect accuracy. The electronics of these systems can be very sensitive and affect accuracy.

Water surface-to-sensor-to-recorder errors

The communication link between the stream water surface, the stage sensor and the data recorder can sometimes develop problems, or have inherent problems, that result in gauge height errors. For instance, for a stilling well and float system the intakes may become clogged or excessive sediment may settle in the stilling well, or the float tape may hang. These are major problems that usually result in a complete loss of data. More subtle problems can also occur, that are not so obvious, but may result in small gauge height errors as described in the following paragraphs.

Line shift error (LSE) is an error that occurs when part of the float tape passes from one side of the float pulley to the other. This change in weight results in the float depth to change slightly, resulting in a small error of the recorder stage. The error is positive (+) for a rising stage and negative (–) for a falling stage. The equation for the line shift error, LSE, is as follows:

$$LSE = (0.00256)\frac{u}{D^2}\Delta H \qquad (4.1)$$

where u = unit weight of the tape, in kg/m, D = float diameter, in m, and ΔH = the change in stage, in m.

Float lag error is an error that occurs when the recorder is set during either a rising or falling stage. If the float-operated recorder is set to the true water level while the water level is rising, it will therefore show the correct water level, as far as float lag is concerned, for all rising stages. For falling stages, however, the recorded stage will be above the true water level (positive error) by the amount of float lag or change in flotation depth of the float. A reverse effect occurs if the original gauge setting is made when the water level is falling. Float lag varies directly with the force required to move the mechanism of the recorder and inversely as the square of the float diameter.

The equation for maximum float lag error (MFLE) is:

$$MFLE = 0.00256 \frac{F}{D^2} \tag{4.2}$$

where F = the force required to move the mechanism of the recorder, in kg, and D = the float diameter, in m.

Submergence of the counterweight can introduce a small error in the recorded gauge height. When the counterweight and any part of the float line become submerged as the stage rises the pull on the float is reduced and its depth of flotation is increased. The converse is true when the submerged counterweight emerges from the water on a falling stage. Thus the error caused by submergence or emergence is opposite to that of the line shift error and tends to compensate for the line shift error. The submergence error is dependent on the weight of the counterweight and the float diameter.

The equation for submergence error (SE) is:

$$SE = (0.000118) \frac{c}{D^2} \tag{4.3}$$

where c = the weight of the counterweight, in kg, and D = the float diameter, in m.

Although not related to errors inherent in a float-operated stage recorder it might be mentioned here that error in recorded stage may be caused by expansion or contraction of the stilling well of a tall gauge structure that is exposed to large temperature changes. For example, a steel well 25 m high, exposed to an increase in temperature of 40°C, will have its instrument shelf raised 0.012 m, assuming that the instrument shelter is attached to the well.

Likewise, for gas bubbler systems errors can result from gas friction in the line between the bubbler orifice and the non-submerged pressure transducer, the variation in weight of the gas column with stage, and the gas bubble rate. Rantz (1982), Smith (1991) and Kirby (1991) discuss these error sources in detail and quantify their magnitudes. Submersible pressure transducers show variation in measured water pressure because of water temperature, water density and other factors.

In the case of electronic stage sensors, errors may occur as a result of converting an electrical signal to a real number of stage suitable for recording. In other instances errors may occur when mechanical movement is converted to a real number of stage.

Non-contact gauges have other sources of error. For instance radar gauges are often affected by waves on the water surface. Laser gauges, although seldom used, are affected by the clarity of the water.

Hydraulically induced errors

High velocity in the stream near the outside end of the intake pipes can cause drawdown, or sometimes build-up, of the water surface inside a stilling well. A similar condition can occur when high velocity occurs near a bubble-gauge orifice. In some cases, the drawdown or build-up can be very large, on the order of 0.15 m (0.5 ft) or more. This condition should be investigated by making simultaneous readings of outside and inside auxiliary gauges, or recorder readings, during periods of high stages, and/or high velocity. It can also be checked by determining outside and inside high-water elevations (see Section 4.8.6).

Hydraulically induced errors can be reduced or eliminated through the use of an intake static tube, or in the case of a bubble-gauge, an orifice static tube. Relocating the intakes or orifice to a zone of low velocity may also help. Where drawdown or build-up cannot be completely eliminated it may be necessary to develop an inside-outside gauge relationship to use for correcting inside gauge readings to represent the outside gauge height.

Recorder errors

Automatic stage recorders include analog (graphic) recorders, digital (punch tape) recorders and electronic data loggers. Analog and digital recorders are mechanical recorders that can have play in the drive chains, gears and/or other linkages. Graphic recorders may also experience paper expansion and/or reversal errors that can lead to data errors. Electronic recorders can have play in the mechanical float wheels and chain drives connected to them. With care, good maintenance and proper verification, some of these errors can be corrected.

Electronic data loggers may record incorrect stage readings because of errors or inconsistencies in equations and algorithms that convert electrical signals to recorded stages. This is usually a programming problem that can be corrected. Extreme temperatures could cause recording errors, however most electronic data loggers are rated to record accurately from about – 40° to + 60° Celsius, and some data loggers are rated for even greater extremes. Tests indicate that in almost all cases the data loggers record correctly at the extreme temperatures.

Retrieval errors

Data that are recorded correctly on a paper chart, or punch tape, can be retrieved correctly even if it must be done manually. Paper chart and paper tape data can, however, be difficult to retrieve if the data were recorded incorrectly. Retrieval errors resulting from electronic data loggers occur because downloading the data from the data logger to a field computer is an electronic process that can sometimes result in incorrect stage readings. An even more serious problem would be the complete loss of stage data. Data errors or loss can also occur when downloading the data from a removable data card to an office computer.

Verification errors

Stage readings require frequent and consistent verification to insure that errors are reduced or eliminated. Failure to perform proper verification standards can be the source of undetected stage errors that can be significant in some cases. Verification procedures include frequent reading of independent auxiliary gauges, comparison of inside and outside gauge readings, observation of high water marks, redundant recording of peaks and troughs by use of maximum/minimum indicators, use of crest stage gauges and regular maintenance of gauge datum by levels. These checks should be augmented as appropriate for unusual field conditions. Hydrographers* should notice and keep records of instrument performance, including comparisons of recorded stages with reference gauge reading and any corrections applied.

4.3 GAUGE STRUCTURES

Stream and reservoir gauges require some type of instrument shelter, and in the case of gauges that use float sensors they also require a stilling well. In many cases, these structures provide the protection necessary against vandalism and natural hazards such as rain, floods, wind and lightning. The following sections describe the various structures used around the world.

4.3.1 Stilling wells

The stilling well protects the float and dampens the fluctuations in the stream caused by wind and turbulence. Stilling wells are made of concrete, reinforced concrete, concrete block, concrete pipe, steel pipe, aluminum pipe, fiberglass reinforced plastic (GRP) and occasionally wood. They may be placed in the bank of the stream as shown in Figures I.4.1, I.4.2 and I.4.3 but often are placed directly in the stream and attached to bridge piers or abutments as shown in Figures I.4.4 and I.4.5.

* Some parts of the world use the term hydrometrist, hydrologist, or engineer. For purposes of this report, the term hydrographer will be used to designate the person doing field and office work related to stream gauging and stream flow computations.

Figure I.4.1. Reinforced concrete stilling well and shelter

Figure I.4.2. Corrugated-galvanized-steel stilling well and shelter

Figure I.4.3. Concrete block stilling well and shelter

Figure I.4.4. Steel pipe stilling well and shelter attached to bridge wingwall

The stilling well should be deep enough for its bottom to be at least 0.3 m (1 ft) below the minimum stage anticipated and its top above the level of the 200 year flood. The inside of the well should be big enough to permit free operation of all equipment. Normally a pipe 1.2 m (4 ft) in diameter or a well with inside dimensions of 1.2 m by 1.2 m (4 ft by 4 ft) is of satisfactory size, but pipes as small as 0.5 m (1.5 ft) in diameter have been used for temporary installations where equipment requirements are not substantial. The 1.2 m by 1.2 m (4 ft by 4 ft) well provides ample space for the hydrographer to enter the well from the top, via a ladder, to clean it or to repair equipment. The smaller metal wells and the deep wells should have doors at various elevations to facilitate easy entry for cleaning and repairing. All confined space entry by personnel should follow appropriate safety rules and regulations.

When placed in the bank of the stream the stilling well should have a sealed bottom so that ground water cannot seep in nor stream water leak out. Water from the stream enters and leaves the stilling well through one or more intakes so that the water in the well is at the same elevation as the water in the stream. If the stilling well is in the bank of the stream the intake consists of a length of pipe connecting the stilling well and the stream. The intake should be at an elevation at least 0.15 m (0.5 ft) lower than the lowest expected stage in the stream and at least 0.15 m (0.5-ft) above the bottom of the stilling well to prevent silt buildup from plugging the intake. In cold climates the intake should be below the frost line. If the well is placed in the stream, holes drilled in the stilling well may act as intakes, taking the place of pipe intakes. Some wells placed in the stream have a sloping hopper bottom that serves as an intake. These are designed to allow silt to slide out of the stilling well thus preventing a buildup of silt that might cause a loss of gauge height record.

Figure I.4.5. Corrugated steel pipe stilling well and shelter attached to bridge pier

Two or more pipe intakes are commonly installed at vertical intervals of about 0.3 m (1 ft). During high water silt may cover the stream end of the lower intakes while the higher ones will continue to operate. The intakes should be properly located and sized to minimize surge.

Stations that have intakes subject to blockage are provided with flushing systems as shown in Figure I.4.6, whereby water under several feet of head can be applied to the gauge-well end of an intake. Ordinarily a pump raises water from the well to an elevated tank. The water is then released through the intake by operation of a valve. Intakes without flushing systems may be cleaned with a plumber's snake or by building up a head of water in the well with a portable pump or buckets to force an obstruction out of the intakes. Figure I.4.7 shows a typical silt flushing system.

At some stations where silt is a persistent and recurring problem a silt trap is constructed. The silt trap is a low well located between the main stilling well and the stream and through which the intakes pass before reaching the main stilling well. Baffles are sometimes placed in the silt trap to facilitate settlement of the silt in the silt trap before reaching the main stilling well, thereby leaving the main stilling well clear of silt so that it can function without clogging. The silt trap usually has a large entrance on top so that it can be accessed and cleaned easily. Figure I.4.8 is a schematic illustrating a typical silt trap installation.

The intakes for stilling wells placed in the bank of the stream are usually galvanized-steel, fireclay or concrete pipe. The most common size used is 0.05 m (2-inch) diameter pipe, but in some places up to 0.1 m (4-inch) diameter pipe is used. After the size and location of the well have been decided the size and number of intakes should be determined. The intake pipes should be of sufficient number and size for the water in the well to follow the rise and fall of stage without significant delay. It should be understood that there will always be some intake lag during a change in stage, but it can be minimized if sufficient intakes are provided. This equation, although not exact, will provide a reasonable indication of the lag that can be expected for various combinations of intakes. An intake system designed to keep intake lag at 0.03 m (0.1 ft) or less, for the maximum expected rate of rise or fall, is probably adequate.

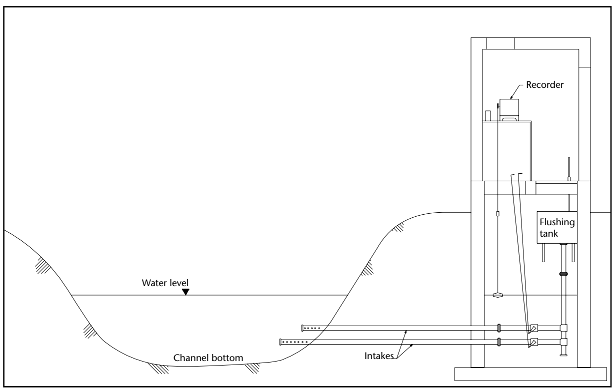

Figure I.4.6. Schematic of typical flushing system for intakes

Figure I.4.7. Intake silt flushing system

$$\Delta h = \frac{0.01}{g}\frac{L}{D}\left(\frac{A_w}{A_p}\right)^2\left(\frac{dh}{dt}\right)^2 \qquad (4.4)$$

where, Δh = lag, in mat any given time, g = acceleration of gravity, = 9.81 m s^{-1}, L = average length of intake pipes, in m, D = average diameter of intake pipes, in m, A_w = area of stilling well, in square m, A_p = combined cross-sectional area of intake pipes, in square m, and dh/dt = rate-of-change of stage, in m per second.

If two or more intakes are present, of different sizes and lengths, use the average length and the average diameter of the pipes for L and D respectively. However, use the total cross-section area of all of

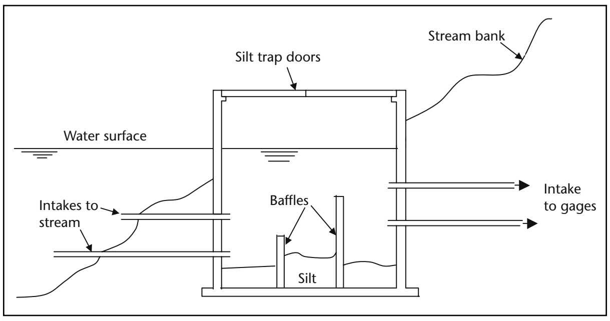

Figure I.4.8. Schematic of typical in-bank silt trap

the pipes for A_p. These recommendations will not give exact results for intake lag but will be approximately correct.

Smith, Hanson and Cruff (1965) have studied intake lag in stilling well systems relating it to the rate of change of stage of the stream and to various types and sizes of components which are used in the stilling well intake system. Their study provides additional information to account for various pipe fittings, such as valves, tees and static tubes.

The intake pipes should be placed at right angles to the direction of flow and should be level. If the velocity past the ends of the intakes is high, drawdown or pileup of the water level in the stilling well may occur. To reduce the drawdown effect static tubes can be attached on the stream end of the intake pipes. A static tube is a short length of pipe attached to an elbow or tee on the end of the intake pipe and extending horizontally downstream. The end of the static tube is capped and water enters or leaves through holes drilled in the tube.

For a bubble-gauge station equipped with a gas-purge system, a special orifice static tube should be used. This will be described in the subsequent sections of this Manual dealing with bubble gauges.

The usual methods of preventing the formation of ice in a stilling well are insulating measures such as sub-floors and heaters. Sub-floors are effective if the station is placed in the bank and has plenty of earth fill around it. If the sub-floor is built in the well below the frost line in the ground, ice will not normally form in the well as long as the stage remains below the sub-floor. Holes are cut in the sub-floor for the recorder float and weights to pass through and removable covers are placed over the holes. Sub-floors prevent air circulation in the well and the attendant heat transfer.

Figure I.4.9. Instrument shelter located on a stream bank

An electric heater or heat lamps with reflectors may be used to keep the well free of ice. The cost of operation and the availability of electric service at the gauging station are governing factors. Heating cables can be placed in intake pipes to prevent ice from forming.

Oil is sometimes used in oil cylinders placed in the stilling well to prevent freezing, however the danger of leakage of the oil from the cylinder to the stream makes it necessary to highly discourage this practice. The United States Geological Survey (USGS) uses a food grade vegetable oil called Isopar that may be acceptable in some other places. Oil should never be placed directly in the stilling well. Likewise, an oil cylinder should not be used if there is a danger that the oil could escape to the stream. It is recommended that wherever possible an alternative measure such as an insulated sub-floor or heater should be used. Pressure transducers and bubble gauges, as described in subsequent sections of this report, are also alternatives.

4.3.2 **Instrument shelters**

Instrument shelters are made of almost every building material available and in various sizes and shapes depending on local custom and conditions. See Figures I.4.5 and I.4.7 for examples of shelters placed on top of stilling wells. Instrument shelters are also required for gauges, such as bubble gauges, that do not require stilling wells. These may be placed on a concrete slab or other suitable foundation directly on a stream bank, on a bridge or on some other structure located near the stream, as shown in Figures I.4.9 and I.4.10. Many instrument shelters have an electronic data logger with antenna for data transmission and solar panels for maintaining battery charge. Some type of mast is required for these sites.

A walk-in shelter is the most convenient type, allowing the hydrographer to enter standing and be protected from the weather. A shelter with inside dimensions 1.2 m by 1.2 m (4 by 4 ft) with ceiling height 2.1 m (7 ft) above the floor is about the ideal size, either for a stilling well gauge or a bubble gauge where a stilling well is not required.

Look-in shelters are sometimes used at sites where a limited amount of equipment is to be installed and a portable and inexpensive shelter is desired. Figures I.4.11 is an example of a small look-in type of shelter. Figure I.4.12 is an example of a more elaborate type of look-in shelter over a stilling well.

In humid climates shelters should be well ventilated and have a tight floor to prevent entry of water vapor from the well. Screening and other barriers should be used over ventilators and other open places in the well and shelter to prevent the entry of insects, rodents and reptiles.

Instrument shelters not requiring a stilling well, such as those for a bubble gauge or pressure transducer, may be installed at any convenient location above the reach of floodwaters. Shelters similar to those in Figures I.4.9 through I.4.11 would be adequate. Such a gauge may be used to take advantage of existing features in a stream without costly excavation for well or intake and without need for any external structural support. The bubble orifice or pressure sensor is placed at least 0.5 foot below the lowest expected stage in the stream. The plastic tube or cable connecting the orifice or pressure sensor and the instrument is encased in metal conduit or buried to protect it from the elements, animals and vandalism. The pressure sensor is especially well suited for short-term installations because the entire station is readily dismantled and relocated with practically no loss of investment.

Figure I.4.10. Instrument shelter located on a bridge pier

Figure I.4.11. Look-in type of instrument shelter

4.3.3 Lightning protection

A lightning-protection system is needed for gauging structures to ensure uninterrupted data collection. This will minimize expensive repairs to instruments and equipment that might otherwise be damaged by lightning. The best and most effective lightning protection for instruments, such as satellite or cellular telephone data collection platforms (DCPs), electronic data loggers, stage sensors, telephone modems, computers, and other microprocessor-based instrument systems, is protection designed for and built into the instrument circuitry. Built-in protection can more closely match the protection needs of the circuitry than can protection that is added to the instrument after it is manufactured. When built-in lightning protection is inadequate or not part of the equipment supplemental protection should be provided.

Supplemental protection includes alternating current power-line and telephone surge suppressor devices. When telephone lines are used, but ac power is not part of a system, a telephone line surge suppressor should be used. Coaxial cables used for antennas should have a transient protector device to protect the Data Collection Platforms (DCP) transmitter, not only from lightning but also from voltage differences in an electrostatic discharge. Sensor lines should be grounded and protected from induced lightning transients.

An effective low-resistance and low impedance grounding system is also required. Common-point grounding is necessary to keep the system components within the gauge house at the same voltage potential relative to one another anytime the system becomes part of the lightning discharge circuit. The common-point ground should be connected to a low-resistance (5-10 ohm) earth ground. A grounding rod buried below the soil frost line provides a year-around, uniform, low-resistance ground. In some cases the stilling well may provide a low-resistance earth ground.

As many layers of lightning protection as possible should be employed. However, even with internal instrument protection and supplemental protection a direct lightning strike will likely destroy the electronic components.

4.4 INSTRUMENTATION

Many instruments are available for observing, sensing, recording and transmitting stage data. Such instrumentation ranges from the simple non-recording auxiliary gauges to sophisticated water level sensors, electronic data loggers and telemetry systems such as satellite DCPs. This section of Chapter 4 will describe most of the currently available instruments used for stage data collection in open channels and reservoirs.

4.4.1 Non-recording gauges

One method of obtaining a record of stage is by the systematic observations of a non-recording gauge. In the early days of stream gauging this was

Figure I.4.12. Look-in type of instrument shelter over a stilling well

the means generally used to obtain records of stage, and is still used at a few gauging stations, but today water-level sensors and automatic water-stage recorders are the predominant instruments used at practically all gauging stations. Non-recording gauges are still in general use as auxiliary and reference gauges at water-stage recorder installations to serve the following purposes:

(a) As an auxiliary or reference gauge to indicate the water-surface elevation in the stream or reservoir. This gauge is considered the outside gauge by most countries and is used for setting the automatic recorder;
(b) As an auxiliary or reference gauge to indicate the water-surface elevation in a stilling well. Readings from this gauge are compared to gauge readings in the stream to determine whether outside stream stage is accurately transmitted into the stilling well via the intake pipes. This gauge is considered the reference gauge in some countries, primarily the United States, however most countries consider this gauge as an auxiliary gauge;
(c) As a temporary substitute for the recorder when the intakes are plugged or there is an equipment failure. The auxiliary gauge in the stream or reservoir can be read as needed by a local observer to continue the record of stage during the malfunction.

The types of non-recording gauges generally used are staff, wire-weight, chain, float-tape and electric-tape. Staff, wire-weight and chain gauges are normally used as reference or auxiliary gauges at recording gauging stations. Float- and electric-tape gauges and the vertical staff gauge are used inside

Figure I.4.13. Standard metric, outside, vertical staff gauge, attached to 2 x 6 wood backing

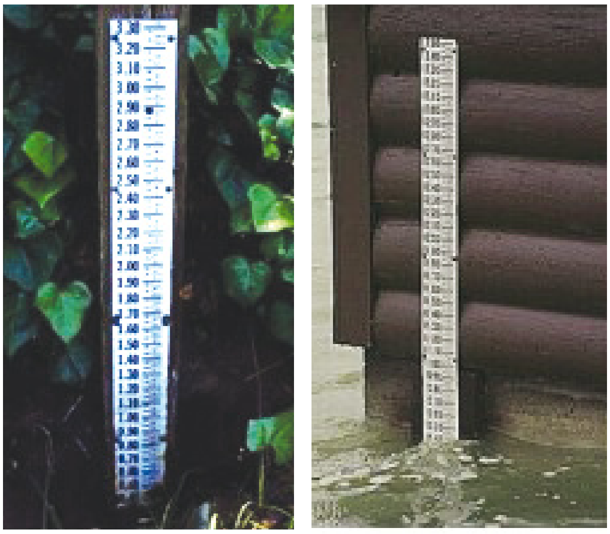

Figure I.4.14. Standard, outside, vertical staff gauge, used in the United States

stilling wells. Staff gauges are read directly whereas the other four types are read by measurement from a fixed point to the water.

Staff gauges

The staff gauge is either vertical or inclined. The standard vertical staff gauge in the United States consists of porcelain enameled iron sections 150 mm wide, 1 m long, and graduated every 10 mm. In Europe staff gauges are made from GRP, a fibre glass type of plastic. In the United States, the standard vertical staff gauge is 4 inches wide, 3.4 feet long, and graduated every 0.02 feet. Figure I.4.13 shows a standard metric, vertical, outside staff gauge. Figure I.4.14 shows a standard, vertical, staff gauge used in the United States. The vertical staff gauge is also used in stilling wells as an inside reference or auxiliary gauge. Vertical staff gauges are set by leveling directly to the gauges.

An inclined staff gauge is used for an outside gauge and usually consists of a graduated heavy timber securely attached to a permanent foundation. Inclined staff gauges built flush with the stream bank are less likely to be damaged by floods, floating ice or drift than are projecting vertical staffs. Inclined staff gauges must be individually calibrated by leveling to several points along the length of the gauge, interpolating intermediate points and marking these points with a relatively permanent marking system. An inclined staff gauge is shown in Figure I.4.15.

Figure I.4.15. Inclined staff gauge

Electric tape gauges

The electric-tape gauge, as shown in Figure I.4.16, consists of a steel tape graduated in feet and hundredths to which is fastened a cylindrical weight, a reel in a frame for the tape and a voltmeter. Terminals are provided so that a voltmeter can be connected to a battery. The negative terminal of the battery is attached to a ground connection and the positive terminal to the positive terminal of the voltmeter. The

Figure I.4.16. Electric-tape gauge

negative terminal of the voltmeter is connected to the weight, through the frame, reel and tape. One of two power supplies may be used with electric-tape gauges. Prior to 1989 electric-tape gauges were manufactured with a 5-volt meter, which requires a 4.5-volt battery. After 1989 electric-tape gauges were manufactured with a 15-volt meter, which requires a 12-volt battery. The old 5-volt meter can be replaced with a 15-volt meter so that the older model electric-tape gauges can be connected to a 12-volt battery.

The electric-tape gauge should be attached to the instrument shelf and insulated from the shelf if the shelf is metal. The negative terminal of the battery should be grounded to the stilling well if the stilling well is metal, such as a culvert-pipe well. If the stilling well is nonmetallic then the ground wire should be attached to a small metal plate which should be placed on the floor of the stilling well beneath the water surface. The positive terminal of the electric-tape gauge should be attached to the positive terminal of the battery.

Levels should be run to determine the gauge datum elevation of the index marker of the electric-tape gauge. Lower the weight so that at the index marker the tape reads an elevation somewhat less than the elevation of the index marker that was determined by levels. For example, lower the tape to read 0.3 m (1.00 ft) less than the elevation of the index marker. Hold the tape at this position and adjust the position of the weight on the tape so that the bottom of the weight is exactly 0.3 m (1.00 ft) below the index marker, as measured with a pocket tape or carpenter's rule. The electric-tape gauge is now set to read water levels precisely to gauge datum.

To determine the gauge height of the water level the weight is lowered until it barely contacts the water surface. This contact completes the electric circuit and produces a signal on the voltmeter. With the weight held in the position of first contact, the tape reading is observed at the index marker, which is the gauge height of the water level.

Wire-weight gauges

The type A wire-weight gauge (almost exclusive to the United States) is usually attached to a bridge handrail and is generally used as an outside auxiliary gauge. At some sites it is used as a reference gauge. It consists of a drum wound with a single layer of cable, a bronze weight attached to the end of the cable, a graduated disc, and a Veeder counter, all within a cast-aluminum box. Figure I.4.17 shows a type A wire-weight gauge. The disc is graduated in tenths and hundredths of a foot and is permanently connected to the counter and to the shaft of the drum. The disk is adjustable by loosening the set screws and moving the disk to the desired setting. The cable is made of 1 mm (0.045-inch) diameter stainless-steel wire and is guided to its position on the drum by a threading sheave. The reel is equipped with a pawl and ratchet for holding the weight within

Figure I.4.17. Type A wire-weight gauge

Figure I.4.18. Horizontally cantilevered wire-weight gauge

about 30 mm (0.1 ft) of any desired elevation. The diameter of the drum of the reel is such that each complete turn represents 0.3 m (l-foot) of movement of the weight. A horizontal checking bar is mounted at the lower edge of the instrument so that when the bar is moved to the forward position the bottom of the weight will rest on it.

The gauge should be set by lowering the weight to a position about 2 metres (about 6 ft) above the water surface and where leveling can be used to determine the elevation of the bottom of the weight. Hold the weight at this position and set the Veeder counter and graduated disk to read the same elevation as determined by levels. The elevation of the check bar should be determined by levels, and also by setting the weight on the check bar and reading the elevation from the Veeder counter and dial. These two elevations should be identical, however there will sometimes be a small difference, especially if the vertical distance between the gauge and low water is large. Both check bar elevations should be recorded and the one determined from the dial reading should be used for future checking to verify that the gauge adjustments have not changed. Some hydrographers record the correct check bar elevation, and the date it was determined, inside the wire-weight gauge box so it is readily available anytime the gauge is used.

The gauge height of the water surface is determined by lowering the weight to the water surface until it just touches the water surface. The Veeder counter and dial are read to obtain the gauge height. If there are waves or turbulence it may be necessary to take several readings at the crest and trough, and use the average of these for the water surface elevation. In very still water it is sometimes difficult to tell when the weight touches the water. Various methods are used, such as creating a slight pendulum motion that will disturb the water surface at the low point of the swing. Another method is to lower the weight into the water a few hundredths of a foot, and raise it a hundredth at a time, each time making a quick upward movement of the weight. If the weight is in the water or just at the water surface the quick vertical movement will create a visible disturbance of the water surface. The elevation where no disturbance is noted indicates that the previous elevation is the water surface elevation.

A unique installation of a wire-weight gauge is the horizontally cantilevered wire-weight gauge. This gauge is installed on a cantilevered arm that extends over the stream so that no part of the gauge is in the water. The wire-weight gauge is connected to an electrical circuit with a battery and voltmeter similar to the electric tape gauge described in the previous section. When the weight touches the water, the electrical circuit is closed which is visible on the voltmeter, and the wire-weight gauge dial can then be read to determine the gauge height. Figure I.4.18 shows an example of the horizontally cantilevered wire-weight gauge.

Chain gauges

A chain gauge is used where outside staff gauges are hard to maintain and where a bridge, dock or other structure over the water is not available for the location of a wire-weight gauge. The chain gauge can be mounted on a cantilevered arm which extends out over the stream, or which is made in such a way that it can be tilted to extend over the stream.

The chain gauge consists of the cantilevered arm which is held permanently in place, one or more enamel gauge sections mounted horizontally on the cantilever, and a heavy sash chain which runs over a pulley on the streamward end of the cantilever. A weight is attached to the streamward end of the chain, and a marker is attached to the chain near the other end. Additional markers can be attached to the chain at appropriate intervals to obtain gauge heights greater than that directly obtainable from the mounted gauge sections. The chain is mounted so that it moves along the gauge sections.

The chain gauge is set usually by leveling to the bottom of the weight, similar to the method described in the previous section for wire-weight gauges. The gauge plates, or the marker on the chain, can be adjusted to read the correct elevation.

Stage is determined by lowering the weight until the bottom of the weight just touches the water surface, just as described previously for a wire-weight gauge. The gauge height then is read from the mounted gauge plate at the location of the appropriate chain marker.

Float-tape gauges

The float-tape gauge consists of a float, a graduated steel tape, a stainless steel counterweight and a pulley, as shown in Figure I.4.19. The float pulley is usually 0.15 m (6 inches) in diameter, grooved on the circumference to accommodate the tape, and mounted in a standard. An arm extends from the standard to a point slightly beyond the tape to carry an adjustable index. The tape is connected to the float by a clamp that also may be used for making adjustments to the tape reading if the adjustments necessary are too large to be accommodated by the adjustable index. Floats ranging from 0.05 m (2 inches) to 0.30 m (12 inches) in diameter are commonly made of plastic, although older floats were made of copper. The float-tape gauge is used chiefly as an inside reference gauge.

The float-tape gauge can be an independent assembly, as shown in Figure I.4.19, or it can be an integral part of a strip-chart recorder or an analog-digital water-stage recorder as shown in Figures I.4.34. and I.4.35. It can also be an integral part of an electronic encoder interfaced to a DCP or electronic data logger as shown in Figure I.4.37. In the United States the index marker is set by determining the water surface elevation by reading the reference gauge in the stilling well and setting the index marker to read the same on the float tape. In most other countries the index marker is set by determining the water surface elevation from a reference gauge in the stream or reservoir and setting the index marker to that reading.

Float tape maximum and minimum stage indicators

An advantage of a float-tape gauge is that maximum and minimum stage indicators can be used on the tape so that the maximum and minimum stage can be determined for the period between station visits. These indicators are wire clips (similar to a paper clip) or small magnets that are attached to the float tape beneath the instrument shelf. However, magnets will not work on stainless steel tapes. The clips or magnets are designed so they slide easily on the float tape and are large enough that they will not pass through the float-tape hole in the instrument shelf. As the stage rises, the clip or magnet will come against the instrument shelf and slide along the tape until the stage reaches a peak and begins to recede. The clip or magnet will then retain its position on the tape, unless a higher stage occurs at a later date. The hydrographer, during the inspection visit to the gauge, can then raise the float until the clip or magnet just touches the instrument shelf and read the tape indicator at that

Figure I.4.19. Float-tape gauge and various float types

point to determine the peak stage. Alternately, the float-tape reading at the clip or magnet can be read, and a correction subtracted from this reading that will account for distance between the float-tape indicator and the bottom of the instrument shelf.

A wire clip or magnet, as described above, can also be used on the counterweight side of the float tape to determine the minimum stage that occurred since the last visit to the gauging station. The operation of the minimum stage indicator is similar to the peak stage indicator as described above. After obtaining the readings of both the maximum and minimum stage indicators, the clips or magnets should be reset on the float tape before leaving the station so they are against the bottom of the instrument shelf.

Crest-stage gauges

The crest-stage gauge is a simple, economical, reliable and easily installed device for obtaining the elevation of the flood crest. Although many different types of crest-stage gauges have been tested the one found most satisfactory is a vertical piece of 50 mm (2-inch) galvanized pipe containing a wood or aluminum staff held in a fixed position with relation to a datum reference, as shown in Figure I.4.20. The bottom cap has six intake holes located around the circumference of the cap so that drawdown or head build-up inside the pipe is kept to a minimum. Five of the holes are located on the upstream facing side of the cap and one hole on the downstream facing side, also shown in Figure I.4.20. Tests have shown that this arrangement and orientation of intake holes will be effective with velocities up to 3 metres per second (10 feet per second) and at angles up to 30 degrees with the direction of flow. The top cap contains one small vent hole. For additional information see Friday (1965) and Carter and Gamble (1963).

The bottom cap or a perforated tin cup or copper screening in cup shape attached to the lower end of the staff contains re-granulated cork. As the water rises inside the pipe the cork floats on its surface. When the water reaches its peak and starts to recede the cork adheres to the staff inside the pipe, thereby retaining the crest stage of the flood. The gauge height of a peak is obtained by measuring the interval on the staff between the reference point and the flood mark. Scaling can be simplified by graduating the staff. The datum of the crest-stage gauge should be checked by levels run from a reference mark to the top of the staff or to the bottom cap.

4.4.2 Water level sensors

A water level sensor was defined in a previous section of this report as a device that automatically determines the vertical position of the water surface. Stage sensors are usually not a single device but a combination of two or more components that work together to sense the water level. The two most commonly used stage sensors include float-driven systems and gas-purge (bubbler) systems. Other less commonly used stage sensors are submersible pressure transducers, and non-contact methods such as acoustic, radar and optical (laser) systems. The following sections describe the various instruments used for automatically determining water levels.

Float-driven sensors

Basic float system

The basic float sensor consists of a float resting on the water surface in a stilling well. The float is attached to a tape or cable passing over a pulley with a counter weight attached to the other end of the tape or cable. This is identical to the float-tape reference gauge described in a previous section of this Manual, and as shown in Figure I.4.19. The float follows the rise and fall of the water level and if the system uses a graduated tape the water stage can be read visually by using an index marker. For automatically recording the water stage the pulley is connected with a shaft to

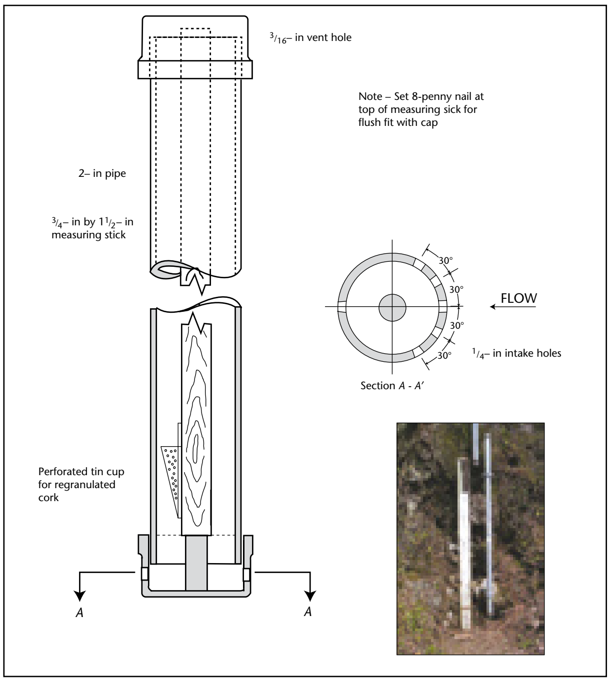

Figure I.4.20. Crest-stage gauge

an analog strip-chart recorder, or analog digital water-stage recorder, as shown in Figures I.4.34 and I.4.35.

Float an shaft encoder

A shaft encoder is a float-driven device that is connected with a shaft to the pulley of a basic float system. The shaft encoder interprets the rotational position and the number of revolutions of the shaft to determine the water stage. Shaft encoders may or may not have visual read-outs to indicate the water stage. In addition, shaft encoders are programmable to transmit the encoded stage to an Electronic Data Logger (EDL) or to a Data Collection Platform (DCP) based on user-specified instructions. Some shaft encoders have their own internal recording system such as a computer memory card.

Several commercial manufacturers in the United States and in Europe, such as Vaisala, Inc., Schmitz Engineering Liaison, Logotronic and Design Analysis Associates, Inc., make various models of shaft encoders. Figure I.4.21 shows a Vaisala model QSE104 shaft encoder and a Schmitz Engineering Liaison model 24MT shaft encoder, both of which are designed to transmit water stages to an EDL or DCP. Figure I.4.22 shows a WaterLOG model H-510 shaft encoder by Design Analysis Associates, Inc. which is designed to record water stage data on an internal computer memory card. Figure I.4.22 also shows a Gealog shaft encoder made by Logotronic.

The float system that drives a shaft encoder must produce enough initial force to overcome the starting torque of the encoder. For example, when the float is displaced 0.003 mm (0.01-ft), the driving force must be sufficient to produce movement of the encoder shaft. The driving force

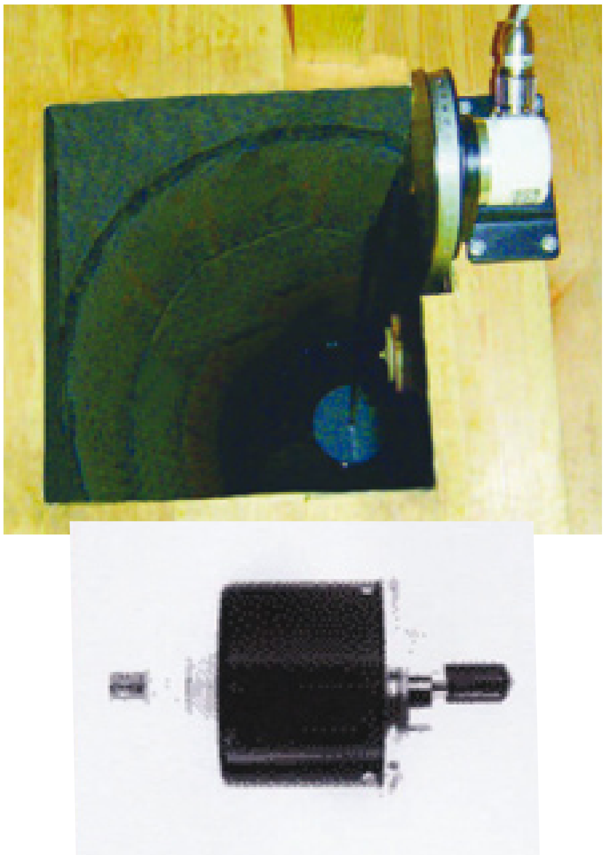

Figure I.4.21. Schmitz model 24MT (top) and Vaisala model QSE104 (bottom) shaft encoders

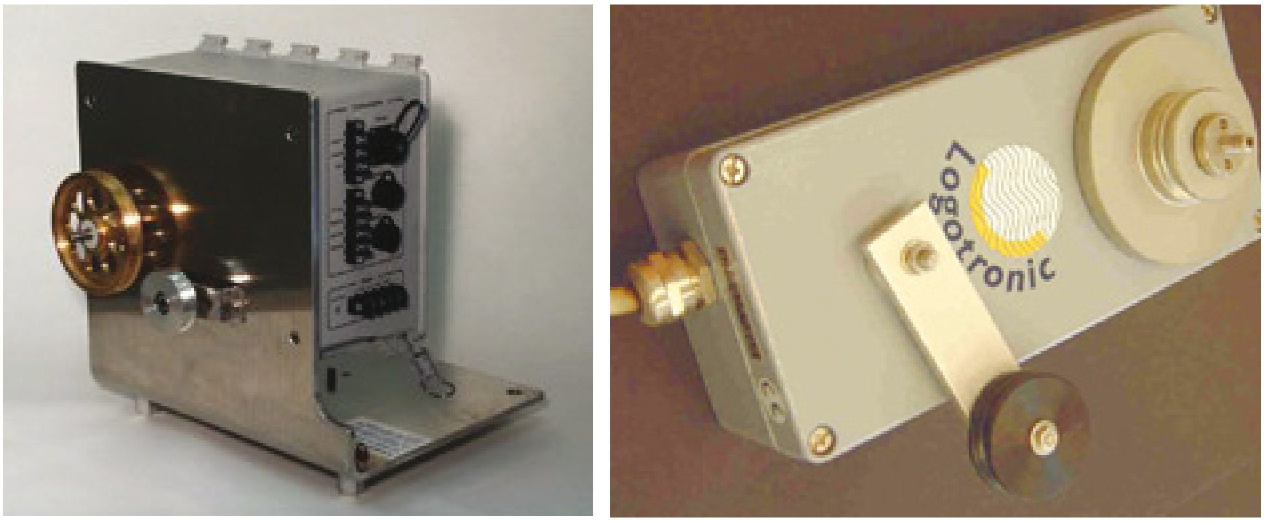

Figure I.4 22. Design Analysis Associates WaterLOG model H-510 shaft encoder (left) and the Logotronic Gealog shaft encoder (right)

will vary depending on the diameter of the float and on the diameter of the pulley attached to the encoder shaft. Table I.4.1 shows the driving force and torque developed for 0.003 mm (0.01-ft) movement of various sizes of floats and a 0.3 m (12-inch) pulley. To allow for other sources of error it is recommended that the starting torque of shaft encoders be no more than one-third of the values shown in Table I.4.1. Also, it is recommended that shaft encoders should have a starting torque that meets the criterion for a 40 mm (1.5-inch) float (less than 0.06 gram-meters). This would allow interchange of shaft encoders among gauging sites where smaller floats might be used.

Table I.4.1. Driving force and torque developed when a float of the indicated size is displaced by 0.003 mm (0.01-feet)

Float diameter, inches	12.00	10.00	2.50	1.50
Driving force, ounces	7.84	5.45	0.34	0.123
Driving torque, inch-ounce (12-inch pulley)	15.00	10.40	0.65	0.230
One-third of driving torque, inch-ounce	5.00	3.47	0.217	0.077

Float and potentiometer

The stage potentiometer, shown in Figure I.4.23, is designed as a stage sensor when used in combination with a basic float system. The stage potentiometer does not meet the accuracy requirements for primary stage records and errors of +/– 15 mm (0.05 ft) are common. However, it is useful for non-critical applications with moderate accuracy requirements, and where low cost is an important factor. Stage potentiometers should not be used where continuous float motion occurs. Continuous movement of the water surface can cause wear of the internal contacts of the potentiometer resulting in unreliable data.

Bubble gauges

The term *bubble gauge* is a general term that refers to various configurations of instrumentation designed to measure the water level on the basis of pressure differentials. A gas such as nitrogen is bubbled through a fixed orifice mounted in the stream and the water pressure at the orifice is transmitted through the gas tube to a pressure sensor located in the gauge house where it is converted to a measurement of stream stage. Original bubble gauges, first used in the early 1960 slipped, or s, used a mercury manometer to measure pressure differences. However, because of the hazardous nature of mercury, these manometers were banned in the late 1980 slipped, or s, and installation of new mercury manometers was no longer permitted. All existing mercury manometers were scheduled for removal over a period of the next few years.

Modern day bubble gauges use some type of non-submersible pressure transducer as the method of measuring pressure differentials. A number of different pressure transducers, some of which are described in the following sections of this Manual, are available that meet accuracy requirements.

Two essential components of a bubble gauge system, in addition to the pressure sensors, are (a) a gas-purge system, and (b) a bubble-gauge orifice. These are described in the following sections.

Gas-purge systems

The gas-purge system is a critical component of a bubble gas system. It is designed to feed a gas, usually nitrogen, through a system of valves, regulators and tubing to an orifice located at a fixed elevation in the stream. The continuous formation of bubbles at the orifice transmits the pressure head (depth of water over the orifice) caused by the stream stage to the pressure sensor located in the gauge house. The pressure sensor is vented to the atmosphere to compensate for barometric pressure changes.

Several gas-purge systems are available for use with bubble gauges. The standard United States

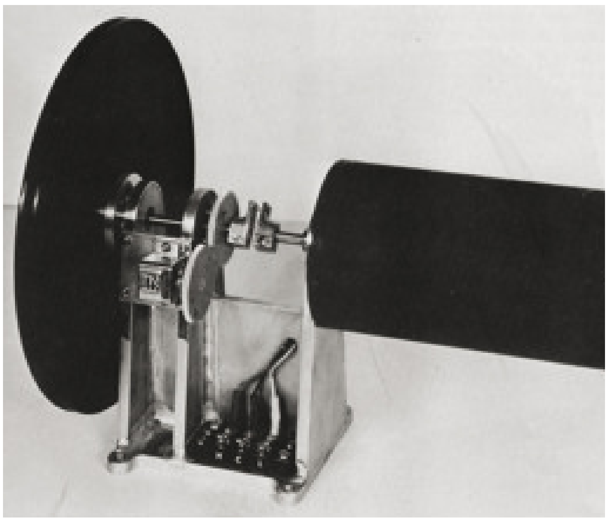

Figure I.4.23. Stage potentiometer

Figure I.4.24. Conoflow gas-purge system

Figure I.4.25. Hydrological Services Model HS-23 dry bubble gas purge unit

Geological Survey (USGS) Conoflow bubbler system is the most commonly used gas-purge system in the United States. Other gas-purge systems include the Hydrological Services Model HS-23 dry bubble unit, the Fluid Data Systems Safe Purge II unit, the Design Analysis Associates Model H-355 and the Sutron Accububbler.

The Conoflow bubbler system is shown in Figure I.4.24. It consists of the Conoflow differential regulator, the sight-feed and needle-valve assembly and various valves and tubing. The sight-feed and needle-valve assembly includes an oil reservoir where the bubble rate can be visually adjusted. One tube leads to the stream and orifice, and another tube leads to the pressure sensor. The Conoflow bubbler system is a proven method that meets both ISO and USGS accuracy standards.

The Hydrological Services Model HS-23 dry bubble unit is a gas-purge system that converts water-level head to pressure in conjunction with a water-level pressure-sensing instrument. It is designed to supply a constant bubble rate by way of a regulator. Bubble rate is set by rotating a micrometer dial, thus eliminating the need for a sight glass and oil reservoir as used in the Conoflow system. The unit has a gas-supply inlet for use with dry nitrogen, an instrument outlet for connection to a pressure-sensing device, and an orifice outlet for head-pressure measurement and system-purging operations. Tests have shown that it compares favourably with the Conoflow system. Figure I.4.25 shows an HS-23 dry bubble unit.

The Fluid Data Systems Safe Purge II (SPII) hydrologic purge-gas control unit is a dry bubble unit and is similar to the previously described HS-23 unit in that it eliminates the need for a sight glass and oil reservoir. It is also similar in how it connects to the gas supply, orifice and instrument lines. The bubble rate is not regulated and therefore is not constant, but varies with stream stage. Tests have shown that it compares favourably to the Conoflow system.

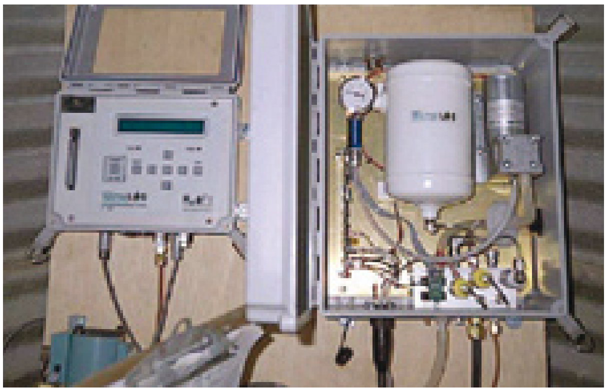

Figure I.4.26. Design Analysis Associates Model H-355 self-contained gas-purge system

The Design Analysis Associates Model H-355 and the Sutron Accububbler model 5600-0131-1 are self-contained gas-purge systems with pressure sensors and controller units. They do not produce constant bubble rates but instead purge the orifice line before each scheduled reading. These systems can be automatic and/or manually controlled to purge the orifice line. They wait for the bubble turbulence to subside before taking a stage reading. These types of systems are considered an advantage in streams where sediment may cause clogging of the orifice lines. Tests have shown that they compare favorably to the Conoflow system. Figures I.4.26 and I.4.27 show the Design Analysis Associates Model H-355 self-contained gas purging system and the Accububbler model 5600-0131-1 gas-purge units, respectively.

Bubble-gauge orifices

The gas-purge system bubbles a gas into the stream through an orifice. The standard USGS orifice is mounted in a 300 mm (2-inch) pipe cap so that it can be attached to a 300 mm (2-inch) pipe. Figure I.4.28 is a detailed drawing of an orifice assembly and Figure I.4.29 shows an inside and outside view of a standard USGS orifice assembly.

The orifice assembly is installed so the orifice remains at a fixed elevation in the stream and the proper placement in the stream is essential for obtaining an accurate record of stage. If possible it should be installed where stream currents are not high and where sediment accumulations are not likely to cover the orifice. If high velocities are expected near the orifice it is recommended that it be installed inside a static tube, which should be a vertical mount perpendicular to the direction of flow. Details of a standard orifice static tube are shown in Figure I.4.30. A more detailed description for the installation of a bubble-gauge orifice is given by Craig, 1983.

Another option for high velocity situations is the use of an orifice gas chamber. The chamber is a bell-shaped housing with a bottom plate that contains numerous holes. The chamber is attached to the orifice with the plate facing down; this allows the formation of a large bubble of gas. The large bubble stabilizes the line pressure and also can eliminate painting that is sometimes seen at bubble gauges. They are vented to the atmosphere to compensate for barometric pressure changes.

Non-submersible pressure transducers

Non-submersible pressure transducers are generally used as the pressure sensor for bubble-gauge

Figure I.4.27. Sutron Accububbler model 5600-0131-1 self-contained gas-purge system

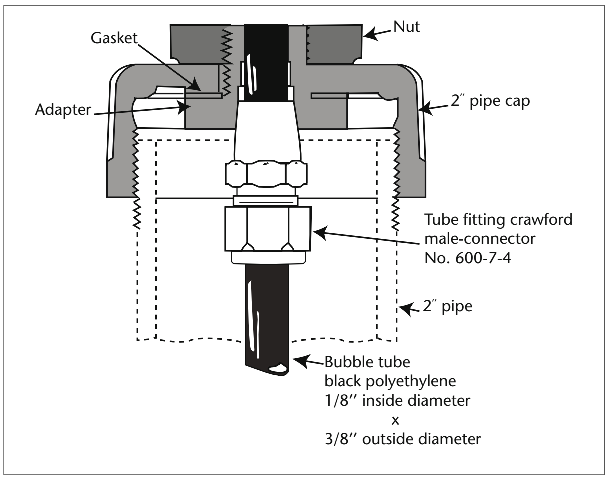

Figure I.4.28. Details of a bubble-orifice assembly

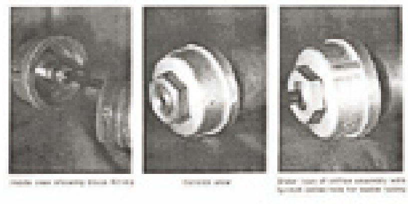

Figure I.4.29. Inside and outside view of a bubble-orifice assembly

systems. These transducers are connected to the gas-purge unit to receive the pressure input from the stream. The transducers are internally programmed to convert the gas pressure to units of water head (Metres of water over the orifice) and to then transmit the data to an Electronic Data Logger (EDL) or Data Collection Platform (DCP). Pressure transducers generally have internal compensation for temperature changes and various other adjustment provisions to compensate for water density variations, purge-gas weight, local gravity variations and gauge-datum adjustments.

Three non-submersible pressure transducers tested by the USGS that have been found to meet the accuracy requirements are: (a) the Paroscientific Model PS-2, (b) the Sutron Accubar Model 5600-0125-3 (Accubar-3), and (c) the Design Analysis Associates Model H-350 LITE. These are shown in Figures I.4.31, I.4.32 and I.4.33.

Submersible pressure transducers

A number of submersible pressure transducers are commercially available for use primarily in ground-water wells. However, in some applications they have been used for streams and reservoirs.

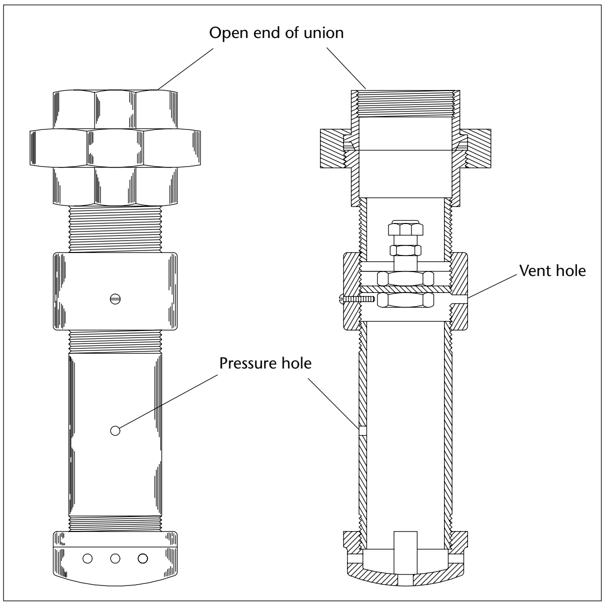

Figure I.4.30. Details of a standard orifice static tube

Submersible pressure transducers are usually self-contained units having a transducer element and electronic circuitry. Some also contain a data logger and battery. These units can be mounted directly in the stream or reservoir. Water level is determined by the difference between hydrostatics pressure and atmospheric pressure. Relative submersible pressure sensors have an integrated vented tube to atmosphere for measuring and compensating for atmospheric pressure. Absolute submersible pressure sensors measure only hydrostatic pressure and atmospheric pressure is measured by additional pressure sensor integrated with an EDL. Temperature variations affect the accuracy of pressure measurement and traditional units do not generally meet the accuracy requirements for primary stage recorders. Currently there are units on the market that have an integrated temperature sensor. Effect of temperature variations is corrected by mathematical calculations in the sensor electronic circuitry and measurement accuracy is substantially better. Submersible units may be damaged or destroyed by freezing temperatures thus they must not be placed in streams or reservoirs that freeze around the sensor.

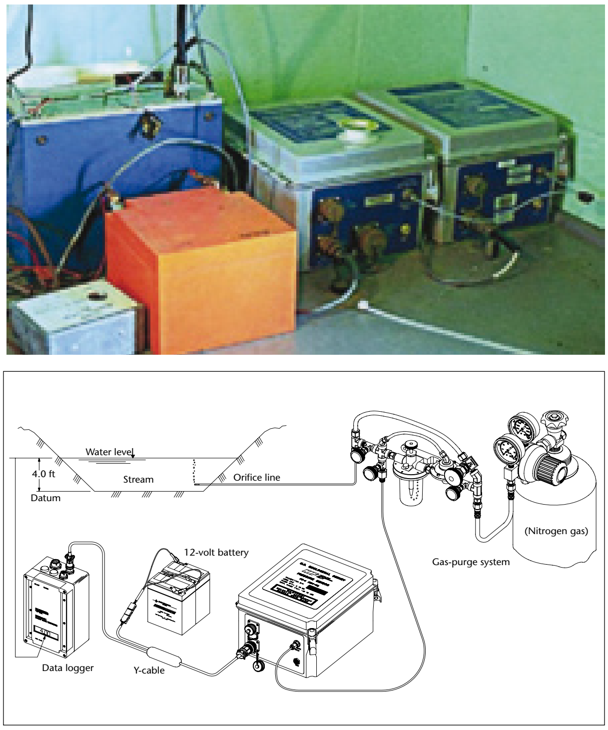

Figure I.4.31. Paroscientific Model PS-2 non-submersible pressure transducer

Non-contact water-level sensors

Almost all of today's instrumentation for measuring water levels requires that some part of the stage-sensing element be in contact with the water. The use of ultrasonic, radar and optical methods are used with instrumentation that will eventually provide accuracy and convenience of measuring water surfaces without direct contact. Most of these types of instruments are still in the development stage and consequently are relatively expensive. Some are not yet able to achieve the accuracy standards for primary

Figure I.4.32. Sutron Accubar Model 5600-0125-3 (Accubar-3) non-submersible pressure transducer

Figure I.4.33. Design Analysis Associates Model H-350 LITE non-submersible pressure transducer

stage records. However, in some countries, such as the United Kingdom of Great Britain and Northern Ireland, air ranging and upward looking ultrasonics (in water) have been used for about 20 years. The following sections provide a brief description of non-contact water-level sensors.

Ultrasonic (acoustic)

The ultrasonic method uses a high-frequency acoustic transducer that propagates a sound wave through the air to the water surface. The reflected acoustic wave is received at the transducer, converted to an electrical signal and processed into a water-surface stage. The transmitted and reflected signals are affected by air temperature and air density, which in turn affects accuracy, especially when the vertical distance between the transducer and the water surface is large (that is greater than about 3 m, or 10 feet). Some manufacturers have temperature compensation built into their devices. Ultrasonic depth sensors can also be installed below the water level, directing the beam upwards towards the water surface to determine water depth.

Radar

Radar (radio detecting and ranging), or radio wave transmission, is a distance measuring method that has been used since prior to World War II. A radio wave is the propagation of an electromagnetic field and, therefore, is performed at the speed of light. The advantages of radar are that the signal is generally immune to weather conditions and the radio wave used for this application is harmless to humans and wildlife. The useable sensor-to-water sensing range is typically from near zero to about 35 m (115 feet). The technology for using radar for measurement of water levels is still new, although several commercially developed instruments are available and are being tested.

Optical (Laser)

Optical methods of measuring distance are confined mainly to the use of laser slipped, or , technology. This technique shows considerable promise in measuring water level stage. Eye safety is a consideration with lasers because direct exposure of the eyes to a laser beam could be dangerous. For water level measurements this will not generally be a problem because the power levels of the beam are well within safe standards. As with radar a few commercially developed laser instruments are now available and are being tested.

4.4.3 Water-level recorders

A water-level recorder is an instrument that automatically records a continuous, or quasi-continuous, record of the water-surface stage with respect to time. Water-level recorders may be paper chart recorders (analog), paper punch-tape Analog Digital Recorders (ADR), Electronic Data Loggers (EDL), or Data Collection Platforms (DCP). The basic requirements for a recorder are to systematically and accurately keep a record of the gauge height with respect to time so that a stage hydrograph of the fluctuations of the water surface can be produced for archiving and analysis. Accuracy requirements are described in a previous section of this Manual.

Paper chart recorders

A paper chart recorder, also referred to as an analog recorder, provides a continuous graphical trace of water stage with respect to time. It was the primary type of stage recorder used from about 1900 until the mid-1960s when the ADR paper-punch tape

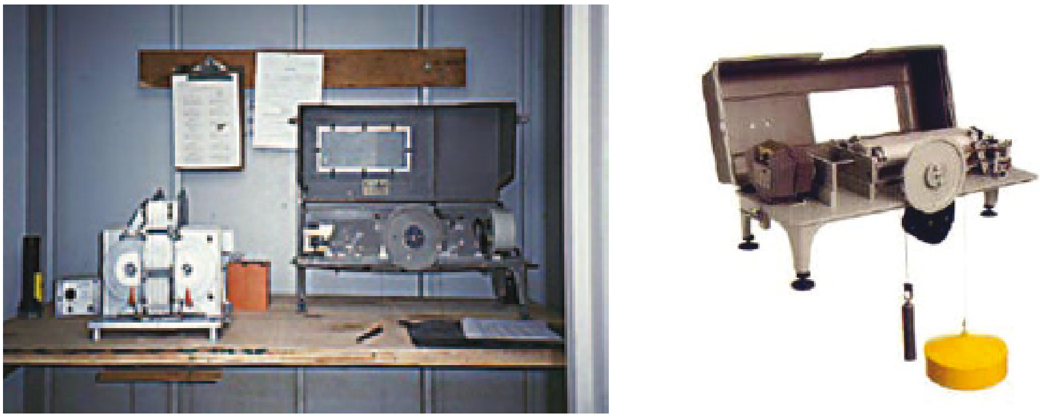

Figure I.4.34. Stevens analog digital recorder (left) and Stevens A-35 strip-chart recorder (right)

recorder was introduced. The paper chart recorder is still used at some gauging stations, but in most instances it is used only as a back-up recorder. The Stevens A-35 strip-chart recorder, as shown in Figure I.4.34, is the most common analog recorder, but there are variations such as the Au strip-chart recorder and the horizontal-drum recorder.

The Stevens A-35 recorder uses a roll of chart paper called a strip chart which is driven by a clock at a uniform speed to provide the time of the stage hydrograph. Clocks are either spring-driven or weight-driven and are reasonably accurate if properly adjusted. Timing errors of one to two hours are normal over a period of several weeks, however, larger errors can sometimes occur. In recent years many chart recorders have been retrofitted with an electronic driven clock which provides better accuracy than the older mechanical clocks. An ink pen and/or pencil that ride on the strip chart are connected through a series of gear-and-chain linkages to the float wheel. As the stage rises and falls, the pen and/or pencil draws a trace on the 250 mm (10-inch) wide chart at a fixed scale, usually 300 mm (1 foot) of stage per 50 mm (2 inches) of chart scale. Other scales can be used. A reversal mechanism allows the pen and/or pencil to reverse direction at the edge of the chart, thus providing a stage scale limited only by the vertical travel range of the float. Most strip-chart records will operate for several months without servicing (although this is not recommended) whereas the horizontal-drum recorder must be serviced at weekly intervals. A horizontal drum recorder can be fitted with different time scale gears for longer operation but with a reduction in definition. Attachments are available for the strip-chart recorder to record water temperature or rainfall on the same chart with stage.

The Stevens A-35 recorder is a time-proven instrument that served as the primary method of recording stream stage for many years. A major advantage of this recorder is that it produces a graphical record of stage that gives a visual picture of the rise and fall of the stream. Also, the graphical record provides an easy means of spotting malfunctions, such as blocked intakes. A disadvantage is that the graphical stage record cannot be easily scanned for electronic archiving and analysis.

Paper punch-tape recorders

Paper punch-tape recorders, commonly referred to as Analog Digital Recorders (ADR), were introduced in the early 1960s as a replacement of the paper strip-chart recorders. The most common type of ADR used by the USGS is the Stevens digital recorder shown in Figure I.4.35. Recorders of this type are still in use today as the primary stage recorder at many gauging stations, although they are being replaced by electronic data loggers and data collection platforms.

The analog digital recorder is a battery-operated, slow-speed, paper-tape recorder that punches a 4-digit number on a 16-channel paper tape at pre-selected time intervals, see Figure I.4.36. In the

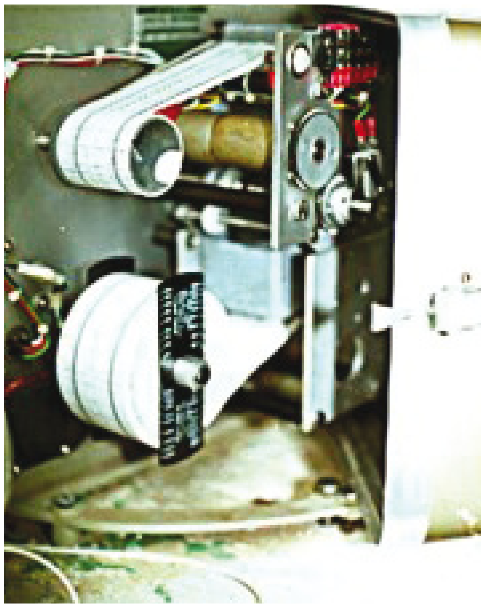

Figure I.4.35. Stevens analog digital recorder

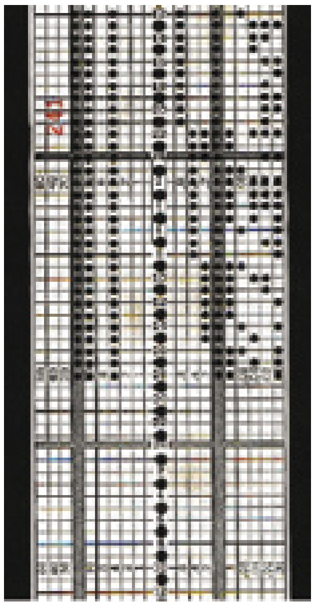

Figure I.4.36. Stevens analog punch tape

United States stage is recorded in increments of 3 mm (0.01 ft) from zero to 30.477 m (99.99 feet) and is transmitted to the instrument by rotation of the input shaft, which is driven by a basic float system. In other countries such as the United Kingdom, stage is recorded in increments of 1 mm or 1 cm, depending on the range of level measurement required. Shaft rotation is converted by the instrument into a coded punch-tape record that can be read visually from the tape, but is designed to be read by digital tape readers for input to an electronic computing system.

The code consists of four groups of four punches each. In each group the first punch represents “1,” the second “2” the third “4,” and the fourth “8.” Thus a combination of up to three punches in a group represents digits from one to nine, with a blank space for zero, and the four groups of punches represent all numbers from 1 to 9 999. Coding is done by means of two discs containing raised ridges in accordance with the punch code outlined above. One disc is mounted directly on the input shaft. The second code disc is connected to the first by a 100:1 worm gear so that one hundred revolutions of the input shaft rotate the second, or high-order disc, one complete revolution. A paper tape is moved upward through a punch block that is mounted on a movable arm hinged at the base of the recorder. The punch block contains a single row of 18 pins, 16 pins for the information punches and 2 for punching feed holes.

A major advantage of analog digital recorders over chart recorders is that the paper-punch tapes can be automatically read into a computing system for archiving and analysis. However, it is a slow and tedious process to read tapes by visual inspection. This makes field inspection of the stage record difficult for spotting malfunctions of the gauge. Also analog digital recorders may miss the absolute peak stage, especially on flashy streams. However, for recorders linked to a float tape gauge, a measure of the maximum peak that occurs between inspections of the recorder can be determined by using maximum stage indicators, as described in the section about peak stage indicators. The mechanically punched tape of an ADR is very practical for field use under widely varying conditions of temperature and moisture.

Electronic Data Loggers

Electronic Data Loggers (EDLs) are devices that can be programmed to electronically record stage data on a specific, regular time interval or on a user-defined schedule which may vary according to stage or other variable. The number of manufacturers and vendors are too lengthy to list in this report. In addition, new EDL models and new manufacturers are being offered frequently. For this reason it is not practical to provide a detailed description of specific manufacturers or models in this Manual, because such listings and descriptions are subject to change in a short time. EDLs are commonly combined with data collection platforms (DCP slipped, or s) that are described in the next section of this report.

EDLs are usually powered by an external battery which may be rechargeable. In many cases the batteries are charged through the use of a solar panel. Where available, power may be supplied from an ac source. Most also have internal batteries to maintain programming and data in the event of a power failure.

Electronic clocks are used in EDLs and can generally be relied upon to keep accurate time. See a subsequent section of this report on “Timers” about the method of setting the timers.

The stage of a stream or reservoir is sensed by either a float driven shaft encoder, by a bubble gauge and non-submersible pressure transducer arrangement, or by a submersible pressure transducer. The encoded stage data are then relayed to the EDL through a hard-wired or wireless connection for storage and/or transmittal to a remote site. See Figure I.4.37 for a typical installation of a float driven shaft encoder and EDL.

Electronic data loggers store stage data either internally on a memory module, or the EDL may store the data on a removable memory card. Most allow for both methods of data storage. Data are retrieved either by downloading the data directly from the EDL to a field computer or removing the memory card and transferring the data from the card to a field computer. The portable memory card may then be carried to the field office for downloading. Data may be retrieved also by transmitting the data by telephone, radio or satellite. This will be discussed in subsequent sections of this Manual.

The advantages of an EDL for data storage are that they can store large amounts of data without frequent servicing and they can be programmed to collect data according to specific needs. They can be connected to transmitting devices, such as radios, telephones or satellite systems easily so that data can be retrieved in near real-time mode. The internal electronic clocks of EDLs are generally very accurate. A major disadvantage is that the recorder

Figure I.4.37. Typical installation of float tape gauge, electronic shaft encoder, and electronic data logger

stage cannot be easily viewed in the field by graphical methods without downloading the data to a portable field computer that has plotting software designed for stage hydrograph viewing.

Data Collection Platforms

Data Collection Platforms (DCPs) are referred to in the commercial industry as a unit designed to acquire data and transfer the data to another location. Data may be transferred by a telephone line, cellular phone, a radio or satellite. The USGS uses the term DCP in reference to devices that collect the data and transfer it over the Geostationary Operational Environmental Satellite (GOES), which can be used in North America Region. The transmitter is considered an integral part of a DCP.

DCPs do not necessarily store substantial amounts of data however some units may provide for storage modules or cards that can store a large amount of data. In some cases the GOES transmitter may be an integral part of an electronic data logger or it may be connected to an EDL in such a way that large amounts of data can be stored as well as transmitted. Like EDLs, there are a number of makes and models of DCPs. Descriptions of specific models are not given in this report because those that are currently available are subject to change and new makes and models are being introduced frequently.

The power source for a DCP can be a rechargeable battery or a reliable ac source. Solar panels are frequently used to keep batteries charged and the same battery may provide power to other instruments in the gauge house. DCPs, HDRs in particular, may use considerably more power than an EDL because of the telemetry devices they employ. Care should be taken to make sure solar panel charged systems are adequate to keep them operational.

Electronic timers are used for DCPs and are generally very accurate. See a subsequent section of this report on "Timers" for a description of a device that can be used for setting the electronic clock of a DCP. Some HDR transmitters include internal atomic clocks.

DCPs receive stage data from a shaft encoder, submersible or non-submersible pressure transducer or bubble gauge, as described previously for EDLs. The data are transmitted on a pre-defined schedule, such as a 4-hour interval (1-hour in the case of HDRs), thus providing near real-time access to the data. During some critical events such as flooding the data transmission interval can be even more frequent, for example 15 minutes. Overlapping periods of data are transmitted each time a transmission is made to provide a certain amount of redundancy. This allows for the recovery of data that may sometimes be lost during a transmission.

4.4.4 **Telemetry Systems**

The primary telemetry method used by the USGS is the GOES satellite transmission system which is an integral part of Data Collection Platforms (DCPs) as described in the previous section of this Manual. Other telemetry systems for transmitting data from a gauging station to a remote location include electronic modems, cellular telephones, radio transmitting systems, new satellite technology, and HDR data collection platforms. Some of these are relatively new, and still in the development stage. Wireless data transmitting systems are in a rapid state of development with new systems on the horizon.

A short-range radio transmitter can be used to send data from a stage sensor, such as a float-tape gauge or bubble gauge, to a data logger. Two units are required, one for sending and one for receiving. An example of a radio system of this type is the Adcon SDI-12 Radio Link. This unit has a range of 1/8 to 1/4 mile; however this range can be increased by substituting a directional antennae for the omni-directional antennae that comes with the unit. It is basically a line-of-sight unit, but can tolerate minor obstructions. Other short-range radio transmitters are available from other manufacturers.

The primary telemetry method used by the USGS is the GOES satellite transmission system which is an integral part of DCPs as described in the previous section in this report. Using GOES is applicable only in North and South America. There are other satellite transmission providers that cover other parts of the world (Meteosat, Insat, Inmarsat, Iridium, Orbcomm). Most of these satellite transmission providers work on a commercial basis so data transmission by satellite can be expensive, especially if the data sending interval is short. Nevertheless, if the water level station is situated in a remote location and automatic real-time data collection is required as in flood warning systems, satellite transmission may be the only option.

Other telemetry systems for transmitting the data from a gauging station in a remote location include telephone network modems, Global System for Mobile Communications (GSM)/ General Packet Radio Service (GPRS) data modems and radio transmitters. Wireless data transmitting systems are in a rapid state of development with new systems on the horizon. One prominent option is to use GPRS which is provided by most GSM network operators. In addition to standard GSM data call operation, this service offers more functions. Data is transmitted as packages according to the user-defined intervals or when a certain limit value is exceeded. The data can be initiated by using File Transfer Protocol (FTP) and data are placed on the FTP server's hard disk. Another option is to use commercial service providers and send the data to an e-mail address. With GPRS, the stations are practically always on-line and all the data are immediately available. Using GPRS can be very cost-effective as most service providers charge only according to the amount of data transmitted.

Short range radio transmitters can be used to send data from water level stations. Two units are required, one for sending and one for receiving. The main limitation of the radio modems is that they require line-of-sight without any obstacles in the radio signal path. Typically, the signals can be transmitted only over distances of a few tens of kilometres. The distance can be increased by using additional radio modems as repeaters. Radio modems can be installed independently to act as repeaters, or a water level station having a radio modem can act as a repeater for other water level stations. Thus, it is possible to build an observation network of stations that communicate with each other via radio modems. One water level station can then be equipped with, for example, a satellite transmitter or a GSM/GPRS modem that transmits data of the entire network.

It should be noted that the various telemetry systems mentioned in this section require a small shelter and adequate power supplies which add to the cost and complexity of the gauge.

4.4.5 Timers

Timers for data recorders and data loggers range from mechanical clocks to highly sophisticated electronic clocks. These have been referenced in previous sections of this Manual that describe data recorders and DCPs. The accuracy of the time associated with each recorded stage is dependent on the accuracy of the timer, and the advent of electronic timers in recent years has resulted in highly accurate time records.

A portable, electronic clock can be used for setting the time on Electronic Data Loggers (EDLs), Data Collection Platforms (DCPs), and other instruments requiring an accurate time setting. In the United States, the clock receives a radio signal directly from the Atomic Clock (WWVB) in Boulder, Colorado. The unit uses an omni-directional antennae and reception of the signal automatically sets and displays the time, in hours, minutes, and seconds, on a liquid crystal display.

Commercial Global Positioning System (GPS) receivers can be used for adjusting internal time of EDLs or DCPs. The GPS receiver can be connected to the EDL or DCP and it receives accurate time signals from GPS satellites. Time of EDL or DCP internal clocks can be set according to this time signal, for instance once a week. GPS satellites cover most of the world, so these systems are applicable practically everywhere and by using them manual setting of timers.

4.4.6 Power supplies

Most gauging stations require electrical power to operate the various instruments. Dry cell and wet cell batteries are the usual dc power sources. However, if a reliable source of ac current is available it can be used, usually with an ac-to-dc converter. Very few, if any, instruments operate directly from ac current. If a system is operating from ac power it should be equipped with an appropriate Uninterrupted Power Supply (UPS) and surge protector. Automatic battery chargers that operate from ac power can be used to keep rechargeable batteries fully charged. Where ac

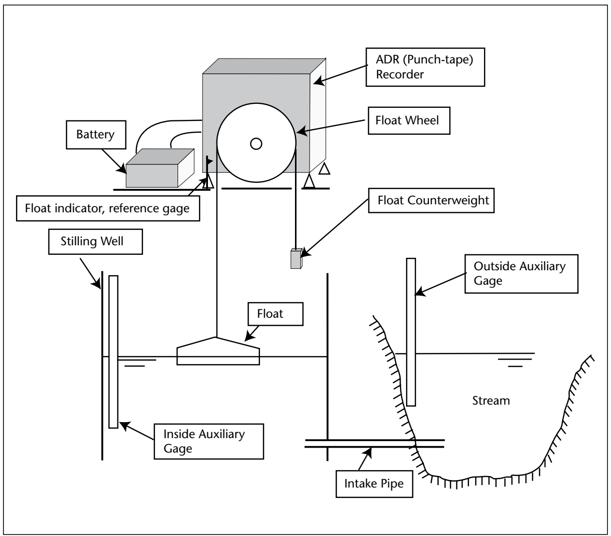

Figure I.4.38. Schematic of stilling well, float sensor, and non-electronic recorder

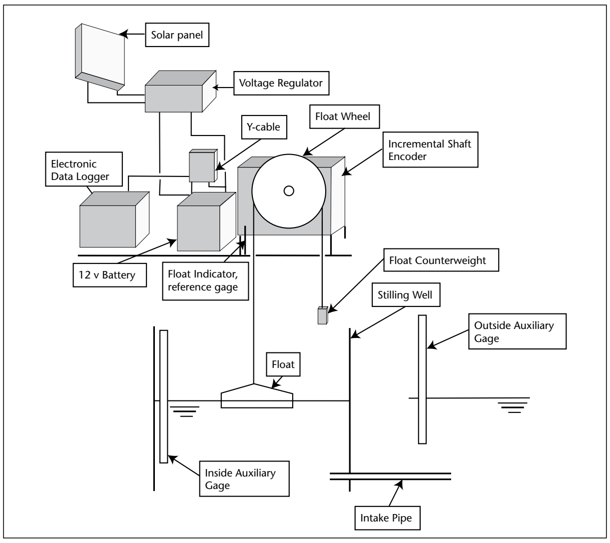

Figure I.4.39. Schematic of stilling well, float sensor, incremental shaft encoder, and electronic data logger

power is not available solar panels are frequently used as the primary method of recharging batteries.

4.5 TYPICAL GAUGING STATION INSTRUMENTATION CONFIGURATIONS

The diversity of instruments available for sensing, recording, and transmitting stage data results in several configurations of instrument setups that will provide stage data suitable for meeting primary stage accuracy requirements. In the following sections, three typical configurations are described. Other variations of these configurations are possible and likely.

4.5.1 Stilling well, float sensor and non-electronic water level recorder

The traditional, basic stage recording system, used since the early 1900s, consists of a stilling well located in or near the stream with a float used as the stage sensor. Stage is recorded by a graphical or digital (non-electronic) recorder. This has proven to be a very reliable system and is still used at many gauging stations today. The gauge used for setting the recorder differs in different countries. Some use the inside auxiliary gauge, some use the float tape indicator and some use the outside auxiliary gauge. Pictures of stilling wells are shown in Figures I.4.1 to I.4.5. Non-electronic recorders are shown in Figures I.4.34 and I.4.35. This configuration is shown schematically in Figure I.4.38.

4.5.2 Stilling well, float sensor, shaft encoder and electronic water level recorder

A variation of the configuration shown in Figure I.4.38 consists of the same stilling well and float sensor, but with an incremental shaft encoder (see Figures I.4.21 and I.4.22) and an electronic data logger. The encoded stage, as determined by the float-actuated shaft encoder, is passed to the EDL by a hard-wired connection. The inside reference gauge is used for setting the shaft encoder. This configuration is shown in Figure I.4.39. A data collection platform or other type of transmitting system can be added to this setup so that stage data can be transmitted to a remote location.

4.5.3 Instrument shelter, bubble gauge, non-submersible pressure transducer and electronic water level recorder/Data Collection Platforms

At gauge sites where a stilling well is not used or is not practical stage may be sensed with a pressure transducer coupled to a compressed-gas purge system (bubble gauge). Nitrogen is the usual type of gas used. Because of safety concerns and operation efficiency the compressed-gas purge system can be replaced with a self-contained system. The gas-purge system is connected with an orifice line to a bubble-gauge orifice that is located in the stream and to the pressure transducer located in the instrument shelter. This system is very accurate and replaces its predecessor, the

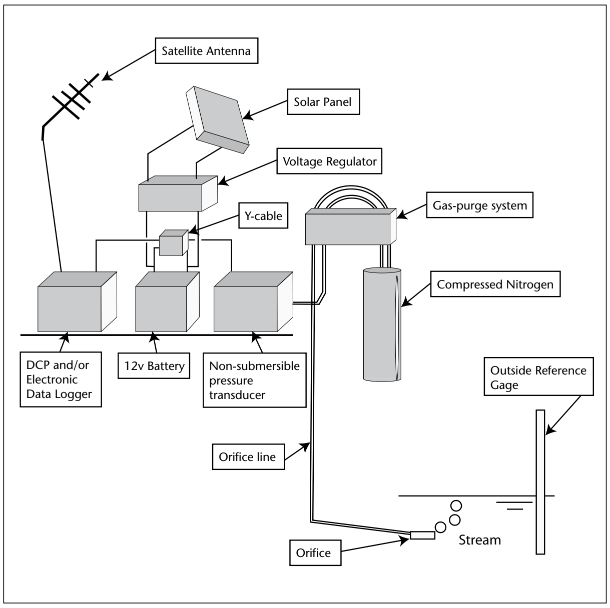

Figure I.4.40. Schematic of bubble gauge, non-submersible pressure transducer, DCP and electronic data logger

mercury manometer. The stage, as sensed by the pressure transducer, is passed to the EDL and/or DCP by a hard-wired connection. The encoded stage is stored for later transmittal to a satellite or for retrieval from the EDL at a later time. All instruments are housed in an instrument shelter. An outside reference gauge is used for setting the pressure transducer. This configuration is shown in Figure I.4.40.

4.6 DATA RETRIEVAL AND CONVERSION

The retrieval of stage data varies according to the type of recorder in use and to the urgency of acquiring the data. Data recorders that produce a paper chart or paper punch tape require manual retrieval during a visit to the gauging station. EDLs may also require a visit to the gauging station to retrieve the data, however if the EDL is part of a DCP configuration, the data are retrieved by radio transmission via a satellite. Data may also be retrieved from an EDL and/or DCP by downloading to a portable field computer, or by extracting a data card from the EDL and downloading data from the card to a computer. Data may be retrieved by transmission via telephone, radio or one of the other telemetry methods described in previous sections of this report. Some systems may be programmed to provide an automatic alert at critical stages such as flood stage or rate-of-change of stage.

All data retrieved by way of charts, tapes, electronic data cards and electronic downloads are considered original stage data and should be protected against loss or damage. In some cases even data that are transmitted via DCP and satellite, or other telemetry method, are considered original and must be preserved. Original data for automated data-collection sites are defined as unaltered data acquired from the primary sensor (and back-up sensor, as needed) and converted to engineering units and a standard format.

EDLs and DCPs record data in various formats that must be deciphered and converted into a standard format that can be easily used by data processing systems. The conversion software for performing these tasks will vary in different countries and is beyond the scope of this Manual. The requirements for data processing systems are described in Volume II of this Manual.

4.7 NEW STAGE STATION DESIGN

A new gauging station is usually established for the purpose of obtaining a record of streamflow, reservoir contents, tidal fluctuation or other purpose. Acquisition of stage data may be only one of several factors that must be considered. For purposes of this chapter of this Manual, however, only the requirements for obtaining an accurate and representative stage record are described. The following sections pertain primarily to sites where an automatic recording gauge will be used. Table I.4.2 summarizes some of the equipment requirements.

4.7.1 Site selection

Details for choosing an acceptable gauge site are described in this and other chapters of this Manual. The following criteria summarize the requirements for choosing a site where accurate stage records can be obtained:

(a) If the stage record is to be used for computing streamflow, the requirements for controls, rating curves, backwater and other streamflow variables must be considered in selecting the site as well as the acquisition of stage data. See chapters 2 and 3 of this Manual and Rantz, 1982;
(b) The site should be selected so the intakes or orifice are in a pool if possible, where stream velocity is low and not subject to significant turbulence. If this is not possible the intakes should be located in a slack-water zone where they are protected from high velocity;

Table I.4.2. Equipment requirements for stage-recording gauge site

Sensor	*Recorder*	*Stilling-well required*	*DC power required*	*Meets USGS stage accuracy requirements*
Basic float	Graphic or ADR	Yes	Graphic (No) ADR (Yes)	Graphic (Yes) ADR (Yes)
Float with shaft encoder	EDL and/or DCP	Yes	Yes	Yes
Float with potentiometer	EDL and/or DCP	Yes	Yes	No
Bubble gage with non-submersible pressure transducer	EDL and/or DCP	No	Yes	Yes
Submersible pressure transducer	EDL and/or DCP	No	Yes	No*
Non-contact sensor	EDL and/or DCP	No	Yes	No*

* Does not meet USGS stage accuracy requirements at the time of this writing.

(c) The gauge stilling well and the instrument shelter may be located on a stream bank, bridge, dam or other suitable structure provided the other site selection criteria are met as closely as possible. The gauge structure should be located to avoid damage during floods;

(d) If the gauge is located at or near a bridge, it is recommended that it be located on the downstream side. If this is not possible then it should be located far enough upstream to be out of the zone of drawdown caused by the bridge during medium and high water;

(e) The site should be selected where either a stilling well with intakes can be easily installed or where an instrument shelter can be installed for housing a bubble gauge. If a bubble gauge is to be used the site must provide suitable conditions to install the necessary bubble tubing and orifice static tube. For bank installations the tubing is usually placed underground between the gauge shelter and the stream. For bridge installations the tubing may be attached to the bridge members and pier or piling. The orifice static tube must be firmly anchored in the stream, preferably in a zone of low velocity;

(f) The gauge intakes should be low enough to record the lowest expected stage. In cold climates they should be below the frost line and protected from freezing if possible;

(g) The instrument shelter should be above the 200 year flood level;

(h) The distance between the stream and the stilling well and/or instrument shelter should be minimized;

(i) The site should have a suitable location for one or more outside auxiliary gauges. These could be staff gauges, chain gauge, wire-weight gauge or tape-down reference point. The auxiliary gauges should be easily accessible and located in a position so that accurate gauge readings can be made easily. They should be in the same pool as the gauge intakes and should provide readings that are indicative of the readings obtained through the intakes;

(j) If the gauge site is for the purpose of measuring stage in a lake or reservoir and it is near the outlet structure, the gauge intakes should be located upstream of the zone of drawdown of the outlet structure;

(k) Conditions at the site should be such that an accurate datum can be maintained. Appropriate reference marks and reference points should be located both on and off the gauging structure to maintain accurate and timely level surveys of the gauge.

4.7.2 **Sensor selection**

There are five basic categories of stage sensors as described in previous sections of this chapter. They are (a) float-driven sensors, (b) bubble gauge with non-submersible pressure transducer,

(c) submersible pressure transducers, (d) upward looking ultrasonic/submersible acoustic sensors and (e) non-contact water-level sensors. All of these stage sensing devices can be configured with other instruments to provide stage data that, in most cases, meet the accuracy requirements for a primary stage record. The third, fourth, and fifth sensors listed above may be somewhat less accurate than the first two methods, but can be adapted to provide stage data that are acceptable for primary stage records in many situations.

In some situations it may be impossible to maintain a stilling well or bubble gauge orifice in the stream because of very high velocities, or debris that can damage or wash out the gauge. The non-contact sensor may be the best alternative for such a site, even though it does not provide the accuracy of a float gauge or a bubble gauge.

Float-driven sensors require a stilling well whereas the other sensor types do not. All, however, require some type of instrument shelter. See previous sections of this chapter on "Water level sensors" for detailed descriptions.

4.7.3 Recorder selection

Four types of recorders are currently available for recording stage data, as described in section 4.4.3 – Water-level recorders. These include analog paper chart, analog digital punch tape, electronic data loggers and data collection platforms. Selection of a recorder is dependent to some degree on the type of sensor used and the other related instruments as shown in Table I.4.2. For instance, an Electronic Data Logger (EDL) can be used with any of the stage sensors, with the exception of a *basic* float system. A graphic recorder or a digital punch tape recorder can only be used with a *basic* float system.

4.7.4 Power requirements

Almost all of the currently used instruments require 12-volt dc electrical power as described in previous sections of this Manual. This may be supplied by dry or wet-cell batteries, or ac power may be converted to dc power. Battery chargers or solar panels may be used to keep rechargeable batteries fully charged. If solar panels and batteries are used, the power requirements of all sensors and recorders should be computed and the panels and batteries sized accordingly. Solar panel manufacturers can offer formulas to compute panel sizes based on equipment power draws and the estimated available sunshine for a geographic area.

4.8 OPERATION OF STAGE MEASUREMENT STATION

There are numerous and varied types of stage measuring equipment as well as different configurations of this equipment, as described in previous sections of this chapter. Today's equipment options are considerably greater than in previous years when only a graphic or digital punch-tape recorder was used. It is not practical to provide detailed instructions for servicing every type of recorder, data logger, DCP, gas-purge system and other stage-measuring device. The operation of each stage station will vary according to the type of equipment used and the hydrographer must become familiar with each device that he or she services. In addition each hydrographer will generally have a specific routine that he or she follows and which may be different than that of another hydrographer. Therefore, the following operational steps do not necessarily apply to all sites, nor are they in a specific order that cannot be varied. Operation of a stage measurement station should, however, be based on the following steps as closely as possible.

4.8.1 Clock, timer and battery check

One of the first things that the hydrographer should do when arriving at a gauging station is to check the recorder clock or timer. For paper strip-chart recorders this usually involves a check of the pen and chart to see if the recorder is running slow or fast, or if the clock may have stopped. The point at which the pen is resting on the chart is marked and the date and watch time are recorded on the chart. Adjustments should be made to the clock if there is a significant time error. In some cases it may be necessary to replace the clock.

Digital punch-tape recorders should be manually caused to punch a set of holes and the holes marked to indicate the end of the tape. The date and watch time should be recorded on a tape leader attached to the end of the tape. The clock should be adjusted if any significant time error is noted, or if necessary, the clock should be replaced.

Electronic Data Loggers (EDLs) and Data Collection Platforms (DCPs) have internal electronic clocks that are usually very accurate. Even so these should be compared to watch time and reset as necessary. A portable electronic clock and antennae, as described in a previous section of this chapter, can be used to set the time very accurately on an EDL or DCP. Future instrumentation will have internal time checks and automatic time adjustments as described in previous sections of this chapter.

Most gauging stations use 12-volt batteries for clocks, timers, EDLs and DCPs. Battery voltage should be checked each time the station is visited and the battery replaced if voltage is below the specified limit. The battery should also be replaced if it is at the end of its life cycle. If an ac electrical source or a solar charging panel is used these should be inspected to be sure they are operating correctly. Solar panels should be cleaned periodically to insure optimal performance.

4.8.2 Gauge readings

All auxiliary gauges and the reference gauge should be read before the gauge height record is retrieved from the recorder. These readings, along with the time of making the readings should be recorded on paper strip-charts and/or on digital punch tape leaders if either of these types of recorders is used. The gauge readings and times should also be recorded on the gauge inspection form. Electronic Data Loggers (EDLs) and Data Collection Platforms (DCPs) have no way to input auxiliary gauge readings, therefore the readings recorded on the gauge inspection forms are used for comparison and verification that the EDL and/or DCP are operating correctly. If the EDL or DCP stage is reset the time and magnitude of the reset should be noted on the gauge inspection form.

The reference gauge reading should be compared to the recorder reading at the time of the reference gauge reading to determine if any discrepancy exists. If a discrepancy is found then the cause of that discrepancy should be determined and corrected if possible.

4.8.3 Record retrieval

The gauge record should be retrieved from the recorder(s) after time and gauge readings have been determined and recorded. Paper charts and digital paper tapes are removed from the recorders and inspected for malfunctions. Electronically recorded data are retrieved from EDLs and/or DCPs by removing the data card and downloading the data from the card to a portable computer. In some cases it may be possible to make a direct download of the data into a portable computer. The electronically recorded data should be inspected for malfunctions and unusual events such as floods through the use of data files and/or graphics on the portable computer.

4.8.4 Float sensor, gauge well and intake inspection

Float sensor gauges require that the gauge well and intakes be free and clear of sediment and other obstructions that would impair their performance. Following the initial readings of time and gauge readings and the removal of the gauge record, the float, gauge well and intakes should be carefully inspected to be sure everything is in good working order. The inspection of the gauge record, as mentioned in the preceding paragraphs, may reveal sediment and/or intake problems that cannot be detected otherwise. The bottom of the gauge well should be probed for sediment deposits and cleaned if necessary. Likewise if there is any indication of intake problems the intakes should be flushed. The outside end of the intakes should be inspected, if possible, and sediment or other debris cleaned to allow free inflow to the intakes. In some cases a problem may develop from the float itself. The float should be inspected for leaks, slippage of the float-tape-clamp screw, or debris that may be lodged on top of the float.

Inspection of the gauge record may indicate a problem at an unusually low or high stage. When this occurs the hydrographer should try to find the cause and make appropriate corrections. For instance, it may be that the lowest intake is not low enough for the lowest stage experienced. In this case, a lower intake may need to be installed. Or it may be that the float hangs on an obstruction in the gauge well at a medium or high stage. The obstruction should be moved, or removed, so the float can freely pass. All problems and corrections should be documented.

4.8.5 Bubble gauge, gas system and orifice inspection

If the station has a bubble-gauge sensor the bubble orifice should be inspected to make sure it hasn't been buried by sediment. A log of gas-feed rate, gas consumption and gas-cylinder replacement should be kept to insure a continuous supply of gas and to help in checking for leakage in the system. There can be no serious leak in the gas-purge system if (a) the manometer operates to indicate stage correctly and (b) the gas consumption based on the average bubble rate over a period of time corresponds with the gas consumption computed from the decrease in cylinder pressure. If a leak is evident its location can be determined by isolating various parts of the gas-purge system by the sequential closing of valves in the system. Gas leaks should be fixed to ensure continuous operation of the bubble gauge. For newly established bubble gauges in cold areas it is common for leaks to occur during the first extremely cold period of the winter. This is because fittings installed in warm weather can contract in the cold. The hydrographer would be prudent to carry a liquid gas leak detector; this product is applied to fittings and will produce foam or small bubbles at a leak site.

4.8.6 Maximum and minimum stage determinations

If a high discharge has occurred since the previous visit to a stilling-well station, high-water marks should be sought both in the stilling well and outside the well. After obtaining high-water marks inside a stilling well the marks should be cleaned to prevent confusion with high-water marks that will be left by subsequent peak stages. In the case of a bubble-gauge station high-water marks should be sought along the stream bank in the vicinity of the gas orifice. High water marks along the bank should be marked if possible, for example if on a tree with a nail, and their location documented so that levels can be run to the mark at a future date. A crest-stage gauge as described in a previous section of this report should be used as the source of high-water marks at gauges that do not have a stilling well. When a low peak follows a high peak it may be possible to find high-water marks for both peaks. Regardless of the source high-water marks are used as an independent check on the maximum stage shown by the stage recorder and determination of these marks should be considered a very important part of the gauging station operation. Float tape maximum and minimum indicators, as described in a previous section, are another method for determining maximum and minimum stages.

Crest-stage gauges should be serviced on a regular basis to insure there is always fresh cork in the cup and to be sure the intake holes and vent hole are open. Cork lines defining high-water marks should be marked and dated, then the cork line erased after their elevations are determined to avoid confusion with high-water marks left at a later time. The staff should never be removed when water stands high in the pipe because re-inserting the staff will result in a surge of the water in the pipe thereby resulting in an artificial “high-water mark” on the staff.

4.8.7 Final recheck

After the hydrographer has done other work at the gauging station, such as making a discharge measurement, he or she should return to the gauge and make a final check of the timer, recorder and other instruments. Another complete set of gauge readings should be made and all readings including the watch time should be recorded on paper charts, digital-punch tape leaders and inspection sheets, as appropriate. The hydrographer should not leave the station without assuring he, or she, that all instruments are properly functioning.

4.8.8 General considerations

A few generalities may be stated concerning the maintenance of recording stream-gauging stations to increase the accuracy and improve the continuity of the stage record:
(a) Mechanical recorders and equipment should be periodically cleaned and oiled;
(b) Electronic equipment should be kept clean and properly ventilated;
(c) Intakes, stilling wells and sediment traps should be thoroughly cleaned, at least once a year, but more often when sediment is a recurring problem;
(d) Gas-purge systems of bubble gauges should be carefully checked for leaks during each visit;
(e) Excessive humidity and temperatures in the gauge shelter should be reduced to a minimum by providing proper ventilation, and in some cases, extreme cold temperatures should be modified by the use of heating units or insulation.

Experience has shown that a program of careful inspection and maintenance will result in a complete stage record about 98 per cent of the time.

4.9 SAFETY

Safety should be a primary consideration when working at gauging stations. Many times hydrographers work alone at a gauging station in a remote location and should therefore always consider the possible hazards that may be involved in doing the work and the fact that help may not be available in case of an accident. Even when working with other people, one’s personal safety and the safety of others should be of paramount importance.

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CHAPTER 5

MEASUREMENT OF DISCHARGE BY CONVENTIONAL CURRENT METER METHODS

5.1 INTRODUCTION

Streamflow, or discharge, is defined as the volumetric rate of flow of water in an open channel, including any sediment or other solids that may be dissolved or mixed with it. Streamflow is usually expressed in dimensions of cubic metres per second (m^3/s). Streamflow cannot be measured directly, but must be computed from variables that can be measured directly, such as stream width, stream depth and flow velocity. Even though streamflow is computed from measurements of other variables, the term streamflow measurement is generally applied to the final result of the calculations.

Discharge measurements are made at each gauging station to determine the discharge rating for the site. The discharge rating may be a simple relation between stage and discharge, or a more complex relation in which discharge is a function of stage, slope and rate of change of stage or other factors. Initially the discharge measurements are made at various stages at the station to define the discharge rating. Measurements are then made at periodic intervals, usually monthly, and also during extreme events such as floods or droughts to verify the rating, extend the rating, or to define any changes in the rating caused by changes in stream-channel conditions.

Discharge measurements are made by the several methods discussed in Chapters 5 to 9. However, the basic instrument most commonly used in making the measurement is the current meter. The observations of water velocity and depth are usually made by a hydrographer while stationary at each of a number of observation points in the cross section of a stream. This is referred to as the conventional (stationary) method of making a discharge measurement and is described in this chapter. In contrast to this conventional method is the moving boat method, the Acoustic Doppler Current Profiler (ADCP) method and the electromagnetic method, which are described in Chapter 6; Measuring structures, such as weirs and flumes, are described in Chapter 7; miscellaneous methods in Chapter 8 and indirect methods in Chapter 9.

5.2 GENERAL DESCRIPTION OF A CONVENTIONAL CURRENT METER MEASUREMENT OF DISCHARGE

A current meter measurement derives its name from the fact that it uses some type of current meter to measure stream velocity. It is made by sub-dividing a stream cross section into segments (sometimes referred to as partial areas or panels) and measuring the depth and velocity in a vertical within each segment. The total discharge for a current meter measurement is the summation of the products of the partial areas of the stream cross section and their respective average velocities. This computation is expressed by the equation:

$$Q = \sum_{i=1}^{n} a_i v_i \qquad (5.1)$$

where Q = total discharge, in cubic metres per second (m^3/s), a_i = cross-section area, in square metres (m^2), for the ith segment of the n segments into which the cross section is divided, and v_i = the corresponding mean velocity, in metres per second (m/s) of the flow normal to the ith segment, or vertical.

Two basic computation methods are used by most countries for computing the discharge for current meter measurements, the *midsection* method and the *mean-section* method. Other methods, such as graphical methods and the horizontal plane method are not described here, but can be found in ISO 748 (2007).

In the *midsection* method of computing a current meter measurement it is assumed that the mean velocity in each vertical represents the mean velocity in a partial rectangular area. The mean velocity in each vertical is determined by measuring the velocity at selected points in that vertical, as described in a later section of this Manual. The cross-section area for a segment extends laterally from half the distance from the preceding vertical to half the distance to the next vertical, and vertically, from the water surface to the sounded depth as shown in Figure I.5.1.

The cross section in Figure I.5.1 is defined by depths at locations 1, 2, 3, 4. . . . *n*. At each location the velocities are sampled by current meter to obtain the mean of the vertical distribution of velocity. The partial discharge is now computed for any partial section (segment) at location *i* as:

$$q_i = v_i\left[\frac{\left(b_i - b_{(i-1)}\right)}{2} + \frac{\left(b_{(i+1)} - b_i\right)}{2}\right]d_i$$
$$= v_i\left[\frac{b_{(i+1)} - b_{(i-1)}}{2}\right]d_i \quad (5.2)$$

where q_i = discharge through partial section *i*, v_i = mean velocity at location *i*, b_i = distance from initial point to location *i*, $b_{(i-1)}$ = distance from initial point to preceding location, $b_{(i+1)}$ = distance from initial point to next location, d_i = depth of water at location *i*.

Thus, for example, the discharge through partial section 4 (heavily outlined in Figure I.5.1) is:

$$q_4 = v_4\left[\frac{b_5 - b_3}{2}\right]d_4 \quad (5.3)$$

The procedure is similar when *i* is at an end section. The "preceding location" at the beginning of the cross section is considered coincident with location 1; the "next location" at the end of the cross section is considered coincident with location *n*. Thus,

$$q_1 = v_1\left[\frac{b_2 - b_1}{2}\right]d_1 \quad (5.4)$$

and

$$q_n = v_n\left[\frac{b_n - b_{(n-1)}}{2}\right]d_n \quad (5.5)$$

For the example shown in Figure I.5.1, q_1, is zero because the depth at observation point 1 is zero. However, when the cross-section boundary is a vertical line at the edge of the water as at location *n*, the depth is not zero and velocity at the end section may or may not be zero. Equations 5.4 and 5.5 are used whenever there is water only on one side of an observation point such as at the edge of the stream, piers, abutments and islands. It usually is necessary to estimate the velocity at an end section because it normally is impossible to measure the velocity accurately with the current meter close to a boundary. There also is the possibility of damage to the equipment if the flow is turbulent. The estimated velocity is usually made as a percentage of the adjacent section.

The summation of the discharges for all the partial sections is the total discharge of the stream. An example of the measurement notes is shown in Figure I.5.2.

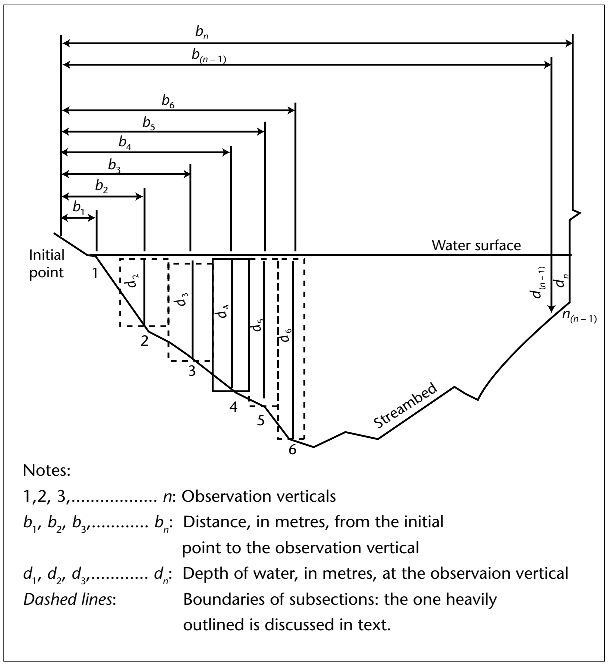


Figure I.5.1. Definition sketch of midsection method of computing cross-section area for discharge measurements

A summary of the discharge measurement, including gauge readings before, during, and after the discharge measurement, is prepared as a part of the discharge measurement. This summary is sometimes referred to as a front sheet. A summary sheet containing only a basic summary of the discharge measurement is shown in Figure I.5.3. Summary sheets can be enhanced to show additional information about the gauge, the discharge measurement, and other measurements made during the course of the visit to the gauging station.

The *mean-section* method was used by the United States Geological Survey (USGS) prior to 1950 and is still used in a few countries. It differs from the *midsection* method in computation procedure. Partial discharges are computed for partial sections between successive verticals. The velocities and depths at successive verticals are each averaged, and each partial section extends laterally from one vertical to the next. Discharge is the product of the average of two mean velocities, the average of two depths, and the distance between verticals. This is repeated for each partial section. The additional discharge in the partial sections adjacent to each bank is estimated on the assumption that the velocity and depth at the banks are zero. If, however, this discharge is a significant part of the total flow then the mean velocity in the vicinity of the bank should be estimated, or measured if

	Distance from Initial Point (m)	Depth (m)	Observation depth	Revs	Time in Seconds	Velocity: At Point (m/s)	Velocity: Mean in Vertical (m/s)	Area (m^2)	Width (m)	Discharge (m^3/s)
REW	4	0		0	0	0	0	0	0	0
	5	0.31	.6	40	60	0.193	0.193	0.31	1	0.060
	6	0.40	.6	45	59	0.219	0.219	0.40	1	0.089
	7	0.51	.6	51	61	0.238	0.238	0.51	1	[illegible]
	8	0.85	.6	52	61	0.243	0.243	0.85	1	0.206
	9	1.23	.2	55	60	0.260	0.235	1.23	1	0.289
			.8	44	60	0.211				
	10	1.58	.2	58	62	0.265	0.240	1.58	1	0.379
			.8	46	61	0.216				
	11	1.69	.2	60	61	0.278	0.251	1.69	1	0.424
			.8	48	61	0.225				
	12	1.71	.2	65	62	0.296	0.274	1.71	1	0.468
			.8	51	63	0.253				
	13	1.87	.2	70	62	0.317	0.287	1.87	1	0.537
			.8	58	64	0.257				
	14	1.84	.2	69	62	0.313	0.287	1.84	1	0.528
			.8	58	63	0.262				
	15	1.71	.2	66	61	0.305	0.278	1.71	1	0.475
			.8	55	62	0.252				
	16	1.65	.2	62	61	0.287	0.262	1.65	1	0.432
			.8	52	62	0.238				
	17	1.50	.2	65	61	0.298	0.258	1.50	1	0.387
			.8	50	60	0.218				
	18	1.36	.2	58	62	0.265	0.241	1.36	1	0.328
			.8	47	62	0.217				
	19	1.19	.2	55	61	0.257	0.228	1.19	1	0.271
			.8	42	63	0.193				
	20	1.17	.2	51	62	0.235	0.211	1.17	1	0.247
			.8	39	60	0.188				
	21	0.92	.6	46	61	0.216	0.216	0.92	1	0.199
	22	0.81	.6	41	63	0.188	0.188	0.81	1	0.152
	23	0.70	.6	39	[illegible]	0.184	0.184	0.70	1	0.129
	24	0.63	.6	36	63	0.167	0.167	0.63	1	0.105
	25	0.55	.6	31	61	0.150	0.150	0.55	1	0.082
	26	0.48	.6	26	64	0.124	0.124	0.36	0.75	0.044
LEW	26.5	0.40		21	67	0.030	0.030	0.10	0.25	0.003
										5.955

Figure I.5.2. Computation notes of a current-meter measurement by the midsection method

possible. The total discharge is obtained by summing the discharges from of all of the partial sections, including the end sections near each bank. A study by Young (1950) concluded that the *midsection* method is simpler to compute and is a slightly more accurate procedure than the *mean-section* method. This is also the conclusion stated in ISO 748.

5.3 INSTRUMENTS AND EQUIPMENT

Current-meter measurements usually are classified in terms of the method used to cross the stream during the measurement, such as wading, cableway, bridge, boat or ice. Instruments and equipment used in making current-meter measurements will vary depending on which of these measurement types are being used. Current meters, timers, and counting equipment are generally common to all types of current-meter measurements. This section describes all of the equipment used in making conventional current meter measurements, including current meters, timers, electronic field

DISCHARGE MEASUREMENT NOTES

STATION........................ METRE NO ....

CHECKED BY ....

DATE .... DATE........................

WIDTH........................ AREA........................

VEL........................ DISCHARGE ....

METHOD........................ CHANGE IN STAGE........................ IN................ HOURS..............

VEL COEFFT.................... HORZ ANGLE COEFFT .... METRE NO........................

SUSP .... SUSP COEFFT........................

METER RATING NO........................ DATE ....

WADING, CABLE, ICE, BOAT BRIDGE NUMBER OF VERTICALS........................

GAUGE READINGS			
Time	Outside	Recorder	Inside
Start			
Finish Weighted MGH			

CONDITIONS:

WEATHER .... FLOW........................

RECORDER .... OUTSIDE GAUGE ....

CONTROL........................ INTAKE ....

REMARKS:

Figure I.5.3. Discharge measurement summary sheet

notebooks, sounding equipment, width-measuring equipment, equipment assemblies and miscellaneous equipment.

5.3.1 Current meters, general

A current meter is a precision instrument calibrated to measure the velocity of flowing water. Several types of current meters are available for use, including rotating-element mechanical meters, electromagnetic meters, acoustic meters and optical meters. All of these meters, when properly maintained and calibrated, are considered suitable and accurate for measuring the velocity of streamflow.

The principle of operation for a mechanical meter, or rotating element current meter, is based on the proportionality between the velocity of the water and the resulting angular velocity of the meter rotor. By placing a mechanical current meter at a point in a stream and counting the number of revolutions of the rotor during a measured interval of time, the velocity of water at that point can be determined from the meter rating. The operational requirements, construction, calibration, and maintenance of rating element current meters is described in ISO 2537 (2007).

An electromagnetic current meter is based on the principle that a conductor (water) moving through a magnetic field will produce an electrical current directly proportional to the speed of movement (Faraday's law). By measuring this current and the resultant distortion in the magnetic field the instrument can be calibrated to determine point velocities of flowing water.

The acoustic meter uses the Doppler principle to determine point velocities of flowing water as well as complete vertical velocity profiles. Acoustic Doppler velocimeters (ADVs) are a class of acoustic meter that measures a point velocity and can thus be used to make measurements with a wading rod. The Acoustic Doppler Current Profiler (ADCP) has been adapted for use with the moving boat method of measuring discharge, as described in a later section of this Manual.

The optical current meter uses a stroboscopic device which is calibrated to measure surface velocities of flowing water. This meter cannot be used to determine sub-surface velocities.

The following sections of this Manual describe the various types of current meters in more detail, give advantages and disadvantages of each, and provide guidance on care and maintenance.

5.3.2 Current meters, mechanical, vertical axis

The meter most commonly used by the USGS is the vertical axis, mechanical current meter. The original prototype for this kind of current meter was designed and built in 1882 by W.G. Price while working with the Mississippi River Commission. The Price current meter has evolved through a number of different models and refinements since 1882, but the basic theory and concepts remain the same. The Price AA meter is currently used for most discharge measurements made by the USGS, however there are other variations of this meter: such as the Price AA slow velocity, the Price pygmy, and the Price AA winter. The following sections of this Manual describe the various Price meters in more detail, and Table I.5.1 summarizes the various configurations and recommendations for the Price current meter.

Price AA

The basic components of the Price AA meter include the shaft and rotor (bucket wheel) assembly, the contact chamber, the yoke and the tailpiece. The rotor, or bucket wheel, is 5 inches in diameter and 2 inches high with six cone-shaped cups mounted on a stainless-steel shaft. A vertical pivot supports the vertical shaft of the rotor, hence the name vertical-axis current meter. The contact chamber houses the upper part of the shaft and provides a method of counting the number of revolutions the rotor makes. Contact chambers that can be used on the Price AA meter are described in a later section of this Manual. The yoke is the framework that holds the other components of the meter. A tailpiece is used for balance and keeps the meter pointing into the current. See Figure I.5.4 for a detailed drawing of the Price AA current meter. A photograph of the Price AA current and Price pygmy meters are shown in Figure I.5.5.

When placed in flowing water the rotor of the Price current meter turns at a speed proportional to the speed of the water. For practical purposes these current meters are considered non-directional because they register the maximum velocity of the water even though they may be placed at an angle to the direction of flow. Advantages of the vertical axis current meter are:

(a) They operate in lower velocities than do horizontal-axis meters;
(b) Bearings are well-protected from silt-laden water;
(c) The rotor is easily repairable in the field without adversely affecting the rating;

Table I.5.1. Price current meter configurations, usage's, and recommended ranges of depth and velocity

Meter	*Contact Chamber*	*Counting Method*	*Rating*	*Velocity Range, feet per second*[1]	*Depth Range, feet*	*Remarks*
Price AA	Standard, cat-whisker and penta gear	Headphones, CMD[2] or EFN[3]	Standard	0.2 to 12	1.5 or greater	The Price AA meter can be used as a low velocity meter if equipped with an optic contact chamber.
			Individual	0.1 to 12		
	Magnetic	CMD[2] or EFN[3]	Standard	0.2 to 12		
			Individual	0.1 to 12		
			Standard or Individual	0.1 to 12		
Price AA, low velocity	Optic Cat-whisker with double contact lobe on shaft. No penta gear	Headphones, CMD[2], or EFN[3]	Individual	0.1 to 12	1.5 or greater	This is the traditional Price AA low velocity meter. An individual rating is recommended, however a standard rating can be used if less accuracy is acceptable.
Price pygmy	Cat whisker	Headphones, CMD[2] or EFN[3]	Standard or Individual	0.2 to 4.0	0.3 – 1.5	
Price, winter WSC[4] yoke, polymer cups	Cat whisker	Headphones, CMD[2] or EFN[3]	Individual, with suspension device	0.1 to 12	1.5 or greater	This meter is recommended for conditions where slush ice is present.
	Magnetic or Optic	CMD[2] or EFN[3]				
Price, winter WSC[4] yoke, metal cups	Cat whisker	Headphones, CMD[2] or EFN[3]	Individual, with suspension device	0.1 to 12	1.5 or greater	This meter is recommended for conditions where slush ice is not present.
	Magnetic or Optic	CMD[2] or EFN[3]				

[1] Low and high velocity limits shown in the table are based on a small to moderate extrapolation of the lower and upper meter calibration limits. It is not recommended that the meters be used for velocities less than the lower limit. The velocity rating for the Price meter may allow additional extrapolation in the upper range to about 20 fps. The upper range of the Price pygmy meter rating may be extrapolated to about 5 fps. Standard errors within the meter calibration limits are less than +/– 5% in all cases. Standard errors in the extrapolated range of velocities are unknown, but are probably within +/– 5%.

[2] Current Meter Digitizer. Observe cautions for low velocities. See text.

[3] Electronic Field Notebook such as Aquacalc or DMX. Observe cautions for low velocities. See text.

[4] Water Survey of Canada

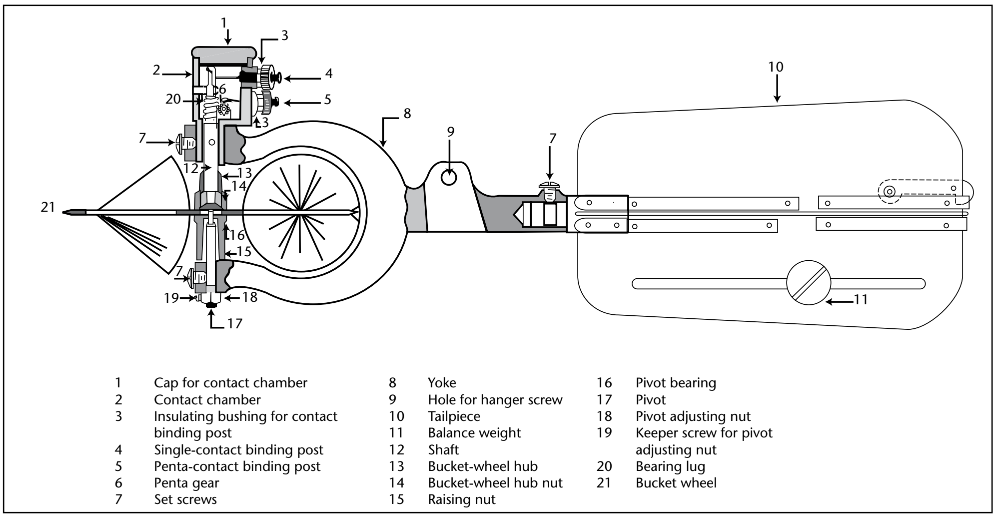


Figure I.5.4. Assembly drawing of the Price AA current meter

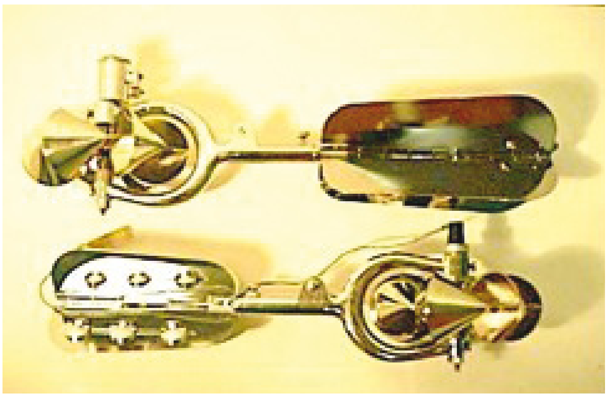


Figure I.5.5. Price AA (right) and Price pygmy (left) current meters

(d) Standard ratings apply to the Price AA and Price pygmy meters and
(e) A single rotor serves for the entire range of velocities.

Price AA, slow velocity

In addition to the Price AA meter described above, there is a Price AA meter modified slightly for use in measuring low velocities. In this meter the penta gear has been removed, which reduces friction. Also, the shaft has two eccentrics making two contacts with the cat's whisker per revolution. The low-velocity meter normally is rated from 0.06 to 0.75 m/s and is recommended when the mean velocity at a cross section is less than 0.30 m/s.

Price pygmy

A miniature version of the Price AA meter is the Price pygmy meter, also shown in Figure I.5.5, which is used for measuring velocities in shallow depths. The Price pygmy meter is scaled two-fifths as large as the standard meter and has neither a tailpiece nor a pentagear. The contact chamber is an integral part of the yoke of the meter. The Price pygmy meter makes one contact for each revolution and is used only with rod suspension.

Price AA winter

In streams where slush ice is present, a modified Price AA meter is recommended, as shown in Figure I.5.6. This meter is built with a Water Survey of Canada (WSC) winter-style yoke, and uses a polymer bucket wheel in place of the standard metal-cup rotor. The solid polymer rotor has the advantage that it does not fill with slush ice during a measurement, and the slush ice does not easily adhere to it. If slush ice is not present it is recommended that measurements be made with metal cup rotors in place of the polymer rotors. Regular Price AA meters with metal-cup rotors are also acceptable for slush-free conditions if cutting the required larger holes through the ice is not a problem.

5.3.3 Current meters, mechanical, horizontal axis

A number of mechanical current meters are available that have a propeller, or vane, type of rotor mounted on a horizontal shaft. These meters are used extensively in Europe and some eastern countries, but very little in the United States Horizontal axis current meters include the Ott (Germany), Neyrpic (France), Haskell (United States of America), Hoff (United States of America), Braystoke (United Kingdom of Great Britain and Northern Ireland), and Valeport (basically the same as the Braystoke). Various models of each of these are also available. As a group, horizontal axis current meters have the following advantages:
(a) The rotor, or propeller, disturbs flow less than do vertical-axis rotors because of axial symmetry with flow direction;
(b) The rotor is less likely to be entangled by debris than are vertical-axis rotors;
(c) Bearing friction is less than for vertical-axis rotors because bending moments on the rotor are eliminated;

Figure I.5.6. Price AA meter with winter-style yoke and polymer rotor

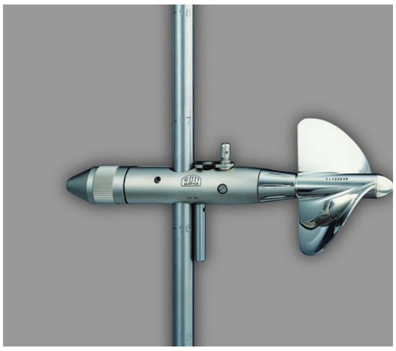


Figure I.5.7. Ott current meter

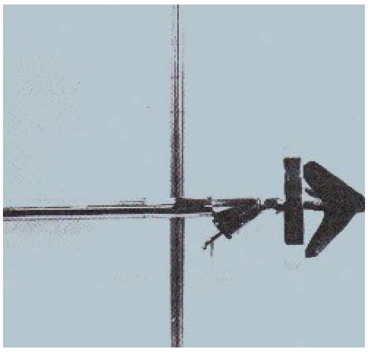


Figure I.5.8. Hoff current meter

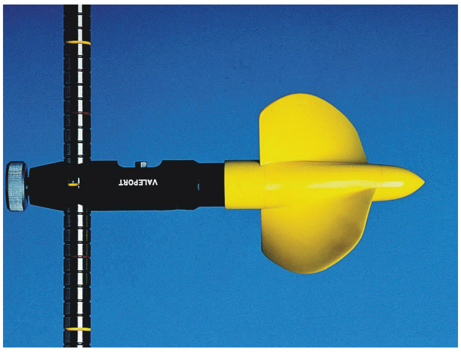


Figure I.5.9. Valeport current meter

(d) In oblique currents, some of these meters (such as the Ott meter) measure the velocity normal to the measuring section when the meter is held normal to the measuring section;
(e) Rotors with propellers of different pitches are available for some of the meters, allowing measurement of a considerable range of velocity.

See Figures I.5.7, I.5.8 and I.5.9 for examples of the Ott, Hoff, and Valeport current meters.

The makers of the Ott meter have developed a component propeller which in oblique currents automatically registers the velocity projection at right angles to the measuring section for angles as much as 45° and velocities as much as 3 m/s. For example, if this component propeller were held in the position AB in Figure I.5.10 it would register V max cos α rather than V max, which the Price meter would register. The Neyrpic and Ott meters are rarely used in the United States.

The Haskell meter is very suitable for use in streams that are deep, swift and clear. By using propellers with a variety of screw pitches, a considerable range of velocity can be measured. The Haskell meter is more durable than most other horizontal axis current meters.

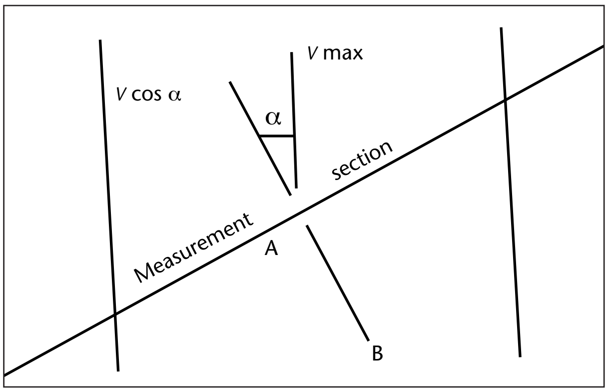


Figure I.5.10. Velocity components measured by Ott (V cos α) and Price current meters (V max)

The Hoff meter has a lightweight propeller with either three or four vanes of hard rubber. The meter is suited to the measurement of low velocities, but is not suitable for rugged use.

5.3.4 Comparison of performance of vertical axis and horizontal axis current meters

Comparative tests of the performance of vertical axis and horizontal axis current meters, under favourable measuring conditions, indicate virtually identical results from use of the two types of meter. This was the conclusion reached in 1958 by the United States. Lake Survey, Corps of Engineers, after tests made with the Price, Ott, and Neyrpic current meters. The results of one of their tests are shown in Figure I.5.11.

Between the years 1958 and 1960, the USGS made 19 simultaneous discharge measurements on the Mississippi River using Price and Ott meters. The average difference in discharge between results from the two meters was – 0.15 per cent, using the measurements made with the Price meter as the standard for comparison. The maximum differences in discharge measured by the two meters was – 2.76 and + 1.53 per cent.

5.3.5 Contact heads for Price, vertical axis current meter

The Price current meter is normally fitted with a contact chamber having a cat whisker type of circuitry used for counting the number of revolutions of the rotor. Two other types of contact chambers, the magnetic switch type and the optical type, can be fitted to the Price AA meter.

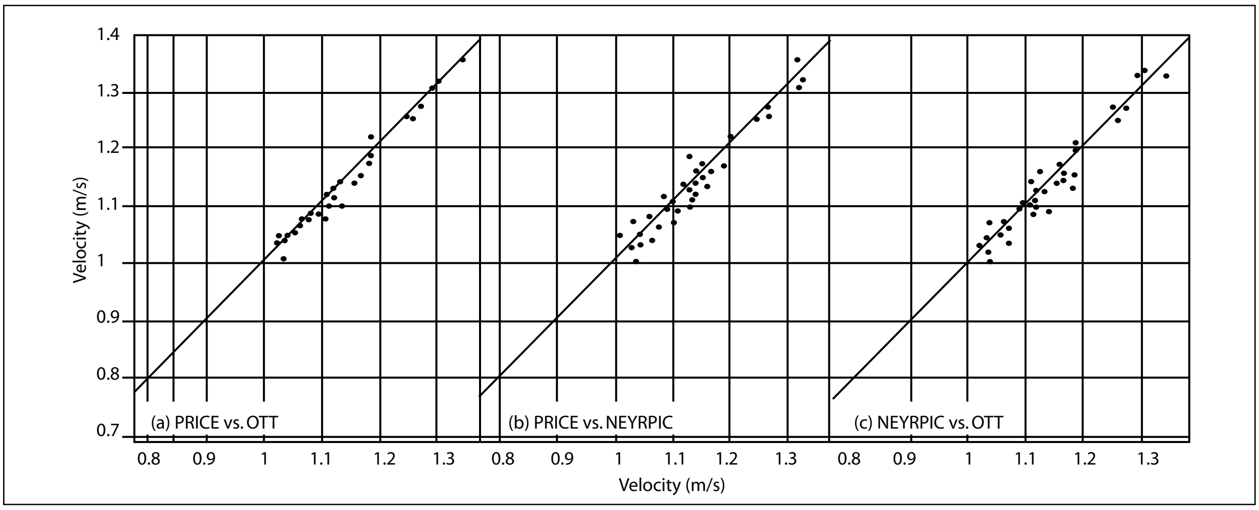


Figure I.5.11. Comparison of mean velocities measured simultaneously by various current meters during 2-minute periods, Stella Niagara section, panel point 5

Cat whisker

When placed in flowing water the rotor of the current meter turns at a speed proportional to the speed of the water. The number of revolutions of the rotor is obtained by counting electrical impulses generated in the contact chamber. An eccentric contact on the upper end of the rotor shaft wipes a slender bronze wire (cat's whisker) attached to the binding post which closes an electrical circuit. This electrical impulse produces an audible click in a headphone or registers a unit on a counting device. Contact points in the chamber are designed to complete the electrical circuit at selected frequencies of revolution, such as twice per revolution, once per revolution, or once per five revolutions. A separate reduction gear (penta gear), wire, and binding post provide a contact each time the rotor makes five revolutions. Figure I.5.12 shows the contact chamber and shaft for the cat whisker type chamber, with the dual binding posts.

Two types of cat-whisker wires have been used, one is the simple bronze wire, and the other is the old type wire with a small solder bead on the end of it. It is recommended that the beaded wire not be used, and that it be replaced with the simple bronze wire. Also, the cat whisker for the penta gear should always be adjusted to touch the penta eccentric, even when the penta counter is not in use. Otherwise the meter rating may be affected.

Magnetic switch

A contact chamber housing a magnetic type switch, as shown in Figure I.5.12, is available to replace the cat-whisker contact chamber. The magnetic switch is glass enclosed in a hydrogen atmosphere and hermetically sealed. The switch assembly is rigidly fixed in the top of the meter head just above the tip of the shaft. The switch is operated by a small permanent magnet rigidly fastened to the shaft. Two types of magnets are in use: (a) a bar magnet and (b) a circular magnet. If the contact chamber uses the bar magnet it should be identified with an "A" stamped on the top surface of the chamber to indicate it has been modified. Older, unmodified, contact chambers with the bar magnet were found to under-register for velocities greater than about 2 fps. The chambers that utilize the circular magnets fit the standard rating throughout its range.

Figure I.5.12. Contact chambers for cat whisker heads and magnetic heads

The magnetic switch quickly closes when the magnet is alined with it and then promptly opens when the magnet moves away. The magnet is properly balanced on the shaft. Any type AA meter can have a magnetic switch added by replacing the shaft and the contact chamber. The magnetic switch is placed in the contact chamber through the tapped hole for the binding post. The rating of the meter is not altered by the change.

An automatic counter as described in a later section of this Manual is used with the magnetic-switch contact chamber. A headphone should not be used with the magnetic head because arcing can weld the contacts.

Optical

A contact head utilizing fibre-optics technology is available for reading the pulse rate of the Price AA current meter. A special rotor, containing two fibre-optics bundles, is attached to the upper end of the bucket-wheel shaft. The rotation of these fibre-optics bundles gates infrared light from a photo-diode to a photo-transistor creating a pulse rate that is proportional to the rotor RPM. The pulses are counted, stored and then compared to a quartz crystal oscillator. This information is processed to display stream velocity on a liquid crystal readout. The display has three averaging periods selected by a rotary switch. The averaging periods range from a minimum of about 5 seconds to a maximum of about 90 seconds. The unit is powered by a 9-volt battery.

Output of pulses from the optical sensing unit can be counted by the current meter digitizer and the electronic field notebooks described in subsequent sections of this Manual. A standard current meter rating table is used to convert pulse rate to stream velocity.

A special tail-fin assembly is required for the optical meter so it will balance properly when submerged. The vertical section of this tail-fin is marked with the letters OAA, and the horizontal section is marked PAA.

5.3.6 Current meter timers and counters used with Price current meter

The determination of velocity using a mechanical current meter requires that the number of revolutions of the rotor be counted during a specified time interval, usually 40 to 70 seconds. Several methods are available for timing and counting the revolutions for the Price current meter as described in the following paragraphs.

Stopwatch and headset

For current-meters having a cat-whisker type contact chamber an electrical circuit is closed each time the contact wire touches the single or penta eccentric of the current meter. A battery and headphone, as shown in Figure I.5.13, are parts of the electrical circuit and an audible click can be heard in the headphone at each electrical closure. Some hydrographers have adapted compact, comfortable, hearing-aid type phones to replace headphones. Beepers that can be heard without the headset are also sometimes used. A headset, or similar device, should not be used with the magnetic contact chamber because arcing can weld the contacts.

The time interval is measured to the nearest second with a stopwatch. Figure I.5.13 shows the standard analog stopwatch that is frequently used, however, a digital wrist watch is also acceptable.

Current-Meter Digitizer

An automatic electronic counter, or Current-Meter Digitizer (CMD), as shown in Figure I.5.14, has been developed for use with the cat-whisker, optic and magnetic contact chambers. It can be used with any of the mechanical, vertical-axis, current meters, but care should be taken to avoid false counts when using it for low velocities when the cat-whisker contact chamber is used. The CMD automatically counts and displays the number of revolutions of the current-meter rotor and the elapsed time. A buzzer produces an audible signal at each contact closure and the total counts and



Figure I.5.13. Analog stopwatch, digital stopwatch, beeper, and current-meter headset

Figure I.5.14. Current-meter digitizer

elapsed time are held in the display at the completion of the velocity measurement. A coded chip can be installed in the CMD that will compute and display the velocity from the standard rating table for the particular meter being used. The CMD is powered with five rechargeable batteries and has an adapter which can be used to attach it to the top of a wading rod.

Other electronic counters

Electronic counters and timers for mechanical current meters are also available in some commercially available devices. Electronic Field Notebooks (EFN) are designed for electronic recording of discharge measurement data and contain built-in digitizers that count and time the current meter rotor revolutions. Built in ratings convert the revolutions and elapsed time to velocity. Just as with the CMD described above, the EFNs should be used with caution to avoid false counts when measuring low velocities with meters equipped with a cat-whisker contact chamber. Electronic field notebooks will be described in more detail in a later section of this Manual.

5.3.7 **Current meters, electromagnetic**

Electromagnetic current meters, with no moving parts, are commercially available for measuring point velocities. These meters are based on the principle that a conductor (in this case, water) moving through a magnetic field will produce an electric current. The velocity of the moving water can be related to the electric current produced and the distortion created in the magnetic field. The electromagnetic meters can be accurately calibrated in a tow tank, similar to the calibration of mechanical meters, however tests have shown that the electromagnetic meters are less accurate and have more variance than rotating element current meters, especially at low velocities (less than about 0.5 fps). A significant limitation of electromagnetic meters is that they are susceptible to electrical interference and require zero stability tests.

The design, selection and use of electromagnetic current meters are described in ISO 15768 (2000). This standard describes the use of EM current meters to determine point velocity for the purpose of measuring flow in an open channel using the velocity area method.

Advantages of the electromagnetic current meter are as follows:
(a) No moving parts;
(b) Direct read-out of velocity;
(c) In oblique flow the velocity measured is normal to the measuring section when the meter is held normal to the measuring section;
(d) Will measure lower velocities than rotating element current meters, even though uncertainties will be relatively high.

Marsh-McBirney 2000

An electromagnetic current meter successfully used by the USGS for making discharge measurements is the Model 2000, produced by Marsh-McBirney. This meter, as shown in Figure I.5.15 along side the Price pygmy meter for comparison, is designed to mount on a standard round or top-setting wading rod. The meter is not designed for cable suspension.

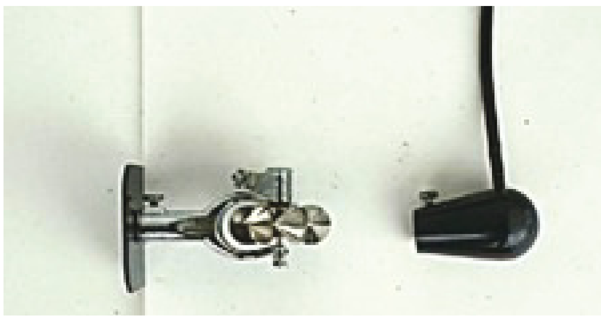


Figure I.5.15. Marsh-McBirney Model 2000 electromagnetic flow meter (right) and Price Pygmy current meter (left)

A display meter for the electromagnetic current meter shows a direct read-out of the velocity. No conversion equation or table is necessary. The meter must be kept clean for accurate readings and it is recommended that the rating be spot-checked occasionally to verify that it is still accurate. This can be done in two ways. First, submerge the meter in a bucket of still water to verify the zero point of the rating. Second, place the meter in close proximity to a Price AA current meter or other mechanical current meter, in flowing water, to verify that it gives the same velocity reading. If differences are found the electromagnetic meter should be re-rated in the tow tank.

Ott

An Ott electromagnetic current meter is available and is used extensively in Europe. The Ott Nautilus C 2000 meter, shown in Figure I.5.16, works in a similar manner to the Marsh-McBirney meter. The Nautilus C 2000 is designed for the measurement of very low flow velocities from zero up to 2.5 m/s. It can be used where rotating element current meters cannot be used such as in streams where water plants are present, and in marginal zones near stream banks and shallow water.

Valeport

The Valeport electromagnetic current meter has been used successfully in Europe for a number of years. This meter comes with a dedicated display unit that both operates the sensor and provides a display of the measured water velocity. The display will show real-time velocity data at a resolution of +/– 1mm/s, as well as the average velocity and standard deviation.

5.3.8 Current meters, Acoustic Doppler Velocimeter (ADV)

The apparent change of frequency of sound waves, which vary with the relative velocity of the source and the observer, is known as the Doppler Effect, or Doppler shift. Instrumentation has been developed in recent years to transmit acoustic signals into a water column where the frequency of backscatter signals reflected off particles in the water can be measured and used to calculate the velocity of the particles, and hence the velocity of the water. Currently there are two types of acoustic velocity meters, for measuring point velocities, and the Acoustic Doppler Current Profiler (ADCP) for measuring velocity profiles in a column of water. The ADCP will be discussed in detail in Chapter 6.

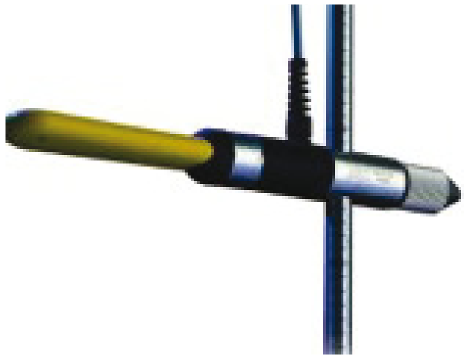


Figure I.5.16. Ott electromagnetic current meter

An Acoustic Doppler Velocity Meter, referred to as the ADV, has been developed commercially for measuring velocities of liquid flow within a few centimetres of the probe. This short measurement distance provides velocity data that can be considered point velocities for practical purposes. The advantages of an ADV meter over mechanical meters include:
(a) No moving parts, thus less maintenance required;
(b) Minimal flow disturbance;
(c) Velocity measurement as low as 0.0015 m/s;
(d) Minimum operating depth of 3 cm.

Note that although the ADV will operate within the stated minimum depth limit, solid boundaries can result in errors in the measured velocity. If the sampling volume of the ADV includes, or is near a solid boundary, the velocity data will be biased low. ADVs at the time of this writing are relatively new although they are being used for making discharge measurements in shallow streams. Much of the information in this Manual is based on research by the USGS.

ADVs have been used in many water resource applications, including laboratory hydraulic studies, mapping flow fields around structures, studying turbulence in water columns and ocean surf-zone studies. ADVs are manufactured by several vendors, including SonTek and Nortek Corporations, and have varying acoustic frequencies and configurations depending on the intended application. SonTek has developed an ADV with the trade name Flowtracker designed specifically for making wading discharge measurements. The Flowtracker is designed to make discharge measurements using the same general method as used for mechanical current meters as described in section 5.3. The Flowtracker is deployed using a top-setting wading rod (Figure I.5.24, described in section 5.3.12) of the same type used for conventional current meters. Flowtrackers can be used with the vertical velocity point sampling methods described in section 5.4. Flowtrackers have been tested by the USGS in the laboratory and in the field; laboratory tests were by tow tank and field tests typically by comparison with conventional meter measurements. The testing found that in general, Flowtracker algorithms for computing discharge are correct, velocity measurements met accuracy standards for conventional current meters and that field discharge measurements compared well with concurrent measurements made with conventional current meters (Morlock and Fisher, 2002). Because the Flowtracker is intended to make discharge measurements with methods described in this report and because use of this instrument appears to have become widespread, this section concentrates on the features and use of this particular ADV.

ADVs are high-precision, point-velocity current meters. They are "point-velocity" meters because they measure velocities within a very small volume. ADVs use a complex pulse-to-pulse coherent sampling scheme that gives the instruments high precision in measurement of stream velocity. An ADV consists of a probe head connected by a stem to cylindrical electronics housing. The probe head houses bi-static transducers ("bi-static" refers to the fact that the transducers either only transmit sound pulses or only receive sound pulses). The probe head consists of a transmitting transducer in the centre and two or three probes containing receiving transducers. The centre transducer transmits sound pulses which reflect back to the receiving probe transducers. Velocities are measured within a sampling volume located at a fixed distance from the transmitting transducer, as illustrated in Figure I.5.18. With three probes, an ADV, such as shown in Figure I.5.17, measures velocities in three dimensions.

Features of the 3-D probe head are as follows:

(a) The probe head would be submerged with the centre transmitting transducer pointed across the stream, perpendicular to the flow. The bottom probe with the red band would be positioned on the downstream side of the probe head;
(b) For a 3-D probe, the bottom two probe heads would measure two-dimensional velocity components (an across-stream or y-component and a downstream or x-component). The third vertical probe would measure the vertical or z-component of velocity (this component would not be necessary to measure discharge). The 2-D probe is adequate for regular discharge measurements;

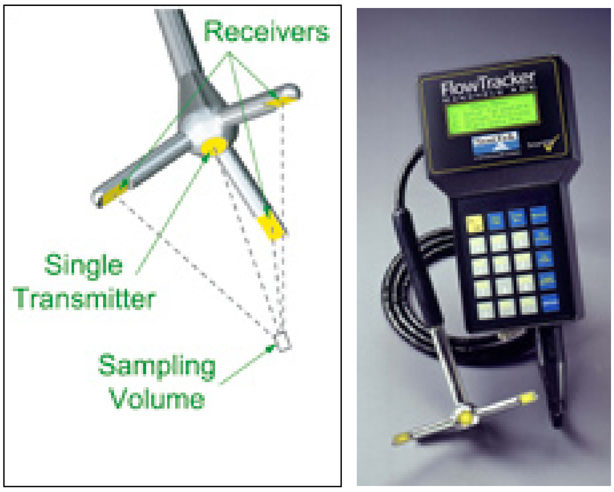


Figure I.5.17. Illustrations of 3-probe ADV probe head and electronic interface, courtesy of SonTek Corporation

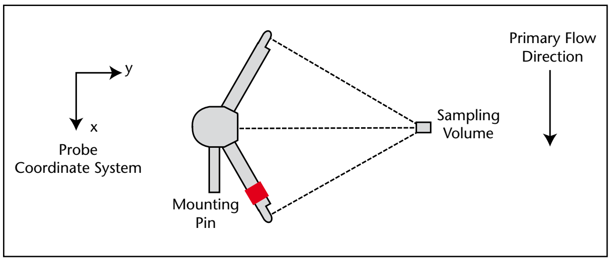


Figure I.5.18. Positioning of ADV unit relative to direction of flow

(c) The x-component of velocity would be used to compute discharge; so for a 3-D probe if only the bottom two probes were submerged, discharge could still be measured with the ADV;
(d) The probe head has a mounting peg that allows it to be attached quickly to a standard top-setting wading rod;
(e) The ADV electronics and power supply are integrated within an interface that includes a keypad for entering discharge measurement parameters and an LCD screen for displaying information;
(f) The interface allows entry of basic parameters, including station, distance, depth and vertical location of the measurement (0.6, 0.2 and 0.8 depth);
(g) The interface displays computed discharge at the end of a measurement. The interface also displays a variety of other data, such as average velocity standard deviation that can be used to evaluate the measurement quality.

It should be noted that early Flowtrackers had problems measuring velocities accurately in clear water. The manufacturer has since made hardware upgrades that dramatically improve the capability to work in low backscatter environments.

When the USGS reviewed a large number of Flowtracker discharge measurements during instrument testing many measurements showed numerous boundary effect problems. A boundary effect occurs when the sampling volume for the Flowtracker includes some solid, stationary boundary such as a cobble or boulder in the stream. These problems generally are more common in streams with rough beds and at shallow depths. The volume of water used by the Flowtracker to make velocity measurements is approximately 0.25 cubic centimetres in size and is located 10 centimetres away from the centre transmitting transducer on the Flowtracker. If the Flowtracker sample volume includes or is near a solid boundary, the velocity data will be corrupted and biased low. The low bias is caused by the Flowtracker measuring a zero velocity from the solid, stationary boundary for at least some part of the sample volume.

5.3.9 **Optical current meters**

An optical current meter, as shown in Figure I.5.19, is a stroboscopic device designed to measure surface velocities in open channels without immersing equipment in the stream. However, because it measures only surface velocity the optical meter is not considered a substitute for conventional equipment in those situations where good measurements can be made by standard techniques. It is a device that has extended the capability of making discharge measurements to a range of situations under which standard current meter techniques cannot be used. Those situations include flood velocities that are too high to be measured by conventional meter, for example, supercritical velocities in flood-ways, or the presence of a debris load during flood periods that makes it hazardous to immerse a current meter.

Figure I.5.19. Optical current meter

Basically, the meter is a stroboscopic device consisting of a low power telescope, an oscillating mirror driven by a variable speed battery operated motor and a tachometer. The water surface is viewed from above through the meter while gradually changing the speed of the motor to bring about synchronization of the angular velocity of the mirror and the surface velocity of the water. Synchronization is achieved when the motion of drift or disturbances on the water surface, as viewed through the meter, is stopped. A reading of the tachometer and height of the meter above the water surface are the only elements needed to compute the surface velocity. The meter is equipped with a tape and weight for measuring the distance to the water surface.

The velocity measurement may be made from any bridge, walkway, or other structure that will support the optical meter. The vertical axis of the meter must be perpendicular to the water surface. Surface velocity (V_s) is computed from the equation:

$$V_s = KRD \tag{5.6}$$

where K is the constant for the meter (K = 0.06415 for the USGS model); R is the readout for the tachometer in percentage of full scale, where 100 per cent is a rotational speed of the tachometer generator of 1,000 revolutions per minute; and D is the distance to the water surface, in feet.

The computed velocity must be corrected by an appropriate coefficient to represent the mean velocity in the vertical. The precise coefficient applicable is, of course, unique to the particular stream and to the location of the vertical in the stream cross-section. However, data abstracted from conventional current meter measurements show that application of a coefficient of 0.90 will not introduce errors of more than ± 5 per cent in concrete lined channels. For natural channels a coefficient of 0.85 has been used.

A unique feature of the optical current meter is the automatic correction that is made for variations in the direction of the streamlines of flow. If the flow approaches the cross section at an angle other than the perpendicular and if the axis of the oscillating mirror in the meter is parallel to the cross section, then at the null point of observation, the water will appear to move laterally across the field of view. The meter measures only the velocity vector normal to the cross section and there is no need to apply horizontal angle corrections.

The range of velocities that can be measured with the optical current meter is limited at the low end by the accuracy of the tachometer and at the upper end by the physical limitations of the human eye. Table I.5.2 shows the range of velocities that can be measured from various heights above the water surface with the USGS model of the meter. The minimum velocities shown in column 3 of the table can be measured with an error of ± 5 per cent; the higher velocities at the various observation heights will be measured with lesser error.

The rating of the optical current meter is relatively simple. Its operation is based on precise mathematical principles and given an accurate tachometer; the meter coefficient is dependent only on the configuration of the cam that oscillates the mirror. A master cam is used in the manufacture of the individual meter cams. The meter is rated by observation of a long endless belt driven at constant speed. That known belt speed is checked against the speed computed by multiplying the height of the meter above the belt by the tachometer reading. If the comparison of known and computed speeds shows a lack of agreement the meter coefficient is changed to bring about agreement.

5.3.10 Care of the current meter

To ensure reliable observations of velocity it is necessary that the current meter be kept in good condition. The care of conventional meters will be discussed first.

Table I.5.2. Range of velocities that can be measured with optical current meter

Observation height (D)	*Maximum velocity*	*Minimum velocity for ± 5 per cent resolution*
(m)	(m s^{-1})	(m s^{-1})
0.30	1.52	0.09
0.61	3.05	0.18
1.52	7.62	0.49
3.05	15.2	0.98
4.57	22.9	1.46
6.10	30.5	1.95

Conventional current meters

Before and after each discharge measurement the meter cups, pivot, bearing and shaft should be examined for damage, wear or faulty alignment. Before using the meter its balance on the cable suspension hanger should be checked, the alignment of the rotor when the meter is on the hanger or wading rod should also be checked, and the conductor wire should be adjusted to prevent interference with meter balance and rotor spin. During measurements the meter should periodically be observed when it is out of the water to be sure that the rotor spins freely.

Meters should be cleaned and oiled daily when in use. (The Braystoke meter is water lubricated and does not require oiling.) If measurements are made in sediment-laden water, the meter should be cleaned immediately after each measurement. For vertical axis meters the surfaces to be cleaned and oiled are the pivot bearing, penta gear teeth and shaft, cylindrical shaft bearing and thrust bearing at the cap.

After oiling, the rotor should be spun to make sure that it operates freely. If the rotor stops abruptly the cause of the trouble should be sought and corrected before using the meter. The duration of spin should be recorded on the field notes for the discharge measurement. A significant decrease in the duration of spin indicates that the bearings require attention. In vertical axis current meters the pivot requires replacement more often than other meter parts and it therefore should be examined after each measurement. The pivot and pivot bearing should be kept separated, except during measurements, by use of the raising nut provided in the Price meter or by replacing the pivot with a brass plug in the pygmy meter. Fractured, worn, or rough pivots should be replaced.

For horizontal axis current meters, during storage the propeller should be removed from the meter and the oil drained from the body. If bearings need to be cleaned, they should be flushed with alcohol or gasoline. Spare bearings have a protective grease coating which should be removed before they are used. Before the meter is used clean oil should be added to the body by holding it upright and half filling. As the axle bush is screwed back onto the carrier any excess oil will be forced up through the capillary gap around the axle and should be wiped away. Spinning the propeller for about a minute to check the condition of the bearings and to ensure proper oil distribution is advisable.

Meter repairs by the hydrologist should be limited to minor damage only. This is particularly true of the rotor, where small changes in shape can significantly affect the meter rating. In vertical axis meters minor dents in the cups can often be straightened to restore the original shape of the cups, but in case of doubt, the entire rotor should be replaced with a new one. Badly sprung yokes, bent yoke stems, misaligned bearings and tailpieces should be reconditioned in shops equipped with the specialized facilities needed.

A full timed spin test for the vertical axis current meter should be an office procedure made under controlled conditions between field trips. Timed spin tests should also be made when the performance of the meter is suspect and before and after repairs. The meter should be placed on a stable, level surface to perform the spin test. There should be no wind currents or drafts that would affect the rotor spin. The rotor is given a sharp, forceful spin by hand and a stopwatch is started simultaneously. The stopwatch is stopped when the rotor comes to a complete stop. The minimum, acceptable spin times are as follows:

All types of Price pygmy meters	45 seconds
All types of Price AA meters	2 minutes

These are considered absolute minimum spin times. Meters in good condition will perform substantially better.

A current-meter log should be maintained to record the results of the timed spin tests for each vertical-axis current meter. In addition, the log should contain information that identifies the meter and rotor, a history of repairs to the meter, as well as the name of the person who checked the meter, and dates of occurrences. Figure I.5.20 shows a recommended format for the current-meter log. The current-meter log should become a permanent record and it should be archived with other water resources data. Similar logs should be used for other standard mechanical current meters.

There are a number of documents describing the care and maintenance of the vertical-axis current meters. Among these, the most significant are by Smoot and Novak (1968) and by Rantz (1982).

5.3.11 **Rating of current meters**

To determine the velocity of the water from the revolutions of the rotor of a mechanical current meter a relation must be established between the angular velocity of the rotor and the velocity of the water turning it. This relation is referred to as the current-meter rating and is expressed in an equation or in tabular format. The following paragraphs describe the current meter rating procedures that are used primarily in the United Kingdom and in the United States Similar procedures are used in other parts of Europe. Standards for calibrating mechanical current meters are described in ISO 3455 (2007).

Current meter tow tanks

A current meter rating is established by first towing the meter at a constant velocity through a long water-filled trough, and then relating the linear and rotational velocities of the current meter. Rating troughs are very similar in different countries, ranging in length from about 50 to 200 metres. Widths range from about 2 to 5 metres, and depths range from about 1 to 5 metres. The Swiss National Hydrological Survey calibration laboratory in Bern, Switzerland, as shown in Figure I.5.21, is used by

CURRENT METER LOG				
Meter Type: AA Pygmy Other _______ (Circle one)		Meter No.		Rotor No.
Date	Meter User	Entry made by	Spin Time	Description of repairs, notation of disassembly, inspections, and remarks

Figure I.5.20. Suggested format for vertical-axis current meter log

two European countries. The equipment comprises basically a tank which is 140 m long, 4 m wide and 2.4 m deep and is filled with water and a remote-controlled carriage which tows a maximum of 4 instruments through the water at a controlled speed of up to 10 m/sec. It also includes an operating console, measuring instruments, equipment for transmitting and analysing the data obtained, and a security device.

Current meter rating tanks generally have a movable carriage mounted above the water surface that can run on rails or a track at a constant speed. The current meter is suspended from the carriage and moved through the water at a constant speed. Calibration runs are made at various speeds that can be accurately measured ranging from about 0.1 m/s to about 4 m/s. Data obtained for measured distances of the carriage run, elapsed time for each carriage run, and revolutions of the current meter propeller or cups during each carriage run can be used to develop current meter rating equations.

The current-meter rating facility operated by the USGS is located at their Hydrologic Instrumentation Facility. It consists of a sheltered reinforced concrete basin 122 m long, 1.83 m wide, and 1.83 m deep. An electrically driven car rides on rails extending the length of the basin. The car carries the current meter at a constant rate through the still water in the basin. Although the rate of travel can be accurately adjusted by means of a hydraulic regulating gear, the average velocity of the moving car is determined for each run by making an independent measurement of the distance it travels during the time that the revolutions of the rotor are

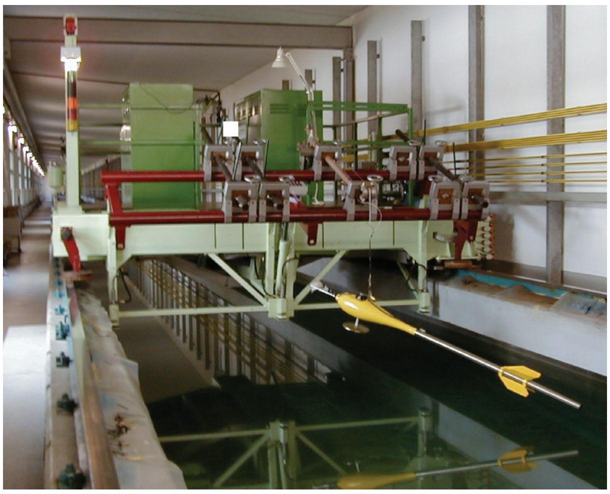


Figure I.5.21. Current meter rating tank located at the Swiss National Hydrological Survey calibration laboratory in Bern, Switzerland

electrically counted. A scale graduated in feet and tenths of a foot is used for this purpose. Eight pairs of runs are usually made for each current meter. A pair of runs consists of two traverses of the basin, one in each direction, at approximately the same speed.

Conditions for satisfactory calibration

To obtain a satisfactory calibration of a current meter several conditions must be satisfied:

(a) The counting of pulses and the measurements of time and distance must be accurate;
(b) The carriage must run smoothly and at constant speed so that oscillatory motion, whether longitudinal or lateral, is not transmitted to the meter. Timing of runs with cable-suspended meters must not be started until to and fro oscillations initiated during acceleration are damped out;
(c) The method of suspension of the meter should be that used during field measurements;
(d) The axis of the meter must be parallel to the water surface and to the long dimension of the tank;
(e) Residual motion of the water must be negligible;
(f) Measurements should not normally be made within a range of speeds where there is an *Epper* effect (see the UNESCO/WMO *International Glossary of Hydrology* (WMO-No. 385), second edition, 1992). The size of the effect and the range of speeds within which the effect is appreciable vary with the size of the meter and the dimensions of the tank. It is larger with larger meters and may be negligible with miniature meters. For a given meter the effect is larger in a small tank than it is in a large one.

Most of these requirements are obvious. Timing and counting of revolutions of the propeller or rotor present no problems, particularly when automatic pulse counters and timers are used. Vibration of the carriage may occur at particular speeds when rod suspended meters are calibrated at a particular depth of immersion. The *Epper* effect (see the UNESCO/WMO *International Glossary of Hydrology* (WMO-No. 385), second edition, 1992) occurs for a range of speeds having values very near to $\sqrt{dg}$. This is the speed of a shallow water wave in water of depth *d (celerity)*. When the carriage has this speed the disturbance caused by the immersed current meter and its suspension equipment moves along the tank with the meter and reduces the rate of revolution of the rotor. At speeds less than the critical velocity the disturbance may be reflected from the ends of the tank and overtake the meter the performance of which may be affected over a range of values. The size of the *Epper* effect may be little more than the uncertainty of a single calibration point. It is a systematic not a random error.

Probably the largest source of error is that due to residual movement of the water. Part of the difficulty may be caused by density currents and part by the disturbance arising from the previous run. Calibration tanks which are underground may be free from density currents which arise from changes in temperature but tanks which are above ground may experience fairly rapid changes of temperature which do not occur simultaneously at all parts of the tank. Residual water movements can be detected by means of floats but cannot be measured simultaneously with a calibration run. They should not be greater than one per cent of the speed of the calibration run.

Another practical point which is not listed under any of the numbered conditions for satisfactory calibration is concerned with acclimatization. When a meter having been lubricated is first immersed in the water there may be small pockets of air trapped in it and there may also be traces of oil which, during use of the meter, will be washed away. If there is a change of temperature there may also be some slight expansion or contraction. If measurements are started immediately after the meter is lowered into the water, the first calibration point is liable to be slightly off the line of the later results. To deal with this difficulty the meter may be run up and down the tank a few times and the water allowed to settle before measurements are started.

The preceding discussion relates primarily to mechanical current meters. The optical (surface velocity) meter and the acoustic type meters cannot be calibrated in the tow tank, and must be used with ratings developed by other means, or provide by the manufacturers. The electromagnetic meter, however, can be calibrated in the tow tank in a similar manner as the mechanical current meters.

Acoustic meter accuracy checks depend on the type of meter used. ADV meters can be tested in a tow tank provided the tow tank can be seeded with material that remains in suspension to provide signal echoes. Various methods for testing ADCPs have included tow tank tests, flume tests, and comparing ADCP discharges with discharges computed from other methods such as mechanical current meters. Oberg (2002) discusses the limitations of these ADCP accuracy tests.

Current meter rating equations and tables

The complete calibration of a propeller or rotor is expressed as a linear equation or series of linear equations of the form:

$$v = \alpha + \beta n \tag{5.7}$$

where v is the velocity, in metres per second, α and β are constants, and n is the rate of revolution of the propeller or rotor, in revolutions per second.

The rating for most current meters will not be linear throughout the range of velocities, and therefore two or more equations will be required to define the rating for the full range of velocities. It should also be noted that most mechanical current meters have a point generally referred to as the minimum speed of response. This is considered the minimum speed at which the propeller or rotor of a current meter attains continuous and uniform angular motion. The minimum speed of response is variable, depending upon the type of propeller or rotor, but is usually a speed less than about 0.03 m/s.

Because there is rigid control in the manufacture of most current meters, such as the Price and Braystroke current meters, virtually identical meters are produced and, for practical purposes, their rating equations are identical. Therefore there is no need to calibrate the meters individually, a significant advantage and time saver. Instead, a standard rating is established by calibrating a group of meters that have been constructed according to strict specifications. This standard rating is essentially an average rating for the calibration group and it is then supplied with all meters manufactured according to the specifications. Identicalness of meters is insured by supplying the dies and fixtures for the construction of Price current meters to the manufacturer who makes the successful bid. Another advantage of the standard rating is that field repairs can be made to a meter without requiring that it be re-calibrated. On the other hand, there are somewhat larger errors associated with the standard ratings, as opposed to the individual meter ratings.

Standard current-meter ratings are not mandatory for use with the Price meters. For some applications, it may be desired to obtain individually rated meters, and avoid the additional uncertainty of the standard ratings. All winter style meters must be individually rated with the suspension device that will be used with it.

Standard current-meter ratings, as of 1999, have been defined for the Price AA with the cat-whisker and magnetic contact chambers, and the Price pygmy with the cat-whisker contact chamber. The standard rating for the Price AA with the fiber-optic contact chamber was defined in 1991. These ratings are as follows:

Price AA with cat-whisker and magnetic contact chambers (Standard rating No. 2)

$$V = 2.2048R + 0.0178 \qquad (5.8)$$

Price pygmy with cat-whisker contact chamber (Standard rating No. 2)

$$V = 0.9604R + 0.031 \qquad (5.9)$$

Price AA with fiber-optic contact chamber

$$V = 2.194R + 0.014 \; (R < 0.856) \qquad (5.10)$$

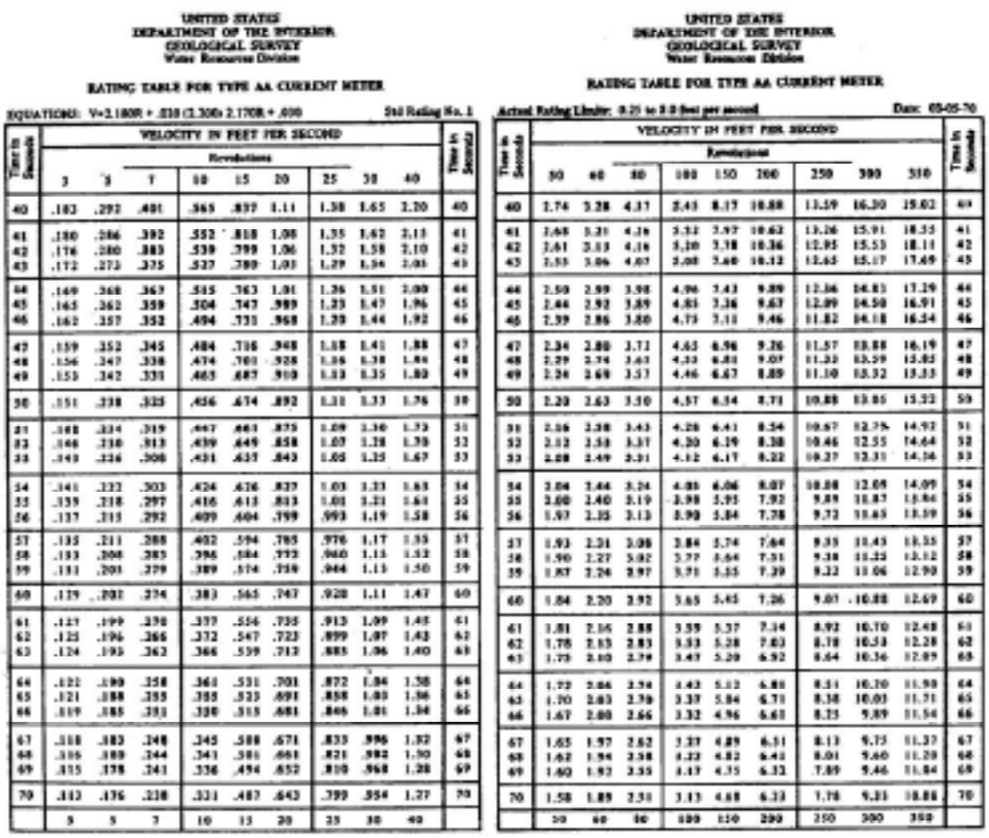
$$V = 2.162R + 0.041 \; (R > 0.856) \qquad (5.11)$$

where V = velocity, in feet per second (fps), and R = the number of rotor revolutions per second.

For convenience in field use, the data from the current-meter ratings are reproduced in tables, a sample of which is shown in Figure I.5.22 for a Price AA current meter with cat-whisker and magnetic contact chambers. This is the rating table for the rating equation 5.8 shown above. This rating table is the format generally used in the United States. The velocities corresponding to a range of 3 to 350 revolutions of the rotor within a period of 40 to 70 seconds are listed in the tables. This range in revolution and time has been found to cover general field requirements. To provide the necessary information for extending a table for the few instances where extensions are required, the equation of the rating table is shown in the heading.

Rating tables for current meters used in the United Kingdom differ from the United States tables in that table look-up is based on revolutions per second. Figure I.5.23 is an example of the first section of a rating table for the miniature Braystoke current meter.

Rating tables are used primarily when current meter measurements are made with hand notekeeping equipment. This method of notekeeping is still in use, however, the use of Electronic Data Loggers (EDLs) is used extensively in many countries. EDLs require only the rating equation(s) which compute the velocity automatically.



UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
Water Resources Division

RATING TABLE FOR TYPE AA CURRENT METER

EQUATIONS: V=2.180R + .020 [illegible] 2.170R + .030 — Std Rating No. 1

Time in Seconds	3	5	7	10	15	20	25	30	40	Time in Seconds
40	.183	.292	.401	.565	.837	1.11	1.38	1.65	2.20	40
41	.180	.286	.392	.552	.818	1.08	1.35	1.62	2.13	41
42	.176	.280	.383	.539	.799	1.06	1.32	1.58	2.10	42
43	.172	.273	.375	.527	.780	1.03	1.29	1.54	2.05	43
44	.169	.268	.367	.515	.763	1.01	1.26	1.51	2.00	44
45	.165	.262	.359	.504	.747	.989	1.23	1.47	1.96	45
46	.162	.257	.352	.494	.731	.968	1.20	1.44	1.92	46
47	.159	.252	.345	.484	.716	.948	1.18	1.41	1.88	47
48	.156	.247	.338	.474	.701	.928	1.16	1.38	1.84	48
49	.153	.242	.331	.465	.687	.910	1.13	1.35	1.80	49
50	.151	.238	.325	.456	.674	.892	1.11	1.33	1.76	50
51	.148	.234	.319	.447	.661	.875	1.09	1.30	1.73	51
52	.146	.230	.313	.439	.649	.858	1.07	1.28	1.70	52
53	.143	.226	.308	.431	.637	.843	1.05	1.25	1.67	53
54	.141	.222	.303	.424	.626	.827	1.03	1.23	1.63	54
55	.139	.218	.297	.416	.615	.813	1.01	1.21	1.61	55
56	.137	.215	.292	.409	.604	.799	.993	1.19	1.58	56
57	.135	.211	.288	.402	.594	.785	.976	1.17	1.55	57
58	.133	.208	.283	.396	.584	.772	.960	1.15	1.52	58
59	.131	.205	.279	.389	.574	.759	.944	1.13	1.50	59
60	.129	.202	.274	.383	.565	.747	.928	1.11	1.47	60
61	.127	.199	.270	.377	.556	.735	.913	1.09	1.45	61
62	.125	.196	.266	.372	.547	.723	.899	1.07	1.43	62
63	.124	.193	.262	.366	.539	.712	.885	1.06	1.40	63
64	.122	.190	.258	.361	.531	.701	.872	1.04	1.38	64
65	.121	.188	.255	.355	.523	.691	.858	1.03	1.36	65
66	.119	.185	.251	.350	.515	.681	.846	1.01	1.34	66
67	.118	.183	.248	.345	.508	.671	.833	.996	1.32	67
68	.116	.180	.244	.341	.501	.661	.821	.982	1.30	68
69	.115	.178	.241	.336	.494	.652	.810	.968	1.28	69
70	.113	.176	.238	.331	.487	.643	.799	.954	1.27	70
	3	5	7	10	15	20	25	30	40	

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
Water Resources Division

RATING TABLE FOR TYPE AA CURRENT METER

Actual Rating Limits: 0.25 to 8.0 feet per second — Date: 05-05-70

Time in Seconds	50	60	80	100	150	200	250	300	350	Time in Seconds
40	2.74	3.28	4.37	5.45	8.17	10.88	13.59	16.30	19.02	40
41	2.68	3.21	4.26	5.32	7.97	10.62	13.26	15.91	18.55	41
42	2.61	3.13	4.16	5.20	7.78	10.36	12.95	15.53	18.11	42
43	2.55	3.06	4.07	5.08	7.60	10.12	12.65	15.17	17.69	43
44	2.50	2.99	3.98	4.96	7.43	9.89	12.36	14.83	17.29	44
45	2.44	2.92	3.89	4.85	7.26	9.67	12.09	14.50	16.91	45
46	2.39	2.86	3.80	4.75	7.11	9.46	11.82	14.18	16.54	46
47	2.34	2.80	3.72	4.65	6.96	9.26	11.57	13.88	16.19	47
48	2.29	2.74	3.65	4.55	6.81	9.07	11.33	13.59	15.85	48
49	2.24	2.69	3.57	4.46	6.67	8.89	11.10	13.32	15.53	49
50	2.20	2.63	3.50	4.37	6.54	8.71	10.88	13.05	15.22	50
51	2.16	2.58	3.43	4.28	6.41	8.54	10.67	12.79	14.92	51
52	2.12	2.53	3.37	4.20	6.29	8.38	10.46	12.55	14.64	52
53	2.08	2.49	3.31	4.12	6.17	8.22	10.27	12.31	14.36	53
54	2.04	2.44	3.24	4.05	6.06	8.07	10.08	12.09	14.09	54
55	2.00	2.40	3.19	3.98	5.95	7.92	9.89	11.87	13.84	55
56	1.97	2.35	3.13	3.90	5.84	7.78	9.72	11.65	13.59	56
57	1.93	2.31	3.08	3.84	5.74	7.64	9.55	11.45	13.35	57
58	1.90	2.27	3.02	3.77	5.64	7.51	9.38	11.25	13.12	58
59	1.87	2.24	2.97	3.71	5.55	7.39	9.22	11.06	12.90	59
60	1.84	2.20	2.92	3.65	5.45	7.26	9.07	10.88	12.69	60
61	1.81	2.16	2.88	3.59	5.37	7.14	8.92	10.70	12.48	61
62	1.78	2.13	2.83	3.53	5.28	7.03	8.78	10.53	12.28	62
63	1.75	2.10	2.79	3.47	5.20	6.92	8.64	10.36	12.09	63
64	1.72	2.06	2.74	3.42	5.12	6.81	8.51	10.20	11.90	64
65	1.70	2.03	2.70	3.37	5.04	6.71	8.38	10.05	11.71	65
66	1.67	2.00	2.66	3.32	4.96	6.61	8.25	9.89	11.54	66
67	1.65	1.97	2.62	3.27	4.89	6.51	8.13	9.75	11.37	67
68	1.62	1.94	2.58	3.22	4.82	6.41	8.01	9.60	11.20	68
69	1.60	1.92	2.55	3.17	4.75	6.32	7.89	9.46	11.04	69
70	1.58	1.89	2.51	3.13	4.68	6.23	7.78	9.33	10.88	70
	50	60	80	100	150	200	250	300	350	

Figure I.5.22. Standard current meter rating Table No. 2, for Price AA current meters with cat-whisker and magnetic contact chambers

Price current meters which have been rated by means of rod suspension, and then by means of cable suspension using Columbus-type weights and hangers, have not shown significant differences in rating. Therefore suspension coefficients are not needed and none should be used, if weights and hangers are properly used.

General summary of current meter calibration

As a result of many years of calibration of current meters at hydraulics laboratories the following basic conclusions have been reached:

(a) Efficient maintenance of a current meter is the most important factor governing the accuracy of velocity measurements. This is particularly the case for measurements of low speeds;

(b) The lower the minimum speed of response of a current meter the lower is the speed of flow which is measurable with confidence;

(c) Barring accidents the calibration of current meters shows only a small change with time. This is particularly the case for the higher velocities. Changes of calibration at the lower end of the speed range are proportionally higher and depend as much on the cleanliness and lubrication as on any wear and tear of moving parts;

(d) Current meters which embody mechanical arrangements for making and breaking an electric circuit have a higher minimum speed of response and a less consistent performance at low speeds than those having magnets and reed switches;

(e) The accuracy of calibration equations is greater than the accuracy of the individual calibration points;

(f) The spread of results when several repeat calibrations are made of one meter is much smaller than the spread of results when several current meters of one make and type are calibrated;

(g) The uncertainty of group calibration of modern current meters is not large, however, and provided that it is recognised that higher uncertainty may apply to measurements made with meters having group calibration, such calibrations may be used, and indeed recommended, for routine gauging;

(h) The spread of calibration results at relatively high speeds is less than it is at low speeds. In particular some precision-built meters having

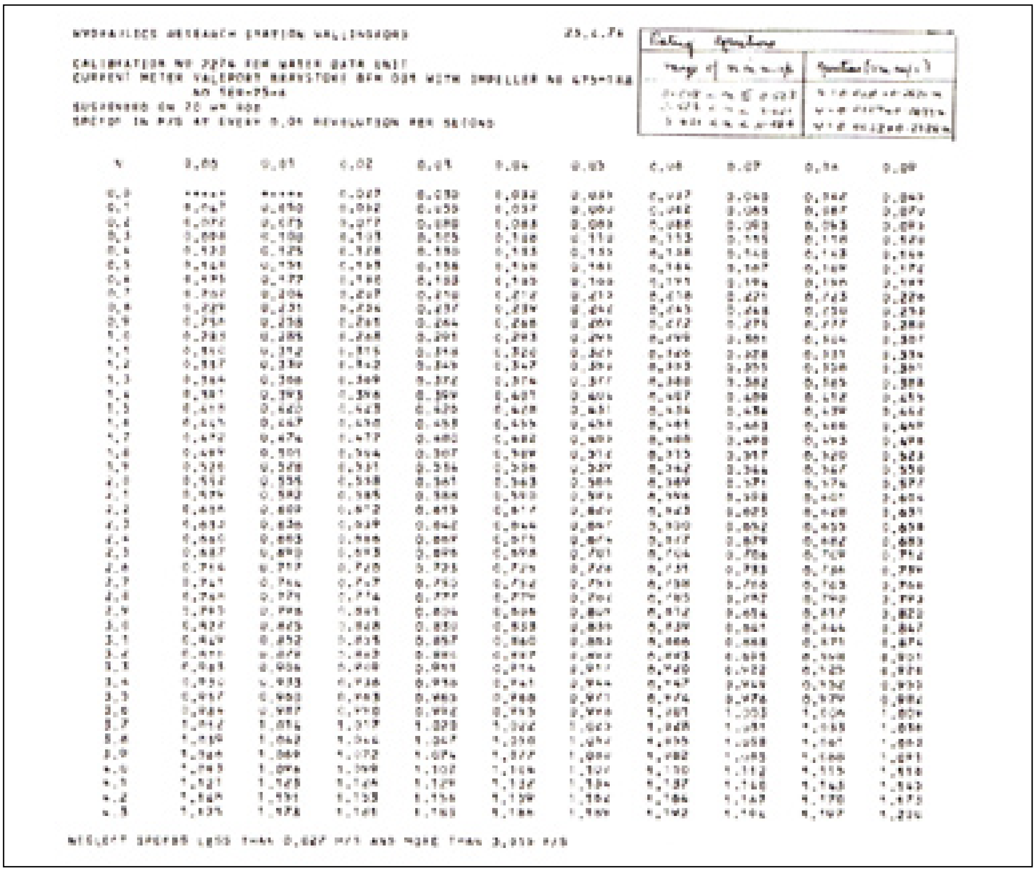


Figure I.5.23. Section 1 of rating table for miniature Braystoke current meter

metal propellers have calibrations such that 19 out of 20 of them lie within ± 1.5 per cent of the average of the group. With some propellers the uncertainty is about ± 0.8 per cent. The spread of calibration results for groups of meters having plastic propellers of almost neutral buoyancy is between ± 2 and ± 2.5 per cent at the higher speeds. At the lower speeds, where some of the main difficulties of flow measurement lie, the spread of results of the meters with metal propellers and magnet and reed switch operation is also slightly smaller than that of the meters with plastic propellers. The performance of the meters with plastic propellers is superior at low speeds to that of meters embodying mechanical operation of contacts;

(i) All the comments in the previous paragraph apply to the meters in the condition in which they were calibrated – they were well maintained;

(j) Meters having plastic propellers and water-lubricated bearings require little attention and adjustment and are ideal for measurements of low speeds and for use generally at sites which are remote from workshop facilities;

(k) Differences in calibration of two well-maintained current meters of the same type are systematic. For special studies involving maximum accuracy individual calibrations are preferred. For example, in the Czech Republic individual calibrations, not older than 2 years, must be valid for all types of current meters (mostly Ott meters in the Czech Republic).

5.3.12 **Sounding equipment**

Sounding (determination of depth) is commonly done mechanically, the equipment used depending on the type of measurement being made. Depth and position in the vertical are measured by a rigid rod or by a sounding weight suspended from a cable. The cable is controlled either by a reel or by a handline. A sonic sounder is also available but it is usually used in conjunction with a reel and a sounding weight. The various equipment used for sounding is described in the following paragraphs.

Wading rods, top setting

The two types of wading rods commonly used are the top-setting rod and the round rod. The top-setting rod is preferred because of the convenience in setting the meter at the proper depth and because the hydrographer can keep his hands dry.

The top-setting wading rod, as shown in Figure I.5.24, has a 1.27 cm hexagonal main rod for measuring depth and a 0.95 cm diameter round rod for setting the position of the current meter.

The rod is placed in the stream so the base plate rests on the streambed and the depth of water is read on the graduated main rod. When the setting rod is adjusted to read the depth of water, the meter is positioned automatically for the 0.6-depth method, as shown in Figure I.5.25. The 0.6-depth setting is the setting measured down from the water surface. It is the same as the 0.4 depth position when measured up from the streambed. When the depth of water is divided by 2, and this value is set on the setting rod, the meter would be at the 0.2-depth position up from the streambed. When the depth of water is multiplied by 2, and this value is set, the meter would be at the 0.8-depth position up from the streambed. These two positions represent the conventional 0.2- and 0.8-depth positions in reverse.

Wading rods, round

The round wading rod, as shown in Figure I.5.26, consists of a base plate, lower section, three or four intermediate sections, sliding support and a rod end (not essential). The parts are assembled as shown in Figure I.5.27. The meter is mounted on the sliding support and is set at the desired position on the rod by sliding the support. The

Figure I.5.24. Top-setting wading rod with meter attached

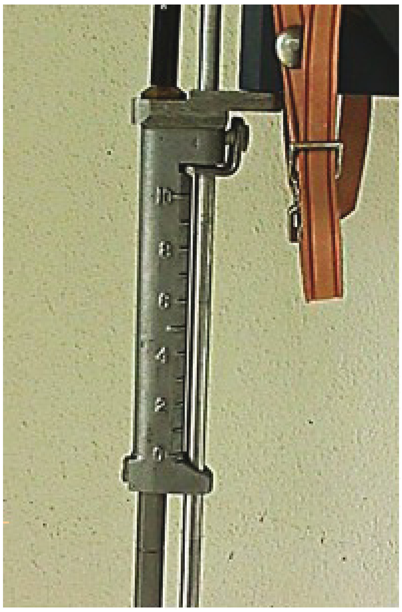


Figure I.5.25. Close-up view of setting scale on handle of top-setting wading rod

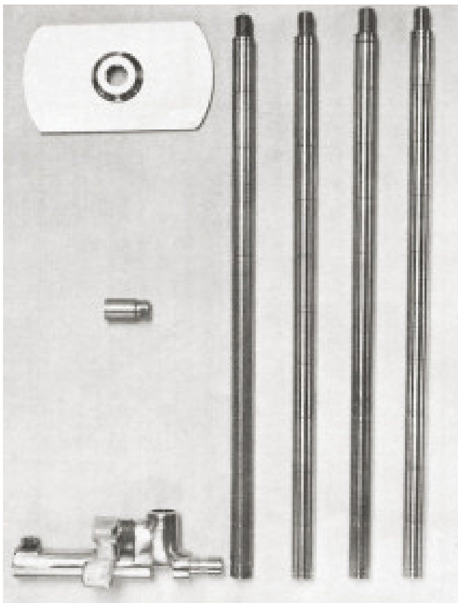


Figure I.5.26. Parts for the round wading rod

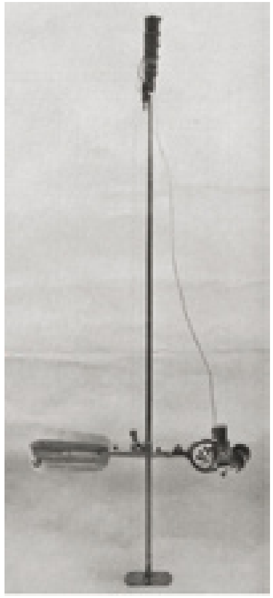


Figure I.5.27. Round wading rod with meter attached

round rod has the advantage that it can be assembled into various lengths using the 0.3 m sections and is easy to store and transport when disassembled.

Winter style suspension rods

Measurements made under ice cover should use the WSC winter sounding rods, either in the 1.27 cm or 2.54 cm diameter versions. These rods are available in sections so that the desired length can be assembled. A special foot fits the lower section, and the rods will accommodate the winter style current meter yoke, as shown in Figure I.5.28.

Sounding weights

If a stream is too deep or too swift to wade, the current meter is suspended in the water from a boat, bridge or cableway. A sounding weight is suspended below the current meter to keep it stationary in the water. The weight also prevents damage to the meter when the assembly is lowered to the streambed.

The sounding weights used in many countries are the Columbus weights, commonly called the C type, are shown in Figure I.5.29. The weights are streamlined to offer minimum resistance to flowing water. The weights are available in 7, 14, 23, 34, 45, 68, 91 and 136 kg sizes. Each weight has a vertical slot and a drilled horizontal hole to accommodate a weight hanger and securing pin.

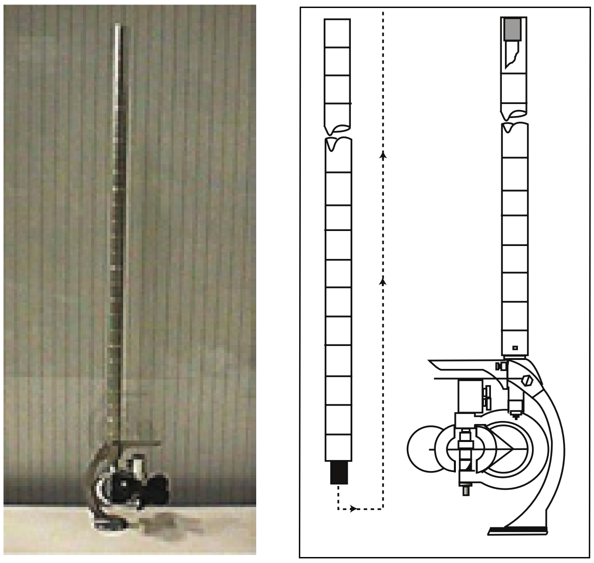


Figure I.5.28. WSC winter style suspension rods

Hanger bars

The weight hanger is attached to the end of the sounding line by a connector. The current meter is attached to the hanger bar beneath the connector and the sounding weight is attached to the lower end of the hanger bar.

There are three types of weight hanger bars, as shown in Figure I.5.30. The sounding weight hangers are designed to accommodate the weights of the various sizes. The height of the meter rotor above

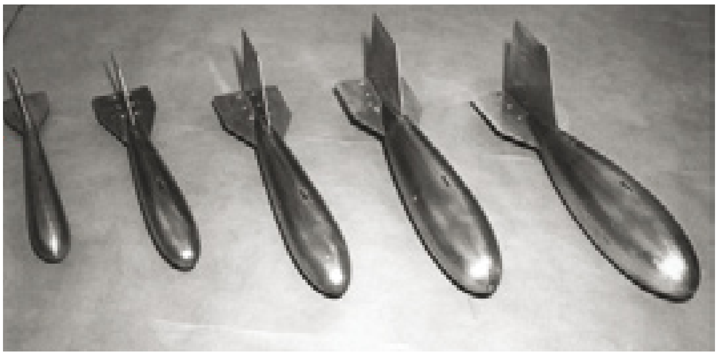


Figure I.5.29. Columbus C-type sounding weights, 7- through 136 kg

Figure I.5.30. Sounding weight hanger bars and hanger pins

the bottom of the sounding weight must be considered in calculations to position the meter for velocity observations at various percentages of the the stream depth.

Weight-hanger pins of various lengths, as shown in Figure I.5.30, are available for attaching the sounding weight to the hanger bar. The stainless steel pins are threaded on one end to screw into the hanger bar and slotted on the other.

For measurement under ice, the 14- or 23 kg C-type weights should be used with a special, collapsible hanger assembly, as shown in Figure I.5.31, that is capable of being passed through a 20 cm hole in the ice.

Sounding reels

In general, a sounding reel has a drum for winding the sounding cable, a crank and ratchet assembly for raising and lowering the weight or holding it in any desired position and a depth indicator. Several different types of sounding reels, as used by the USGS, are available for use with the Columbus C-type weights. Table I.5.3 contains detailed information on each of the five reels most commonly used.

The A-pack reel, as shown in Figure I.5.32, is light, compact, and ideal for use at cableway sites a considerable distance from the highway. It can also be used on cranes, bridge boards, and boat booms. The Canfield reel, is also compact with uses similar to that of the A-pack reel.

The A-55 reel is for general purpose use with the lighter sounding weights.

The B-56 reel, as shown in Figure I.5.34, can handle all but the heaviest sounding weights and has the advantage that it can be used with a hand-crank or power equipment.

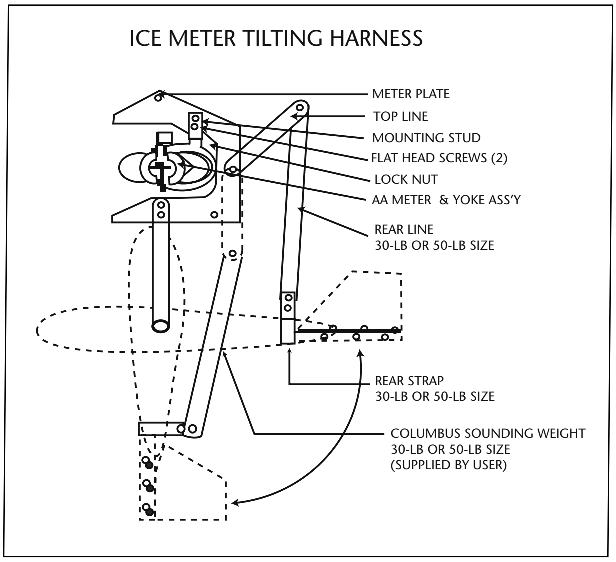


Figure I.5.31. Collapsible hanger assembly for use with 14 and 23 kg C-type weights, for measurements under ice

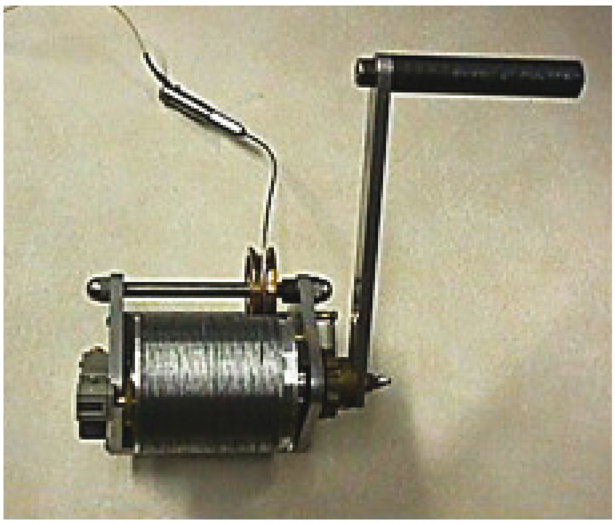


Figure I.5.32. A-pack reel

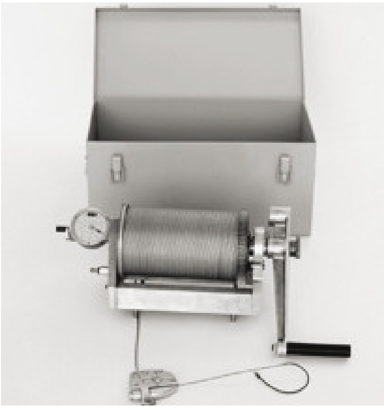


Figure I.5.33. Two versions of the B-56 reel

Figure I.5.34. E-53 reel

The E-53 reel is the largest reel commonly used for current-meter measurements. This reel, as shown in Figure I.5.34, will handle the heaviest sounding weights and is designed exclusively for use with power equipment. It has a hand-crank for emergency use.

Sounding cable

Ellsworth reverse-lay two-conductor stranded cable is normally used on all sounding reels except the single-conductor Canfield reel which uses galvanized steel aircraft cord. Ellsworth cables are normally available in 2.1, 2.5, and 3.2 mm diameters. It is important that the appropriate size cable-laying sheave be used on the reels.

For safety purposes when measuring floods it is important that the sounding cable be connected to the sounding reel in such a way that the cable will break in the event that heavy debris is caught and cannot be released. The cable will usually unwind from the sounding reel until it reaches its end, at which point there is danger to the equipment and the streamgauger unless the cable is cut or breaks. It is recommended that some of the cable strands be pre-cut when installing the cable on the reel so that the remaining strands will break when the load reaches a specified limit. Table I.5.4 provides information about cable strength and number of strands to cut to provide the necessary safety margin.

Connectors

A connector is used to join the end of the sounding cable to the sounding-weight hanger. The three types of connectors generally used are types B, Au, and pressed sleeve as shown in

Table I.5.3. Sounding reel data

Reel	*Sounding Cable*	*Cable diameter (mm)*	*Drum circumference (m)*	*Cable capacity (m)*	*Maximum weight (kg)*	*Depth indicator*	*Brake*	*Type operation*
A-pack	Ellsworth	2.1	0.3	14	23	Counter	No	Hand
Canfield	Single conductor*	1.6	0.3	14	23	Counter	No	Hand
A-55	Ellsworth	2.1 2.5	0.3	29 24	23 45	Self computing	No	Hand
B-56	Ellsworth	2.5 3.2	0.46	44 35	68 91	Self computing	Yes	Hand or power
E-53	Ellsworth	2.5 3.2	0.6	63 50	68 136	Self computing	Yes	Power

* Some Canfield reels have been converted to double-conductor cable but most of them are still used as single-conductor reels.

Figure I.5.35. The type-B connector is used with A-55, B-56, and E-53 reels. The Au connector is used with the A-pack and Canfield reels although the pressed-sleeve connector can be used on these reels. The pressed-sleeve connector is used mainly on handlines.

Depth indicators

A computing depth indicator, as shown in Figure I.5.36, is used on the A-55, B-56, and E-53 reels. The stainless-steel indicator is less than 8 cm in diameter and has nylon bushings which do not require oil. The main dial is graduated in metres and tenths of a metre from 0 to 3 m. The depth is indicated by a pointer. Tens of metres are read on a numbered inner dial through an aperture near the top of the main dial.

The main dial has a graduated spiral to indicate directly the 0.8-depth position for depths up to 10 m.

The A-pack and Canfield reels, as shown in Figures I.5.32 and I.5.33, are equipped with counters for indicating depths.

Handlines

Handlines, as shown in Figure I.5.37, are devices used for making discharge measurements from bridges using a 7 or 14 kg sounding weight. Some of the advantages of the handline are that it is easily set up, it eliminates the use of a sounding reel and supporting equipment and it reduces the difficulty in making measurements from bridges which have interfering members. The disadvantages of the handline are that there is a greater possibility of making errors in determining depth because of slippage of the handline or measuring scale or tape and that it requires more physical exertion especially in deep streams. Handlines can be used from cable cars although this is not recommended because of the disadvantages mentioned above.

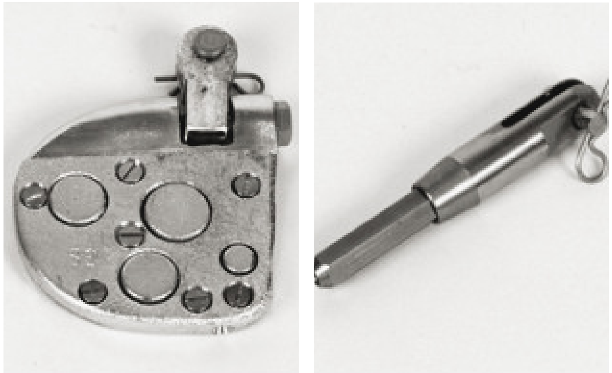


Figure I.5.35. Connectors for attaching sounding cable to sounding-weight hanger; type B (left) and pressed sleeve (right)

Figure I.5.36. Computing depth indicator

Ellsworth cable is recommended for handlines because of its flexibility and durability Two-conductor electrical service cord is used between the headset connector and the handline reel.

The pressed-sleeve connector or the Au connector are used on handlines because they are lighter in weight than the type-B connector yet strong enough for the sounding weights used with handlines.

Table I.5.4. Breaking loads for Ellsworth stranded cable

Sounding cable	*Diameter, (mm)*	*Total number of strands*	*Rated total breaking load, (kg)*	*Recommended breaking load, (kg)*	*Number of strands to cut*	*Number of strands to remain*
Ellsworth 2.1	2.1	36	227	113	15	21
Ellsworth 2.5	2.5	30	454	227	15	15
Ellsworth 3.2	3.2	30	680	227	20	10

Figure I.5.37. Handline

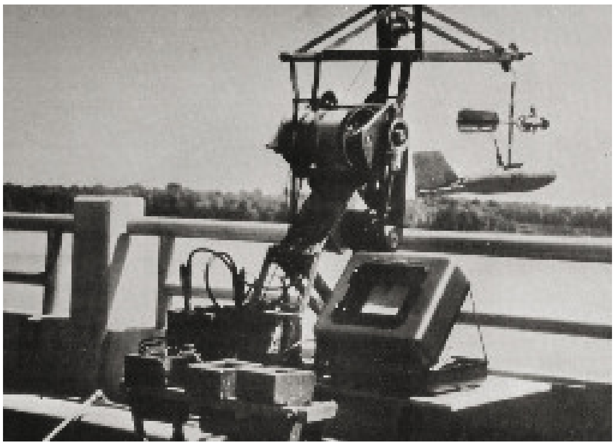


Figure I.5.38. Sonic sounding recorder

Power unit

A power unit, is available for the B-56 and E-53 reels to raise and lower the sounding weight and meter. The power unit can be used with 6, 12, 18 or 24-volt batteries.

Sonic sounder

A commercial, compact, portable sonic sounder has been adapted to measure stream depth. The sonic sounding recorder is shown in Figure I.5.38 and the disassembled weight, compass and transducer is shown in Figure I.5.39.

The sounder is powered by either a 6- or 12-volt storage battery and will operate continuously for 10 hours on a single battery charge. Three recording speeds are available and four operating ranges, to a maximum depth of 80 m. The sounder is portable, weighing only 21 kg. The depth recorded is from the water surface to the streambed. The transducer has a narrow beam

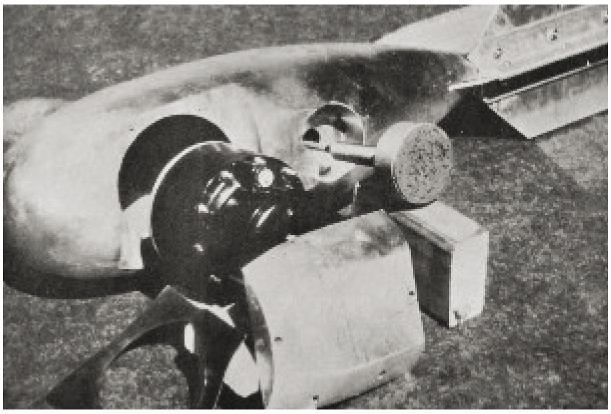


Figure I.5.39. Sounding weight with compass and sonic transducer ready for assembly

angle of 6 degrees which minimizes errors on inclined streambeds and allows the hydrographer to work close to piers or other obstructions.

Measurements can be made with this equipment without lowering the meter and weight to the streambed. As soon as the weight is in the water the depth will be recorded. The meter can then be set at the 0.2 depth or just below the water surface where a velocity reading is obtained. Then a coefficient is applied to convert measured velocity to the mean in the vertical.

Temperature change affects the sound propagation velocity, but this error is limited to about ± 2 per cent in fresh water. This error can be eliminated completely by adjusting the sounder to read correctly at a particular average depth determined by other means.

5.3.13 Width-measuring equipment

The horizontal distance to any vertical in a cross section is measured from an initial point on the bank. Cableways and bridges used regularly for making discharge measurements are commonly marked at 1, 2, 3, or 6 m intervals by paint marks. Distance between markings is estimated or measured with a rule or pocket tape. For measurements made by wading, from boats or from unmarked bridges, steel or metallic tapes or tag lines are used. For very wide streams of about 750 m or more, where conventional measuring methods cannot be used, surveying methods and Global Positioning Systems (GPS) may be used.

Tapes and tag lines

Tag lines used for wading measurements are usually made of either galvanized steel aircraft

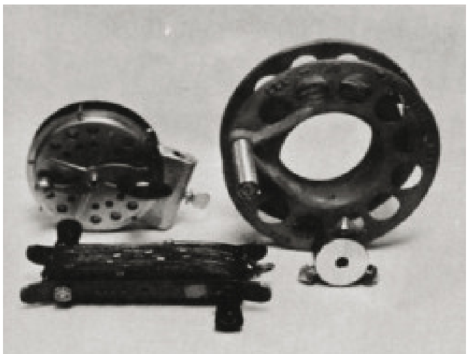


Figure I.5.40. Lee-Au (left), Pakron (right) with removable hub in front, and Columbus Type-A (bottom) tag line reels

Figure I.5.41. Kevlar tag line reel

Figure I.5.42. Horizontal-axis boat tag line reel without a brake

cord with solder beads at measured intervals, or Kevlar, which is marked with black ink and waxed to resist abrasion. A Kevlar tag line consists of a Kevlar core with a nylon jacket.

The standard markings for Kevlar tag lines is one mark every 0.6 m (2 feet), two marks every 3 m (10 feet), and 3 marks every 30.5 m (100 feet). The standard lengths of tag lines are 91, 122, and 152 m, but other sizes are available. In Europe the standard lengths of tag lines are 100, 125 and 150 m but other sizes are available.

Three types of tag line reels used for the steel tag lines are the Lee-Au, Pakron, and Columbus type A, as shown in Figure I.5.40. The Kevlar tag line is shown in Figure I.5.41.

Larger reels, used for boat measurements, are designed to hold more than 900 m of 3.2 mm diameter steel tag line. Three different types of reels are available as follows:

(a) A heavy-duty, horizontal-axis reel without a brake, and with a capacity of more than 600 m of 3.2 mm diameter steel cable as shown in Figure I.5.42;
(b) A heavy-duty, horizontal-axis reel with a brake, and with a capacity of more than 900 m of 3.2 mm diameter steel cable;
(c) A vertical-axis reel without a brake, and with a capacity of almost 250 m of 3.2 mm diameter cable as shown in Figure I.5.43.

Surveying methods of width measurement, transit and electronic total station

For very wide streams where it is not practical to string a tag line for discharge measurements from a boat, surveying methods can be used to measure stream width and stationing for measurement points. Surveying methods require the use of a transit, as shown in Figure I.5.44, or electronic total station instrumentation. The procedure used to determine width with a transit is probably not used very much anymore but it is described in the section of this report on boat measurements.

With the advent of electronic total station instruments, a direct reading of the distance from the instrument setup point to the boat can be made. One example of a commercially available total station instrument is shown in Figure I.5.45. Some of these instruments require a reflector target at the point where a measurement is desired (in this case the boat), however total station instruments are also available that provide accurate measurements of distance without a reflector target. Accurate distance measurements with total station instruments can be made over distances of more than 1.5 km, provided the boat can be seen and is not obstructed by intervening objects.

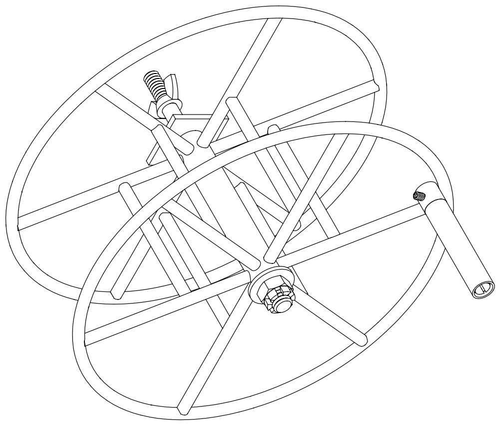


Figure I.5.43. Vertical-axis boat tag line reel

Figure I.5.44. Surveying transit and tripod

Figure I.5.45. Total station surveying instrument

Differential Global Positioning System

Stationing of measurement points for very wide streams, such as flood plains that may be several miles wide or large tidal estuaries, can be determined with a Global Positioning System (GPS) instrument such as that shown in Figure I.5.46. GPS instruments utilize satellite telemetry from a network of 24 satellites and radio trilateration to compute positions for any point on the earth. To obtain the accuracy necessary for a discharge measurement the raw GPS positions must have differential corrections applied on the basis of simultaneous readings at a base station. Some GPS units contain built-in differential correction receivers that automatically make the corrections instantaneously. Other GPS units may use a separate receiver that attaches to the GPS unit with a cable. In either case base station data are received by radio signal from nearby ground base stations.

A surveying type of GPS unit with capability of storing and recalling data is preferred. These units may or may not have built-in or attached differential correction receivers. If instantaneous differential corrections are not made, then the coordinate data must be post-processed using correction data obtained after-the-fact from a separate GPS base station. Various agencies collect and provide the base station data via the internet. Coordinate data for the measurement points are downloaded from the GPS unit to a computer for post-processing. Software is available to make the differential corrections, to compute corrected coordinates of the measurement points, to automatically compute distances between measuring points and to plot a map of the measurement points.

Figure I.5.46. Global Positioning System (GPS) mobile surveying unit by Topcon

Accuracy of GPS coordinates will vary depending on the type of GPS unit and whether or not differential corrections are made. Coordinates without differential corrections can be in error by as much as ± 90 m because of various errors in the system. This obviously is not acceptable for discharge measurements. However with proper care in making observations and after differential corrections are made, errors can be reduced to less than ± 1 m and even less in ideal conditions. This is acceptable for wide flood plains and estuaries.

5.3.14 **Special equipment and suspension assemblies**

Special equipment and suspension assemblies are necessary for making current-meter measurements from cableways, bridges, boats and ice. Basic equipment, such as meters, weights, and reels, have already been described in previous sections of this Manual.

Figure I.5.47. Sit-down cable car in operation

Figure I.5.48. Stand-up cable car in operation

Cableway equipment

The cableway provides a track for the operation of a manned cable car from which the hydrographer makes a current-meter measurement. Most cableways have a clear span of 300 m or less, although a few structures have been built with clear spans approaching 600 m. The design and construction of cableways are described in detail by Wagner (1995).

Cable cars provide a movable platform from which the hydrographer, sounding reel and other necessary equipment are supported. The newer versions of these cable cars are fabricated from aluminium, have a standard follower brake and have integral reel mounts which will accept all standard sounding reels. Cable cars can also be equipped with the Sand point type cable car brake which allows the cable car to be slowed or stopped. Both sit-down and stand-up types of manually propelled cable cars are used in stream gauging, as shown in Figures I.5.47 and I.5.48, and have space for two people to work. Some older cable cars still in use are fabricated partially from wood, may or may not have permanent reel mounts, and may have space for only one person.

Manned cable cars are moved from one point to another on the cableway by means of cable-car pullers, as shown in Figure I.5.47. The standard car puller is a cast aluminum handle with a snub attached. The snub, usually four-ply belting, is placed between one of the car sheaves and the cable to prevent movement of the car along the cable. A second-type puller, is used when a car is equipped with a follower brake. A third type, the Colorado River cable-car puller, is the same in principle as the puller used on cars equipped with a follower brake type of puller. A photo of the Sand point cable car brake is also shown in Figure I.5.49.

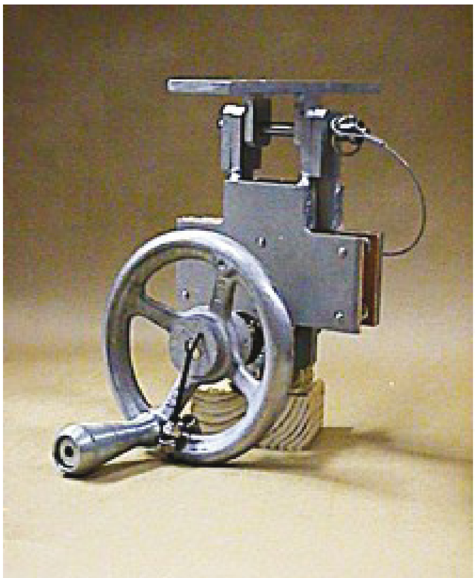


Figure I.5.49. Sand point cable car brake

Power-operated cable cars, both battery and gasolicne powered, as shown in Figure I.5.50, are available for extremely long spans or other special situations where extensive streamgauging and monitoring is required. The power assist on these cable cars is also utilized to operate a type E sounding reel.

Special, unmanned, carrier cables are sometimes used on deep, narrow streams for discharge measurements as well as for sediment sampling. The cable car and sounding equipment can be remotely operated from the stream bank, as shown in Figure I.5.51. They are used in areas where it is impossible to wade, where no bridges are available, and where it is not practical to build a complete manned cableway. Unmanned cableways are used fairly extensively throughout the world.

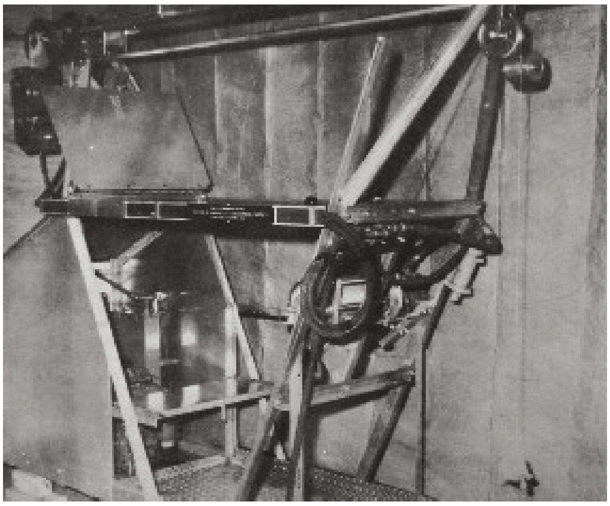


Figure I.5.50. Gasoline powered cable car

Figure I.5.52. Measuring from a bridge with a handline

Figure I.5.51. Remotely operated cable car assembly

Figure I.5.53. Measuring from a bridge with a bridge board

Bridge equipment

Streamflow measurements are frequently made from a bridge. The meter and sounding weight can be supported by a handline, a bridge board or by a sounding reel mounted on a crane.

Handlines and bridge boards

A handline, as described in a previous section of this report, is the simplest form of bridge measuring equipment. It doesn't require any separate reels or handling equipment but it can only be used with light sounding weights such as the 7 and 14-kg size. It also requires that depth must be measured with tags and a tape or measuring stick. Figure I.5.52 shows a handline in use.

A bridge board is a portable platform made from wood or metal upon which a small reel can be mounted. Bridge boards may be used with an A-pack, A-55, or B-56 sounding reel and weights up to 34 kg. A bridge board is usually about 1.5 to 2.5 m long with a sheave at one end over which the meter cable passes and a reel seat near the other end. The board is placed on the bridge rail so that the force exerted by the sounding weight suspended from the reel cable is counterbalanced by the weight of the sounding reel. The bridge board may be hinged near the middle to let one end be placed on the sidewalk or roadway. Figure I.5.53 shows a bridge board in use.

Portable cranes

Two types of hand-operated portable cranes are the type A for weights up to 45 kg, and the type E for heavier weights. The type A crane mounts on a 3-wheel or 4-wheel base, or truck, and the type E crane mounts on a 4-wheel base, or truck. Cranes can be easily moved by hand along the sidewalk or floor of the bridge. Figure I.5.54 shows a type A

Figure I.5.54. Type A crane mounted on a 3-wheel base

Figure I.5.55. Type E crane mounted on a 4-wheel base

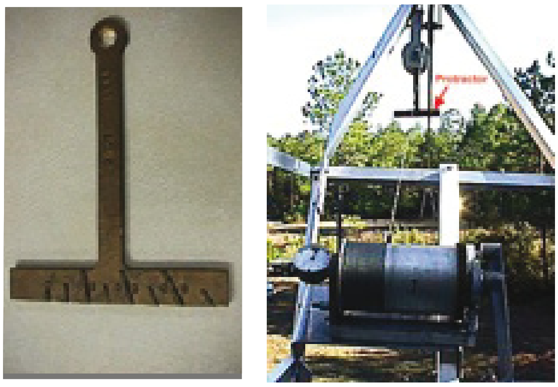


Figure I.5.56. Protractor used for measuring vertical angles

crane mounted on a 3-wheel base, and Figure I.5.55 shows a type E crane mounted on a 4-wheel base.

Any of the reels described in Table I.5.3 may be used on either of the portable cranes, except that the power driven reels (B-56 and E-53) are used only with the Type E crane. Various combinations of cranes, bases, and reels are possible.

All cranes are designed so that the crane can be tilted forward over the bridge rail far enough for the meter and weight to clear most rails and be lowered to the water. Where bridge members obstruct passage of the crane along the bridge, the weight and meter can be raised and the crane can be tilted back to pass by the obstruction.

Cast-iron counterweights weighing 27 kg each are used with four-wheel base cranes. The number of such weights needed depends upon the size of sounding weight being supported, the depth and velocity of the stream and the amount of debris being carried by the stream.

A protractor is used on the outer end of cranes to measure the angle the sounding line makes with the vertical when the weight and meter are dragged downstream by high-velocity water. The protractor is a graduated circle clamped to an aluminum plate. A plastic tube partly filled with colored antifreeze is the protractor index. This tube is fitted in a groove between the graduated circle and the aluminum plate. A stainless-steel rod is attached to the lower end of the plate to ride against the downstream side of the sounding cable. The protractor will measure vertical angles from – 25 degrees to + 90 degrees. Figure I.5.56 is a close-up view of a protractor mounted at the outer end of the boom.

Power driven cranes

Many special arrangements for measuring from bridges have been devised to suit a particular purpose. Vehicle-mounted cranes are often used for measuring from bridges over larger rivers. Monorail streamgauging cars have also been developed for large rivers. The car is suspended from the substructure of bridges by means of I-beams.

Velocity-azimuth-depth assemblies

The velocity-azimuth-depth assembly, commonly called VADA, combines a sonic sounder to record depth, a remote-indicating compass to indicate the direction of flow and a Price current meter to permit observations of velocity. The VADA equipment is shown in Figure I.5.57 mounted on the four-wheel crane with an E-53 power-driven reel. Incorporated within the remote-indicator box is the battery for the current-meter circuit, the headphone jacks and the two-conductor jack for the sonic sounder. A switch allows the remote-indicating unit to be used separately or in conjunction with the sonic sounder. This assembly is useful in tidal investigations and other special studies as well as at regular gauging stations, where it is desirable to determine the direction of flow beneath the surface when it may differ from that at the surface. Although this equipment can be used it is being often replaced now by the Acoustic Doppler Current Profiler (ADCP) methods described in the next chapter.

Boat equipment

Four basic types of boat measurements are made: the manual stationary boat, the manual moving boat, the automatic moving boat and the ADCP moving boat. Only the equipment requirements for the manual stationary boat method are given in

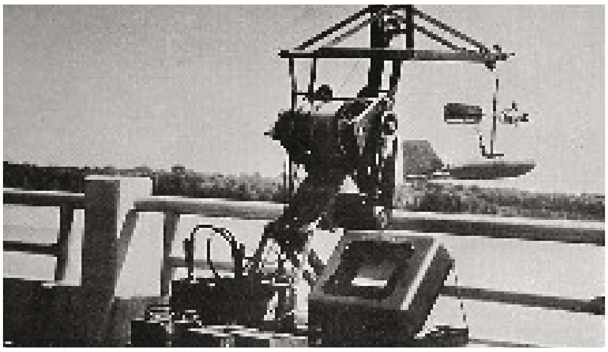


Figure I.5.57. Velocity-azimuth-depth assembly (VADA)

Figure I.5.58. Manual stationary boat equipment assembly

this section because it is the only boat method considered a conventional current meter measurement. Equip-ment requirements for each of the moving boat measurement types are described in Chapter 6.

Manual stationary boat

The manual stationary boat method uses a boat as a platform for the streamgauger and the sounding equipment. The boat is usually attached to a tag line or cable to stabilize the boat at each vertical where soundings are made. The heavy duty tag lines required for boat measurements are described in a previous section of this Manual.

Special equipment assemblies, as shown in Figure I.5.58, are necessary to suspend the meter from the boat when the depths are such that rod suspension cannot be used. A crosspiece reaching across the boat is clamped to the sides of the boat and a boom attached to the center of the crosspiece extends out over the bow. The crosspiece is equipped with a guide sheave and clamp arrangement at each end to attach the boat to the tag line and make it possible to slide the boat along the tag line from one station to the next. A small rope can be attached to these clamps so that in an emergency a tug on the rope will release the boat from the tag line. The crosspiece also has a clamp that prevents lateral movement of the boat along the tag line when readings are being made. The boom consists of two structural aluminum channels, one telescoped within the other to permit adjustments in length. The boom is equipped with a reel plate on one end and a sheave over which the meter cable passes on the other. The sheave end of the boom is designed so that by adding a cable clip to the sounding cable a short distance above the connector, the sheave end of the boom can be retracted when the meter is to be raised out of the water. The raised meter is easy to clean and is in a convenient position when not being operated. All sounding reels fit the boat boom except the A-pack and the Canfield, which can be made to fit by drilling additional holes in the reel plate on the boom.

In addition to the equipment already mentioned, the following items are needed when making boat measurements:

(a) A stable boat big enough to support the hydrographers and equipment;
(b) A motor that can move the boat with ease against the maximum current in the stream, although a motor is not always required;
(c) A pair of oars for standby use;
(d) A personal floatation device, or life jacket, for each hydrographer;
(e) A bailing device.

Ice equipment

Current-meter measurements under ice cover are frequently made with a special winter-style sounding rod, as described in a previous section of this report. When depths are too deep for rod suspension an equipment assembly mounted on runners, such as shown in Figure I.5.59, is used to support the meter, sounding weights and reel. A 14 kg Columbus-type weight should be used, which can be lowered through an 0.2 m diameter hole by using the special tilting harness, also shown in Figure I.5.59. A handline can also be used for making ice measurements.

Ice measurements also require special equipment for cutting holes in the ice through which to suspend the meter. The development of power ice drills has eliminated many of the difficulties of

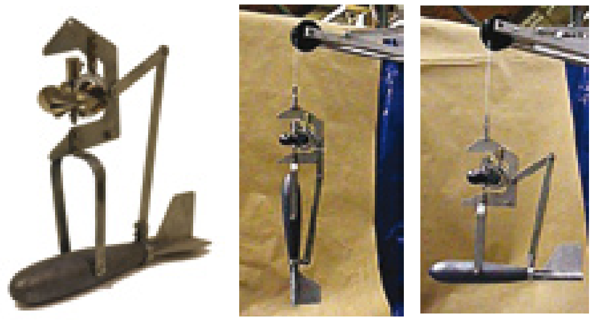


Figure I.5.59. Ice measurement supports, and tilting harness for 30-pound C-type weight

cutting through ice and has reduced considerably the labor and time required to cut the holes. Holes are often cut with a commercial ice drill that cuts a 0.15 or 0.2 m diameter hole. Figure I.5.60 shows two powered ice drills. The drill on the left weighs about 14 kg and under good conditions will cut through 0.6 m of ice in about a minute.

Where it is impractical to use a powered ice drill, ice chisels are used to cut the holes. Ice chisels used are usually 1.2 to 1.4 m long and weigh about 5 kg. The ice chisel is used when first crossing an ice-covered stream to determine whether the ice is strong enough to support the hydrographer. If a solid blow of the chisel blade does not penetrate the ice then it is safe to walk on, providing the ice is in contact with the water.

Some hydrographers supplement the ice chisel with a Swedish ice auger. The cutting blade of this auger is a spade-like tool of hardened steel which cuts a hole 0.15 to 0.20 m in diameter by turning a brace-like arrangement on top of the shaft.

Figure I.5.60. Powered ice drill

When holes in the ice are cut, the water is usually under pressure owing to the weight of the ice, and it comes up in the hole. To determine the effective depth of the stream, ice-measuring sticks are used to measure the distance from the water surface to the bottom of the ice. This is done with a bar about 1.2 m long that is made of strap steel or wood, graduated in metres and tenths of a metre, and has an L-shaped projection at the lower end. The horizontal part of the L is held on the underside of the ice and the depth to that point is read at the water surface on the graduated part of the stick. The horizontal part of the L is at least 0.1 m long so that it may extend beyond any irregularities on the underside of the ice.

5.3.15 Counting equipment and electronic field notebooks

To determine the velocity at a point with a current meter it is necessary to count the revolutions of the rotor in a measured interval of time. The velocity is then obtained from the meter rating table or equation as described in previous sections of this report. The time interval, which can vary from about 40 to 180 seconds, is measured to the nearest second with a stopwatch or Electronic Field Notebook (EFN).

The revolutions of the meter rotor during the observation of velocity are counted by an electric circuit (either by contact wires, magnetic, or optical) that is closed each time a full revolution of the rotor is made. Traditionally a battery and headphone was used to count clicks manually each time the circuit is closed. In many cases compact, comfortable hearing aid phones have been adapted to replace headphones. Although this method is still used in some situations as described in previous sections of this chapter, it has been largely replaced with automated EFNs.

Recent developments in electronics have produced commercially available EFNs designed specifically for the purpose of recording field notes during the process of making a discharge measurement. The recording process is semi-automatic. Information and data must be entered manually for measurements of stream depth, stationing, horizontal angle of flow and equipment. For measurements involving vertical angles, ice or other special conditions, additional information must be entered manually. The notebook performs the task of automatically counting meter revolutions and elapsed times and makes the conversion to stream velocity. It also assists the hydrographer with certain tasks such as locating each subsection so that no more than 10 per cent of the total flow will be included in each subsection. All of the measurement calculations are performed by the notebook to obtain the final discharge and to summarize all pertinent items of the measurement.

Most EFNs can store up to 20 discharge measurement reports with a combined total of up to about 750 subsections. The report produced by the electronic field notebook for a discharge measurement is similar to the paper note-sheets used for manual note-keeping. A header, similar to the paper front sheet, contains site information, equipment information and a summary of measurement data. The report also contains complete measurement data, similar to the paper "inside notes", for all of the individual subsections. In addition the report contains various warning flags and quality control information. Complete reports for each discharge measurement can be downloaded to a computer for viewing, printing and other analysis.

Currently there are several commercially available electronic field notebooks and current meter counters on the market, such as the Aquacalc made by JBS Engineering, the DMX made by Sutron Corporation, the Ott Vota-2 made by Ott and the PVD100 and CSCsp made by Hydrological Services. Two of these units, the Aquacalc electronic field notebook and the Ott Vota-2 current meter counter are shown in Figure I.5.61 All of these units are considered reliable and have been successfully field tested.

5.3.16 **Floats**

Floats have somewhat limited use in stream gauging, but there are two occasions when they prove useful. A float can be used where the velocity is too low to obtain reliable measurements with the current meter. They are also used where flood measurements are needed and the measuring structure has been destroyed or it is impossible to use a meter.

Both surface floats and rod floats are used. Surface floats may be almost anything that floats, such as wooden disks, bottles partly filled or oranges. Floating debris or ice cakes may serve as natural floats. Rod floats are wooden rods weighted on one end so they will float upright in the stream. Rod floats must not touch the streambed and are sometimes made in short sections that can be attached together to form the proper length. A complete description on the use of floats is given in Chapter 8.

Figure I.5.61. Electronic field notebooks, Aquacalc Pro (left) and Ott Vota-2 (right)

Figure I.5.62. Ice creepers for boots and waders

5.3.17 **Miscellaneous equipment**

Several other equipment items are necessary when making discharge measurements or when working in and around rivers, creeks and streams. Waders or boots are needed when wading measurements are made. Waders should be loose fitting even after allowance has been made for heavy winter clothing.

Ice creepers, as shown in Figure I.5.62, strapped on the shoe of boots or waders should be used on steep or icy stream banks and on rocky or smooth and slippery streambeds.

Personal floatation devices are required anytime the hydrographer is working in or over water bodies that are potentially dangerous. This includes wading in deep or swift streams as well as working from boats, bridges, dams, cableways or other structures over water.

5.4 **MEASUREMENT OF VELOCITY**

Current meters generally measure stream velocity at a point. One notable exception is the Acoustic Doppler Current Profiler (ADCP). This method will be discussed in a subsequent section of this Manual. The method of making discharge measurements at a cross section by using a current meter that measures point velocities requires determination of the mean velocity in each of the selected verticals. The mean velocity in a vertical is obtained from velocity observations at several points in that vertical. The mean velocity can be approximated by making a few velocity observations and using a known relation between those velocities and the mean in the vertical. See ISO 748 for detailed descriptions of velocity measurement methods. The various methods of measuring velocity are:

(a) Vertical-velocity curve;
(b) Two point;
(c) Six-tenths depth;
(d) Two-tenths depth;
(e) Three point;
(f) Surface and subsurface;
(g) Integration.

Less commonly used are the following multi-point methods of determining mean vertical velocity:

(a) Five point;
(b) Six point.

5.4.1 Vertical-velocity curve method

In the vertical-velocity curve method a series of velocity observations at points well distributed between the water surface and the streambed are made at each of the verticals. If there is considerable curvature in the lower part of the vertical-velocity curve it is advisable to space the observations more closely in that part of the depth. Normally the observations are taken at 0.1-depth increments between 0.1 and 0.9 of the depth. Observations are always taken at 0.2, 0.6 and 0.8 of the depth so that the results obtained by thevertical-velocity curve method may be compared with the commonly used methods of velocity observation.

The vertical-velocity curve for each vertical is based on observed velocities plotted against depth, as shown in Figure I.5.63. In order that vertical-velocity curves at different verticals may be readily compared it is customary to plot depths as proportional parts of the total depth. The mean velocity in the vertical is obtained by measuring the area between the curve and the ordinate axis with a planimeter, or by other means, and dividing the area by the length of the ordinate axis.

The vertical-velocity curve method is valuable in determining coefficients for application to the results obtained by other methods. It is not

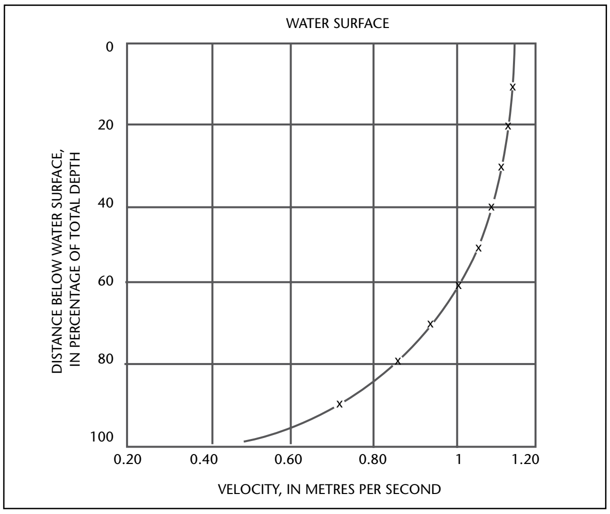


Figure I.5.63. Typical vertical-velocity curve

generally adapted to routine discharge measurements because of the extra time required to collect field data and to compute the mean velocity.

Intensive investigation of vertical velocity curves by Hulsing, Smith, and Cobb (1966) gave the following Table I.5.5 of average ordinates of the vertical velocity curve.

5.4.2 Two-point method

In the two-point method of measuring velocities, observations are made in each vertical at 0.2 and 0.8 of the depth below the surface. The average of these two observations is used as the mean velocity in the vertical. This method is based on many studies of actual observation and on mathematical theory. Experience has shown that this method gives more consistent and accurate results than any of the other methods except for the vertical-velocity curve method. Table I.5.5 indicates that, on average, the two-point method gives results that are within 1 per cent of the true mean velocity in the vertical if the vertical velocity curve is substantially parabolic in shape.

The two-point method is the one generally used for depths of 0.75 m or greater. The two-point method is not used at depths less than 0.75 m because the current meter would be too close to the water surface and to the streambed to give dependable results.

The vertical velocity curve will be distorted by overhanging vegetation in contact with the water or by submerged objects in close proximity to the vertical being measured. Where that occurs the two-point method will not give a reliable value of the mean velocity in the vertical and an additional velocity observation at 0.6 of the depth should be made. The three observed velocities should then be used in the three-point method. A rough test of whether or not the velocities at the 0.2 and 0.8 depths are sufficient for determining mean vertical velocity is given in the following criterion: the 0.2 depth velocity should be greater than the 0.8 depth velocity but less than twice as great.

5.4.3 Six-tenths-depth method

In the 0.6-depth method, an observation of velocity made at 0.6 of the depth below the water surface in the vertical is used as the mean velocity in the vertical. Actual observation and mathematical theory has shown that the 0.6-depth method gives reliable results (see Table I.5.5), and is generally used under the following conditions:

(a) Whenever the depth is between 0.1 m and 0.75 m;
(b) When large amounts of slush ice or debris make it impossible to observe the velocity accurately at the 0.2 depth. This condition prevents the use of the two-point method;
(c) When the meter is placed a distance above the sounding weight which makes it impossible to place the meter at the 0.8 depth. This circumstance prevents the use of the two-point method;
(d) When the stage in a stream is changing rapidly and a measurement must be made quickly.

5.4.4 Two-tenths-depth method

The two-tenths-depth method consists of observing the velocity at 0.2 of the depth below the surface and applying a coefficient to this observed velocity to obtain the mean in the vertical. It is used mainly during times of high water when the velocities are great, making it impossible to obtain soundings or to place the meter at the 0.8 or 0.6 depth.

Table I.5.5. Coefficients for standard vertical velocity curve

Ratio of observation depth to depth of water	*Ratio of point velocity to mean velocity in the vertical*
0.05	1.160
0.1	1.160
0.2	1.149
0.3	1.130
0.4	1.108
0.5	1.067
0.6	1.020
0.7	0.953
0.8	0.871
0.9	0.746
0.95	0.648

A standard cross section or a general knowledge of the cross section at a site is used to compute the 0.2 depth when it is impossible to obtain depth soundings. A sizeable error in an assumed 0.2 depth is not critical because the slope of the vertical-velocity curve at this point is usually nearly vertical. The 0.2 depth is also used in conjunction with the sonic sounder for flood measurements.

The discharge measurement is normally computed by using the 0.2-depth velocity observations without coefficients as though each were a mean in the vertical. The approximate discharge thus obtained divided by the area of the measuring section gives the weighted mean value of the 0.2-depth velocity. Studies of many measurements made by the two-point method show that for a given measuring section the relation between the mean 0.2-depth velocity and the true mean velocity either remains constant or varies uniformly with stage. In either circumstance this relation may be determined for a particular 0.2-depth measurement by re-computing measurements made at the site by the two-point method using only the 0.2-depth velocity observation as the mean in the vertical. The plotting of the true mean velocity versus the mean 0.2-depth velocity for each measurement will give a velocity-relation curve for use in adjusting the mean velocity for measurements made by the 0.2-depth method.

If there are not enough measurements by the two-point method available at a site to establish a velocity-relation curve, vertical-velocity curves are needed to establish a relationship between the mean velocity and the 0.2-depth velocity. The usual coefficient to adjust the 0.2-depth velocity to the mean velocity is about 0.87 (see Table I.5.5). The two-point method and the 0.6 depth method are preferred to the 0.2 depth method because of their greater accuracy.

5.4.5 Three-point method

The three-point method consists of observing the velocity at 0.2, 0.6 and 0.8 of the depth, thereby combining the two-point and 0.6-depth methods. The preferred method of computing the mean velocity is to average the 0.2 and 0.8-depth observations and then average this result with the 0.6-depth observation. However, when more weight to the 0.2 and 0.8-depth observations is desired, the arithmetical mean of the three observations may be used.

The three-point method is used when the velocities in the vertical are abnormally distributed. It is also used when the 0.8 depth observation is made where the velocity is seriously affected by friction or by turbulence produced by the streambed or an obstruction in the stream. The depths must be greater than 0.75 m before this method can be used.

5.4.6 Surface and subsurface methods

Surface and subsurface methods consist of observing the velocity at the water surface or some distance below the water surface. Surface measurements may be made with the optical current meter or by observing and timing surface floats. Experimental studies are being made with the use of stationary or mobile radar instrumentation to measure surface velocity. Subsurface measurements are made with a current meter at a distance of at least 0.6 m below the surface to avoid the effect of surface disturbances. Surface and subsurface measurements are used primarily for deep swift streams where it is impossible or dangerous to obtain depth and velocity soundings at the regular 0.2, 0.6 and 0.8-depths.

Coefficients are necessary to convert the surface or subsurface velocities to the mean velocity in the vertical. Vertical-velocity curves obtained at the particular site are the best method to compute these coefficients. However, the coefficients are generally difficult to determine reliably because they may vary with stage, depth and position in the measuring cross section. Experience has shown that the coefficients generally range from about 0.84 to about 0.90, depending on the shape of the vertical-velocity curve. The higher values are usually associated with smooth streambeds and normally shaped vertical-velocity curves whereas the lower values are associated with irregular streambeds and irregular vertical-velocity curves.

5.4.7 Integration method

In the integration method the meter is lowered in the vertical to the bed of the stream and then raised to the surface at a uniform rate. During this passage of the meter the total number of revolutions and the total elapsed time are used with the current meter rating table to obtain the mean velocity in the vertical. The integration method cannot be used with a vertical axis current meter because the vertical movement of the meter affects the motion of the rotor. However, the integration method is used to some degree in countries where horizontal axis meters are the standard current meters. The accuracy of the measurement is dependent on the skill of the hydrologist in maintaining a uniform rate of movement of the meter. A disadvantage of the method is the inability of the meter to measure stream-bed velocities because the meter cannot be placed that low. Coefficients smaller than unity are therefore required to correct the observed integrated velocity.

5.4.8 Five-point method

Velocity observations are made in each vertical at 0.2, 0.6 and 0.8 of the depth below the surface, and as close to the surface and to the stream-bed as practical. The criteria in ISO 748 (2007) for surface and bottom observations state that the horizontal axis of the current meter should not be situated at a distance less than one-and-a-half times the rotor height from the water surface, nor should it be situated at a distance less than three times the rotor height from the stream-bed. No part of the meter should break the surface of the water.

The velocity observations at the five meter positions are plotted in graphical form and the mean velocity in the vertical is determined by the use of a planimeter, as explained for the vertical velocity curve method. As an alternative the mean velocity may be computed from the equation:

$$V = 0.1\,(V_{surface} + 3\,V_{0.2} + 3\,V_{0.6} + 2\,V_{0.8} + V_{bed}) \quad (5.12)$$

which can be deduced from the logarithmic equation for the vertical velocity curve shown in Figure I.5.63 (Vanoni 1941).

5.4.9 Six-point method

The six-point method may be used in situations where the existence of a distorted vertical velocity distribution is known or suspected; for example, in the presence of aquatic growth, under ice cover or for special studies. Velocity observations are made in each vertical at 0.2, 0.4, 0.6 and 0.8 of the depth below the surface and also close to the surface and to the stream-bed. The criteria for surface and stream-bed observations are those given above for the five-point method.

The velocity observations at the six meter positions are plotted in graphical form and the mean velocity in the vertical is determined by planimetering the area bounded by the vertical velocity curve and the ordinate axis. Alternatively the mean velocity may be computed algebraically from the equation:

$$V = 0.1\,(V_{surface} + 2\,V_{0.2} + 2\,V_{0.4} + 2\,V_{0.6} + 2\,V_{0.8} + V_{bed}) \quad (5.13)$$

which can be deduced from the logarithmic equation for the vertical velocity curve shown in Figure I.5.63 (Vanoni 1941).

5.5 MEASUREMENT OF DEPTH

The water depth of a stream at a selected vertical can be measured in several ways depending on the type of measurement being made, the total depth of the stream and the velocity of the stream. Stream depth is usually measured by use of a wading rod, sounding lines and weights or a sonic sounder, as described in the following sections of this Manual.

5.5.1 Use of wading rod

A wading rod is used for measurement of stream depth when the water is shallow enough for making a wading measurement or when the measurement can be made from a low foot bridge or other structure which will support the streamgauger over the stream. Likewise, the wading rod is used for making measurements from ice cover when depths are not great. Wading rods can even be used from a boat if the water is not too deep. The top-setting wading rod can be used for depths up to 1.2 m. The round wading rod, which is assembled with 0.3 m sections, can be made up into any length but is generally only used for depths up to about 3 m. Velocity of flow is also a consideration because high velocity may not allow the hydrographer to keep a long wading rod in place.

Wading rods have a small foot on the bottom to allow the rod to be placed firmly on the streambed and will not sink into the streambed under most conditions. Where the stream bottom is soft it is sometimes difficult to keep the wading rod from sinking into the streambed. The weight of the rod and meter and the eroding power of the flowing water can cause the foot of the wading rod to sink. The hydrographer must use care in these conditions to be sure the measured water depth and the depth of the current meter placements are accurately based on the surface of the streambed. In some cases this may require that the wading rod be supported in some manner other than resting on the streambed.

When using a wading rod in streams with moderate to high velocity there will be a velocity-head build-up of water on the wading rod. The stream depth should be based on where the surface of the stream intersects the wading rod and not on the top of the velocity-head build-up.

5.5.2 Use of sounding lines and weights

Water depth is measured with sounding lines and weights when the depth is too great for use of a wading rod, and when measuring conditions require that measurements be made from a bridge, cableway or boat. This section will describe the measurement of depth using sounding reels and handlines. It also discusses the procedures used to correct observed depths when high velocity causes the weight and meter to drift downstream.

Use of sounding reels

When using one of the sounding reels described in a previous section of this report, a counter or dial is used to determine the length of cable that has been dispensed. Depths are measured to the nearest 0.03 m when using a sounding line and weight.

The size of the sounding weight used in current-meter measurements depends on the depth and velocity to be found in a cross section. A rule of thumb is that the size of the sounding weight (in kilograms) should be greater than 5 times the maximum product of velocity (in m/s) and depth (in m) in the cross section. If insufficient weight is used the sounding line will be dragged at an angle downstream. If debris or ice is flowing or if the stream is shallow and swift a heavier weight should be used. The rule is not rigid but does provide a starting point for deciding on the size of the weight necessary. Examine notes of previous measurements at a site to help determine the size of the weight needed at various stages.

Some sounding reels are equipped with a computing depth indicator or spiral. To use the computing spiral set the dial pointer at zero when the center of the current-meter rotor is at the water surface. Lower the sounding weight and meter until the weight touches the streambed and read the indicated depth. Add the distance that the meter is mounted above the bottom of the weight. For example, if a 15 kg 0.15 suspension is used and the dial pointer reads 5.64 m when the sounding weight touches the streambed, the depth would be 5.79 m (5.64 + 0.15). To move the meter to the 0.8-depth position merely raise the weight and the meter until the pointer is at the 5.79 m-mark on the graduated spiral, which will correspond to 4.63 m on the main dial (0.8 x 5.79). To set the meter at the 0.2-depth position raise the weight and meter until the pointer is at 1.16 m on the main dial (0.2 x 5.79).

Tags can be placed on the sounding line a known distance above the center of the meter cups as an aid in determining depth. The tags, which are usually streamers of different colored binding tape, are fastened to the sounding line by solder beads or by small cable clips. If debris or ice is flowing, the use of tags keeps the meter below the water surface and helps to prevent damage to the meter. Tags are used for determining depth in two ways:

(a) The preferred procedure. Set the tag at the water surface and then set the depth indicator to read the distance of that tag above the center of the meter cups. Continue as if the meter cups themselves had been set at the water surface. When the weight touches the streambed read the depth indicator and add the distance that the meter is above the bottom of the weight to obtain the total depth. The spiral indicator can be used, as described above, for setting the 0.8-depth meter position;
(b) The alternate method is sometimes used with handlines but can also be used with sounding reels. With the sounding weight on the streambed raise the weight until the first tag below the water surface appears at the surface. Determine the distance the weight was raised either by subtracting before and after readings of the depth indicator if a reel is being used or by using a tape or measuring stick if a handline is being used. The total stream depth is the sum of:
    (i) The distance the weight was raised to bring the tag to the water surface;
    (ii) The distance the tag is above the center of the meter cups and;
    (iii) The distance from the bottom of the weight to the center of the cups.

Use of a handline

When using a handline unwind enough of the lower steel cable from the handline reel to keep the reel out of water when the sounding weight is on the streambed at the deepest part of the cross section. If the bridge is high enough above the water surface, raise and lower the weight and meter by using the upper rubber-covered cable rather than by the lower steel cable.

The usual procedure for determining depths is to set the meter cups at the water surface and then lower the sounding weight to the streambed while measuring the amount of line required to reach the streambed. Measure along the rubber-covered service cord with a steel or metallic tape or a graduated rod to determine the distance the weight is lowered. This measured distance, plus the distance from the bottom of the sounding weight to the meter cups, is the depth of water. When the meter is set for the velocity observation, stand on the rubber-covered cable or tie it to the handrail to hold the meter in place. This frees the hands to record data.

Another method of determining depth when using a handline includes the use of tags set at a known distance above the meter. Lower the sounding weight to the streambed and then raise the weight until one of the tags is at the water surface. Measure along the rubber-covered service cord with a steel or metallic tape or a graduated rod to determine the distance the weight is raised. The total depth of water is then the summation of:

(a) The distance the particular tag is above the meter cups;
(b) The measured distance the meter and weight was raised and;
(c) The distance from the bottom of the weight to the meter cups.

Depth corrections for downstream drift of current meter and weight

Where it is possible to sound but the weight and meter drift downstream, the depths measured by the usual methods are too large. Figure I.5.64 graphically illustrates this condition. The correction for this error has two parts: the air correction and the wet-line correction. The air correction is shown as the distance *cd*. The wet-line correction is shown as the difference between the wet-line depth *de* and the vertical depth *dg*.

As shown in Figure I.5.64 the air correction, *cd*, depends on the vertical angle *P* and the distance *ab*. The correction is computed as follows:

$$
\begin{aligned}
& ab = ac \\
& \cos P = \frac{ab}{ad} = \frac{ab}{ac+cd} = \frac{ab}{ab+cd} \\
& ab + cd = \frac{ab}{\cos P} \qquad (5.14) \\
& cd = \frac{ab}{\cos P} - ab = ab\left[\frac{1}{\cos P} - 1\right]
\end{aligned}
$$

The air correction for selected vertical angles between 5 degrees and 35 degrees and vertical lengths between 1 and 20 m is shown in

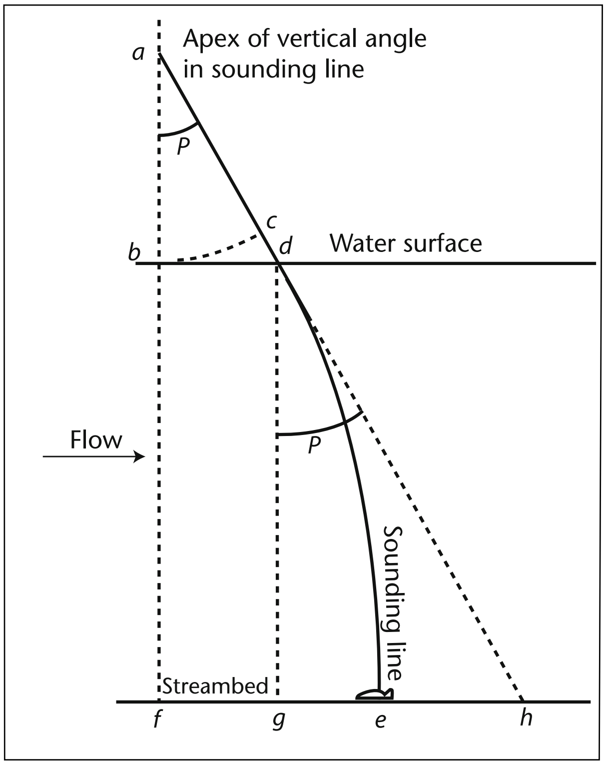


Figure I.5.64. Typical vertical-velocity curve

Table I.5.6. The correction is applied to the nearest 10 mm.

The air correction may be nearly eliminated by using tags at selected intervals on the sounding line and using the tags to refer to the water surface. This practice is almost equivalent to moving the reel to a position just above the water surface.

The correction for excess length of line below the water surface is obtained by using an elementary principle of mechanics. If a known horizontal force is applied to a weight suspended on a cord the cord takes a position of rest at some angle with the vertical. The tangent of the vertical angle of the cord is equal to the horizontal force divided by the vertical force owing to the weight. If several additional horizontal and vertical forces are applied to the cord the tangent of the angle in the cord above any point is equal to a summation of the horizontal forces below that point, divided by the summation of the vertical forces below the point.

The distribution of total horizontal drag on the sounding line is in accordance with the variation of velocity with depth. The excess in length of the curved line over the vertical depth is the sum of the products of each tenth of depth and the function *(1/cosP)–1* of the corresponding angles. The function is derived for each tenth of depth by means of the tangent relation of the forces acting below any point.

The wet-line correction for angles between 10 degrees and 35 degrees and wet-line depths between 1 and 20 m is shown in Table I.5.7. The correction is applied to the nearest 10 mm. The wet-line correction cannot be determined until the air correction has been deducted from the observed depth.

The following assumptions were used in deriving the wet-line correction table:

(a) The weight will go to the bottom despite the force of the current;
(b) The sounding is made when the weight is at the bottom but entirely supported by the line;
(c) Drag on the streamlined weight in the sounding position is neglected;
(d) The table is general and can be used for any size sounding weight or line, provided they are designed to offer little resistance to the current.

Wet-line corrections can also be computed with the following equation. This polynomial equation was derived by Ken Wahl (written communication, May 2000) from the data in Table I.5.7 and reproduces the table values to within a few mm. It can be used in a field computer to quickly and easily compute wet-line corrections. With additional programming, depths and depth settings can also be computed:

$$\begin{aligned} CORR_{wl} &= D_{wl} - D_v \\ &= \left(0.0004081 - 0.0001471 \times P + 0.00005731 \times P^2\right) \times D_{wl} \end{aligned} \tag{5.15}$$

where $CORR_{wl}$ = the correction, in m, to subtract from the wet-line depth to obtain the vertical depth, D_{wl} = the wet-line depth, in m, D_v = the vertical depth, in m, and P = the vertical angle, in degrees.

If the direction of flow is not perpendicular to the measuring section, the observed angle in the measuring line as indicated by the protractor will be less than the true angle of the line. The air correction and wet-line correction will then be too small. To correct for this the horizontal angle between the direction of flow and a perpendicular to the measuring section is measured by using a protractor or by determining the horizontal angle coefficient as described in a subsequent section of this report. The geometry of this condition is illustrated in Figure I.5.65.

Table I.5.6. Air correction table, giving differences, in metres, between vertical length and slant length of sounding line above water surface for selected vertical angles

Vertical length in metres	*Vertical angle of sounding line at protractor (degrees)*																											
	5	*8*	*10*	*11*	*12*	*13*	*14*	*15*	*16*	*17*	*18*	*19*	*20*	*21*	*22*	*23*	*24*	*25*	*26*	*27*	*28*	*29*	*30*	*31*	*32*	*33*	*34*	*35*
1	0.00	0.01	0.02	0.02	0.02	0.03	0.03	0.04	0.04	0.05	0.05	0.06	0.06	0.07	0.08	0.09	0.09	0.10	0.11	0.12	0.13	0.14	0.16	0.17	0.18	0.19	0.21	0.22
2	0.01	0.02	0.03	0.04	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.11	0.13	0.14	0.16	0.17	0.19	0.21	0.22	0.24	0.26	0.29	0.31	0.33	0.36	0.38	0.41	0.44
3	0.01	0.03	0.04	0.05	0.07	0.08	0.09	0.10	0.12	0.14	0.16	0.17	0.19	0.21	0.24	0.26	0.28	0.31	0.34	0.37	0.40	0.43	0.46	0.50	0.54	0.58	0.62	0.66
4	0.02	0.04	0.06	0.07	0.09	0.11	0.12	0.14	0.16	0.18	0.21	0.23	0.26	0.28	0.32	0.34	0.38	0.42	0.45	0.49	0.53	0.57	0.62	0.67	0.72	0.77	0.82	0.88
5	0.02	0.05	0.08	0.09	0.11	0.14	0.16	0.18	0.20	0.23	0.26	0.29	0.32	0.35	0.40	0.43	0.47	0.52	0.56	0.61	0.66	0.72	0.78	0.84	0.90	0.96	1.03	1.10
6	0.00	0.05	0.10	0.10	0.15	0.15	0.20	0.20	0.25	0.30	0.30	0.35	0.40	0.45	0.45	0.50	0.55	0.60	0.65	0.75	0.80	0.85	0.95	1.00	1.05	1.15	1.25	1.35
7	0.05	0.05	0.10	0.15	0.15	0.20	0.20	0.25	0.30	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.75	0.80	0.85	0.90	1.00	1.10	1.15	1.25	1.35	1.45	1.55
8	0.05	0.10	0.10	0.15	0.20	0.20	0.25	0.30	0.35	0.35	0.40	0.45	0.50	0.55	0.65	0.70	0.75	0.85	0.90	1.00	1.05	1.15	1.25	1.35	1.45	1.55	1.65	1.75
9	0.05	0.10	0.15	0.15	0.20	0.25	0.30	0.30	0.35	0.40	0.45	0.50	0.60	0.65	0.70	0.75	0.85	0.95	1.00	1.10	1.20	1.30	1.40	1.50	1.60	1.75	1.85	2.00
10	0.05	0.10	0.15	0.20	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.65	0.70	0.80	0.85	0.95	1.05	1.10	1.20	1.30	1.45	1.55	1.65	1.80	1.90	2.05	2.20
11	0.0	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.3	1.4	1.6	1.7	1.8	2.0	2.1	2.3	2.4
12	0.0	0.1	0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.8	1.0	1.0	1.1	1.2	1.3	1.5	1.6	1.7	1.9	2.0	2.2	2.3	2.5	2.6
13	0.1	0.1	0.2	0.2	0.3	0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.9	0.9	1.0	1.1	1.2	1.4	1.5	1.6	1.7	1.9	2.0	2.2	2.3	2.5	2.7	2.9
14	0.1	0.1	0.2	0.2	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.5	1.6	1.7	1.8	2.0	2.2	2.3	2.5	2.7	2.9	3.1
15	0.1	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.6	1.7	1.8	2.0	2.1	2.3	2.5	2.7	2.9	3.1	3.3
16	0.1	0.2	0.2	0.3	0.4	0.4	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.1	1.3	1.4	1.5	1.7	1.8	2.0	2.1	2.3	2.5	2.7	2.3	3.1	3.3	3.5
17	0.1	0.2	0.3	0.3	0.4	0.5	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.5	1.6	1.8	1.9	2.1	2.2	2.4	2.6	2.8	3.0	3.3	3.5	3.8
18	0.1	0.2	0.3	0.3	0.4	0.5	0.6	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.5	1.7	1.9	2.0	2.2	2.4	2.6	2.8	3.0	3.2	3.5	3.7	4.0
19	0.1	0.2	0.3	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.4	1.5	1.6	1.8	2.0	2.1	2.3	2.5	2.7	2.9	3.2	3.4	3.6	3.9	4.2
20	0.1	0.2	0.3	0.4	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.3	1.4	1.6	1.7	1.9	2.1	2.2	2.4	2.6	2.9	3.1	3.3	3.6	3.8	4.1	4.4

Table I.5.7. Wet-line table, giving difference, in metres, between wet-line length and vertical depth for selected vertical angles

Wet-line length in metres	*Vertical angle of sounding line at protractor (degrees)*																									
	10	*11*	*12*	*13*	*14*	*15*	*16*	*17*	*18*	*19*	*20*	*21*	*22*	*23*	*24*	*25*	*26*	*27*	*28*	*29*	*30*	*31*	*32*	*33*	*34*	*35*
1	0.01	0.01	0.02	0.02	0.02	0.03	0.03	0.03	0.03	0.04	0.04	0.04	0.05	0.06	0.06	0.06	0.07	0.07	0.08	0.08	0.09	0.10	0.10	0.11	0.12	0.12
2	0.02	0.02	0.03	0.03	0.04	0.04	0.05	0.05	0.06	0.06	0.07	0.08	0.08	0.09	0.09	0.10	0.11	0.11	0.12	0.13	0.14	0.15	0.16	0.17	0.18	0.19
3	0.03	0.04	0.04	0.05	0.05	0.06	0.06	0.07	0.08	0.08	0.09	0.10	0.11	0.12	0.13	0.14	0.15	0.16	0.18	0.19	0.20	0.21	0.23	0.24	0.26	0.27
4	0.03	0.04	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.11	0.12	0.13	0.14	0.16	0.17	0.18	0.19	0.21	0.22	0.24	0.25	0.27	0.28	0.30	0.32	0.34
5	0.04	0.05	0.06	0.06	0.07	0.08	0.09	0.10	0.11	0.13	0.14	0.15	0.17	0.18	0.20	0.22	0.24	0.26	0.27	0.29	0.31	0.33	0.35	0.38	0.40	0.42
6	0.05	0.05	0.05	0.05	0.10	0.10	0.10	0.10	0.15	0.15	0.15	0.15	0.20	0.20	0.25	0.25	0.25	0.30	0.30	0.35	0.35	0.40	0.40	0.45	0.45	0.50
7	0.05	0.05	0.05	0.10	0.10	0.10	0.10	0.15	0.15	0.15	0.20	0.20	0.25	0.25	0.25	0.30	0.30	0.35	0.35	0.40	0.40	0.45	0.50	0.50	0.55	0.55
8	0.05	0.05	0.10	0.10	0.10	0.10	0.15	0.15	0.15	0.20	0.20	0.25	0.25	0.30	0.30	0.35	0.35	0.40	0.40	0.45	0.45	0.50	0.55	0.55	0.60	0.65
9	0.05	0.10	0.10	0.10	0.10	0.15	0.15	0.20	0.20	0.20	0.25	0.25	0.30	0.30	0.35	0.35	0.40	0.45	0.45	0.50	0.55	0.55	0.60	0.65	0.70	0.70
10	0.05	0.10	0.10	0.10	0.15	0.15	0.15	0.20	0.20	0.25	0.25	0.30	0.30	0.35	0.35	0.40	0.45	0.45	0.50	0.55	0.60	0.60	0.65	0.70	0.75	0.80
11	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.8	0.8	0.9
12	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9
13	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0
14	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1
15	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2
16	0.1	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2
17	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3
18	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.4
19	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.1	1.2	1.2	1.3	1.4	1.5
20	0.1	0.2	0.2	0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.1	1.1	1.2	1.3	1.4	1.5	1.6

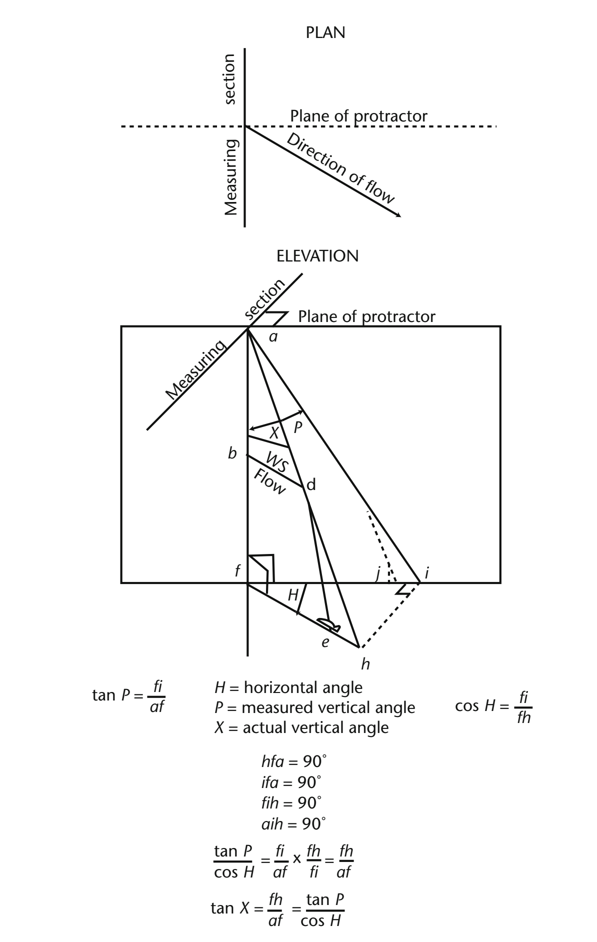


Figure I.5.65. Sketch of geometry of relation of true to measured vertical angle when flow direction is not normal to measurement section

If the horizontal angle of the direction of flow is called *H*, the observed vertical angle *P* and the true vertical angle *X*, the relation between the angles is expressed by the equation:

$$\tan X = \frac{\tan P}{\cos H} \tag{5.16}$$

Table I.5.8 gives the amounts in tenths of degrees to be added to observed vertical angles to obtain the true vertical angles for a range of horizontal angles between 8 and 28 degrees.

The conditions that cause error in sounding the depth also cause error in placing of the meter at selected depths. The correction tables are not strictly applicable to the problem of placing the meter because of the increased pressure placed on the sounding weight by higher velocities when it is raised from the streambed. A meter placed in deep, swift water by the ordinary methods for observations at selected percentages of the depth will be too high in the water. The use of Tables I.5.6, I.5.7 and I.5.8 will tend to eliminate this error in placement of the meter, and although not strictly applicable, their use for this purpose has become general.

For the 0.2-depth position, the curvature of the wet line is assumed to be negligible and the length of sounding line from the apex of the vertical angle to the weight is considered a straight line. The method used to place the meter at the 0.2-depth position is as follows:

(a) Compute the 0.2 value of the vertical depth;
(b) Lower the meter this depth into the water and read the vertical angle;
(c) Obtain the air correction from Table I.5.6. The vertical length used to obtain the air correction is the sum of (i) 0.2 of the vertical depth, (ii) the distance from the water surface to the apex of the angle, and (iii) the distance from the bottom of the weight to the meter;
(d) Let out an additional amount of line equal to the air correction;
(e) If the angle increases appreciably when the additional line is let out, let out more line until the total additional line, the angle and the vertical distance are in agreement with figures in the air-correction table.

To place the meter at the 0.8-depth position, a correction to the amount of line reeled in must be made for the difference, if any, between the air correction for the sounding position and that for the 0.8-depth position. This difference is designated as *m* in Table I.5.9. If the angle increases for the 0.8-depth position the meter must be lowered; if it decreases the meter must be raised.

For the 0.8-depth position of the meter the wet-line correction may require consideration if the depths are more than 12 *m* and if the change in vertical angle is more than 5 per cent. If the vertical angle remains the same or decreases the wet-line correction (Table I.5.8) for the 0.8-depth position is less than the wet-line correction for the sounding position by some difference designated as *n* in Table I.5.9. If the vertical angle increases the difference in correction *n* diminishes until the increase in angle is about 10 per cent; for greater increases in angle the difference between corrections increases also. Table I.5.9 summarizes the effect on air and wet-line corrections caused by raising the meter from the sounding position to the 0.8-depth position.

For slight changes in the vertical angle, because of the differences *m* and *n* in the air and wet-line corrections, the adjustments to the wet-line length of the 0.8-depth position are small and usually can be ignored. Table I.5.9 indicates that the meter may be placed a little too deep if the adjustments are not made. Because of this possibility, the wet-line depth instead of the vertical depth is sometimes used as the basis for computing the 0.8-depth position with no adjustments for the differences *m* and *n*.

Electronic field notebooks do not currently have algorithms for vertical angle corrections at the time of this writing. However, future models of EFNs will most probably be designed and programmed to make vertical angle corrections easily and automatically.

5.5.3 Use of sonic sounder

The sonic sounder, described in a previous section of this report, is used primarily for measurement of depth when making a moving boat measurement and is generally not used for measurements where sounding weights can be used. However, it can be used in swift, debris laden streams where it is difficult or dangerous to lower the sounding weight and meter into the water. The sonic sounder will record the depth when the weight is just below the water surface. For moving boat measurements the sonic sounder records a continuous trace of the

Table I.5.8. Degrees to be added to observed vertical angle, *P*, to obtain true vertical angle, *X*, when flow direction is not normal to measurement section

Observed vertical angle, *P*, in degrees	Horizontal angle, *H*, in degrees					
	8 cos = 0.99	12 cos = 0.98	16 cos = 0.96	20 cos = 0.94	24 cos = 0.91	28 cos = 0.88
8	0.1	0.2	0.3	0.5	0.8	1.0
12	0.1	0.3	0.5	0.8	1.1	1.5
16	0.1	0.4	0.6	1.0	1.4	2.0
20	0.2	0.4	0.7	1.2	1.7	2.4
24	0.2	0.5	0.8	1.4	2.0	2.8
28	0.2	0.5	1.0	1.5	2.2	3.0
32	0.2	0.6	1.0	1.6	2.4	3.3
36	0.2	0.6	1.1	1.7	2.5	3.4

Table I.5.9. Summary table for setting the meter at 0.8-depth position in deep, swift streams

Change in vertical angle	Air correction		Wet-line correction	
	Direction of change	Correction to meter position	Direction of change	Correction to meter position
None	None	None	Decrease	Raise meter the distance *n*
Decrease	Decrease	Raise meter the distance *m*	Decrease	Raise meter the distance *n*
Increase	Increase	Lower meter the distance *m*	Decrease, then increase	Raise meter the distance *n* unless the increase in angle is greater than about 10 per cent, then it is necessary to lower the meter the distance *n*

streambed on a chart, as described in Chapter 6. Details of its setup and use can be found in Smoot and Novak (1969).

5.6 PROCEDURE FOR CONVENTIONAL CURRENT METER MEASUREMENT OF DISCHARGE

Procedures for making most types of current-meter measurements are described in the following sections. These include the selection of a measuring section, laying out the stationing for sub-section verticals, depth measurements, velocity measurements, direction of flow measurements and recording of field notes. Additional details that pertain to specific types of measurements, such as wading, cableway, bridge, boat and ice, are described in subsequent sections. Special procedures such as networks of current meters, measurement of deep, swift streams and measurements during rapidly changing stage are also described. Procedures for making measurements using the moving boat method, the ADCP moving boat method and the electromatic method are described in Chapter 6.

The usual procedure, after selecting and laying out the section, is to measure and record at each vertical the: (a) distance from the initial point, (b) depth, (c) meter position, (d) number of revolutions, (e) time interval, and (f) horizontal angle of flow. The starting point can be either bank. The edge of water, which may have a depth of zero, is considered the first vertical. The hydrographer should move to each of the verticals in succession and repeat the procedure until the measurement is completed at the opposite bank.

5.6.1 Site selection

The first step in making a current-meter measurement is to select a measurement cross section of desirable qualities. If the stream cannot be waded and high-water measurements are made from a bridge or cableway the hydrographer has little or no choice with regard to selection of a measurement cross section. If the stream can be waded or the measurement can be made from a boat, the hydrographer looks for a cross section with the following characteristics, as described in ISO 748 (2007):

(a) The channel at the measuring site should be straight and of uniform cross section and slope to minimise abnormal velocity distribution;
(b) Flow directions for all points on any vertical across the width should be parallel to one another and at right angles to the measurement section;
(c) A stable streambed free of large rocks, weeds and obstructions that would create eddies, slack water and turbulence;
(d) The curves of the distribution of velocities should be regular in the vertical and horizontal planes of measurement;
(e) Conditions at the section and in its vicinity should also be such as to preclude changes taking place in the velocity distribution during the period of measurement;
(f) Sites displaying vortices, reverse flow or dead water should be avoided;
(g) The measurement section should be clearly visible across its width and unobstructed by trees, aquatic growth or other obstacles;
(h) The depth of water at the section should be sufficient at all stages to provide for the effective immersion of the current meter or float, whichever is to be used;
(i) The site should be easily accessible at all times with all necessary measurement equipment;
(j) The section should be sited away from pumps, sluices and outfalls, if their operation during a measurement is likely to create flow conditions inconsistent with the natural stage-discharge relationship for the station;
(k) Sites should be avoided where there is converging or diverging flow;
(l) In those instances where it is necessary to make measurements in the vicinity of a bridge it is preferable that the measuring section be upstream of the bridge. However, in special cases and where accumulation of ice, logs or debris is liable to occur it is acceptable that the measuring site be downstream of the bridge. Particular care should be taken in determining the velocity distribution when bridge apertures are surcharged;
(m) It may, at certain states of river flow or level, prove necessary to carry out current meter measurements on sections other than that selected for the station. This is acceptable if there are no substantial ungauged losses or gains to the river in the intervening reach and so long as all flow measurements are related to levels recorded at the principal reference section;
(n) A measurement section that is relatively close to the gauging station control to avoid the effect of tributary inflow between the measurement section and the control, and to avoid the effect of channel storage between the measurement section and the control during periods of changing stage.

It is usually not possible to satisfy all of these conditions. Select the best possible reach using these criteria and then select a cross section.

5.6.2 Layout and stationing of partial sections and verticals

After the cross section has been selected determine the width of the stream. String a tag line or measuring tape for measurements made by wading, from a boat, from ice cover or from an unmarked bridge. Except for bridges, string the line at right angles to the direction of flow to avoid horizontal angles in the cross section. For cableway or bridge measurements use the graduations painted on the cable or bridge rail. Next determine the spacing of the verticals, generally using about 25 to 30 partial sections for streams that are greater than about 10 metres wide. For streams less than about 10 metres fewer verticals can be used. When the cross section is smooth and with even velocity distribution, fewer partial sections may be used. Space the partial sections so that no partial section has more than 10 per cent of the total discharge. The ideal measurement is one in which no partial section has more than 5 per cent of the total discharge, but this is very seldom accomplished when less than 25 partial sections are used. The discharge measurement shown in Figure I.5.2 had 6.2 per cent of the total discharge in the partial section with the greatest discharge. Equal widths of partial sections across the entire cross section are not recommended unless the discharge is very evenly distributed. Make the width of the partial sections less as depths and velocities become greater. Usually an approximate total discharge can be obtained from the stage-discharge curve. Space the verticals so the discharge in each partial section is about 5 per cent of the expected total discharge from the rating curve. Additional guidance regarding the measurement procedure can be found in ISO 748 (2007).

When using an electronic field notebook, such as the Aquacalc or DMX, the expected total discharge can be entered prior to starting the discharge measurement. Then during the measurement a warning message will be displayed if a partial discharge exceeds 10 per cent of the expected total discharge.

5.6.3 Depth measurements

The first measurement made at a vertical is the stream depth. Depth should be measured using the proper equipment and procedures that apply to the type of measurement being made (that is wading, bridge, cableway, boat or ice). Details of measuring depth using various equipments and under different flow conditions are described in previous sections of this Manual.

5.6.4 Velocity measurements

After the depth at a vertical is measured and recorded determine the method of velocity measurement. Normally the two-point method or the 0.6-depth method is used. Compute the setting of the meter for the particular method to be used at that depth. For the top-setting wading rod or the spiral-computing dial, some meter settings are self-computing. Record the meter position as 0.8, 0.6, 0.2 or some other setting that might be used. After the meter is placed at the proper depth let it adjust to the current before starting the velocity observation. The time required for such adjustment is usually only a few seconds if the velocities are greater than about 0.3 m/s but for lower velocities, particularly if the current meter is suspended by a cable, a longer period is needed. After the meter has become adjusted to the current, count the number of revolutions made by the rotor for a period of 40 to 70 seconds. Start the stopwatch simultaneously with the first signal or click, counting zero, not one. End the count on a convenient number given in the meter rating table column heading. Stop the stopwatch on that count and read the time to the nearest second or to the nearest even second if the hand is on a half-second mark. Record the number of revolutions and the time interval. If the velocity is to be observed at more than one point in the vertical, determine the meter setting for the additional observation, set the meter to that depth, time the revolutions and record the data.

When using a Current Meter Digitizer (CMD) or an Electronic Field Notebook (EFN) the same basic procedure for setting the meter and providing time for the meter to stabilize must be observed. However, with these instruments the counting and timing of the rotor revolutions are performed automatically. The number of revolutions, time and velocity displayed by the CMD must be transferred manually to paper field notes, whereas these data are electronically recorded by EFNs. Care should be observed when using any of these automatic meter counting devices to be sure that multiple counts are not occurring during measurement of slow velocity. This can sometimes be determined by visually observing the rotation of the rotor while simultaneously listening to the audible clicks or beeps from the counting device.

5.6.5 Direction of flow measurements

Consideration must be given to the direction of flow because the component of velocity normal to the measurement section is that which must be determined. The discussion that follows concerns currents that approach the measurement section obliquely, at angle alpha, as shown in Figure I.5.10.

If, in a wading measurement, the meter used is a horizontal-axis meter with a component propeller, such as the Ott meter, the propeller should be pointed upstream at right angles to the cross section, but only if alpha is less than 45 degrees. This will register the desired component of velocity normal to the cross section when alpha is less than 45 degrees. The same procedure should be used for an electromagnetic component meter. These meters also measure the component of velocity normal to the measuring section. For either type of meter, if alpha is greater than 45 degrees, the component meter should be pointed directly into the current and the horizontal angle correction should be applied as described in the following paragraphs.

Other meters on a wading-rod suspension, such as the Price current meter, should be pointed into the current. Any meter on a cable suspension will automatically point into the current because of the effect of the meter vanes. When the meter is pointed into an oblique current the measured velocity must be multiplied by the cosine of the angle (alpha) between the current and a perpendicular to the measurement section to obtain the desired normal component of the velocity.

In the United States, use is made of the field note sheet to measure the cosine of the angle. The note sheet has a point of origin (o) printed on the left margin and cosine values on the right margin. The cosine of the angle of the current is measured by holding the note sheet in a horizontal position with the point of origin on the tag line, bridge rail, cable rail or any other feature parallel to the cross section, as shown in Figure I.5.66. With the long side of the note sheet parallel to the direction of flow, the tag line or bridge rail will intersect the value of cosine alpha on the top, bottom, or right edge of the note sheet. The direction of the current will be apparent from the direction of movement of floating particles. If the water is clear of floating material, small bits of floating material are thrown into the stream and the edge of the note sheet is aligned parallel to the direction of movement. If no such material is available the inelegant but time-honoured method of spitting into the stream is used to obtain an indicator of the direction of flow. The position of the current meter may also be used if it can be seen below the water surface. The measured velocity is multiplied by the cosine of the angle to determine the velocity component normal to the measurement section.

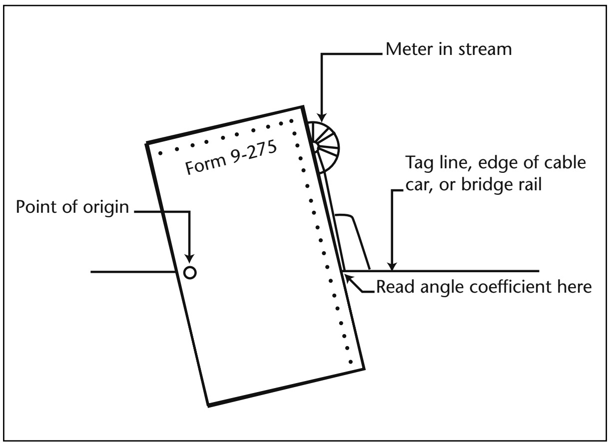


Figure I.5.66. Measurement of horizontal angle with measurement note sheet

The direction of flow, as observed on the surface of the stream, may not always be a reliable indication of the direction of flow at some distance below the surface. For instance, when measuring a stream influenced by tidal fluctuations it is possible to have flow moving downstream near the surface of the stream and flow moving upstream near the bottom of the stream. Therefore, whenever it is suspected that the direction of flow is variable at different depths, other means than given in the above paragraphs must be used to determine the direction of flow. One such method is to use a rigid rod or pole with a vane attached to the bottom and an indicator parallel to the vane attached to the top. Another method is to use a sounding weight with a compass and remote read-out as described in the equipment section of this Manual. If the variation of the direction of flow in the vertical is not great then an average value of the cosine of the angle may be used for computing the component mean velocity for the vertical. However, if the variation is considerable then it may be necessary to sub-divide the vertical and make separate computations for each subdivision. This may require additional measurements of velocity in the vertical.

5.6.6 Recording field notes

Field notes for a discharge measurement may be recorded either on standard paper note sheets, or by using an Electronic Field Notebook (EFN).

These two methods are described in the following paragraphs.

Standard paper note keeping

Paper note sheets, as shown in Figure I.5.2 (computation sheet) and Figure I.5.3 (summary sheet), are the traditional way to record the field observations for a discharge measurement. For each measurement record the following information, at a minimum, on the front sheet of the measurement notes:

(a) Stream name, location and downstream identification number, to correctly identify an established gauging station. For a miscellaneous measurement record the stream name and exact location of site;
(b) Date, party, meter number and type of meter suspension;
(c) Time measurement was started using 24 hour clock time. Also record the time zone;
(d) Type of measurement and location of measurement relative to the gauge;
(e) Control conditions;
(f) Gauge heights and corresponding times;
(g) Water temperature;
(h) Pertinent information regarding the accuracy of the discharge measurement and conditions which might affect the stage-discharge relation;
(i) Other items on the front sheet are usually filled in after the measurement is completed and computed.

On the inside notes identify the measurement starting point by either LEW or REW (left edge of water or right edge of water, respectively, when facing downstream) and record the time that the measurement is started. If a significant change in stage is expected during the measurement, periodically record the time for intermediate verticals during the course of the measurement. This time should, if possible, be synchronized with the recording interval of the digital recorder or data logger. Intermediate times are important because if there is any appreciable change in stage during the measurement. These recorded times are used to determine intermediate gauge heights which are then used to compute a weighted mean gauge height for the measurement, as described in a subsequent section. When the measurement is completed record the time and the bank of the stream (LEW or REW) where the section ends.

Begin the measurement by recording the distance from the initial point to the edge of the water. Measure and record the depth and velocity at the edge of water. Proceed across the measurement section by measuring and recording the distance of each vertical from the initial point, the depth at the vertical, the observation depths as 0.6, 0.2, 0.8, the revolutions and time for each velocity observation and the horizontal angle coefficient if different than 1.00.

Complete all computations required for the inside notes to determine the total width, area, and discharge. Transfer these values to the front sheet and complete other items on the measurement front sheet. It is generally expected that the measurement computations will be made and the note sheets completed before the hydrographer leaves the gauging station.

Erasures of original field data are not allowed. This includes items such as gauge readings, distances, depths, meter revolutions, times, horizontal angle coefficients and other field measurements that cannot be repeated. If a variable is re-measured and it is necessary to change the originally recorded value of that variable it should be marked through and the new measurement should be recorded above or adjacent to the original. The original measurement should remain legible even though it is marked out. However, it is permissible to erase computed values such as velocities, areas, widths and discharges.

Electronic field notebooks

If an electronic field notebook is used for recording field measurements and notes the same basic information is required as for paper field notes, except that much of the information is entered using the keyboard of the electronic notebook. Some data, such as meter revolutions and time, are entered automatically and most of the measurement computations are made automatically. However, the electronic field notebooks currently available do not have the capability to receive all field data that may be required. Therefore, it is necessary to record some information on paper field notes.

Each of the electronic field notebooks has different operating characteristics. Users must become familiar with how data are entered in each type to become proficient in using this type of discharge measurement recording system. User's manuals are provided with each electronic notebook. An electronic field notebook can be an efficient method to record, compute and store field notes. It provides numerous quality control features and should eliminate most arithmetic errors.

5.6.7 Current-meter measurements by wading

Current-meter measurements by wading are preferred, if conditions permit. Wading measurements offer the advantage over measurements from bridges and cableways in that it is usually possible to select the best of several available cross sections for the measurement. Figure I.5.67 shows a wading measurement being made with a top-setting rod.

Propeller current meters of diameter 100 mm or more should not be used for depths less than 0.30 m because the registration of the meter is affected by its proximity to the bed. If a Price current meter is used, use Table I.5.10 to determine the type of meter and velocity method to use for wading measurements for various depths.

If a type AA meter is being used in a cross section where most of the depths are greater than 0.5 m, do not change to the pygmy meter for a few depths less than 0.5 m or vica versa. The Price AA meter is not recommended for depths of 0.3 m or less because the registration of the meter is affected by its proximity to the water surface and to the streambed. However, it can be used at depths as shallow as 0.15 m to avoid changing meters if only a few verticals of this depth are required. Do not use the type AA meter or the pygmy meter in velocities less than 0.06 m/s unless absolutely necessary.

When natural conditions for measuring are in the range considered undependable, modify the measuring cross section, if practical, to provide acceptable conditions. Often it is possible to build dikes to cut off dead water and shallow flows in a cross section, or to improve the cross section by removing the rocks and debris within the section and from the reach of stream immediately upstream from it. After modifying a cross section allow the flow to stabilize before starting the discharge measurement.

Table I.5.10. Current meter and velocity-measurement method for various depths

Depth, in m	*Current meter*	*Velocity method*
0.75 and greater	Price Type AA	0.2 and 0.8
0.5 – 0.75	Price Type AA	0.6
0.1 – 0.5	Price Pygmy	0.6
0.5 and greater	Price Pygmy	0.2 and 0.8

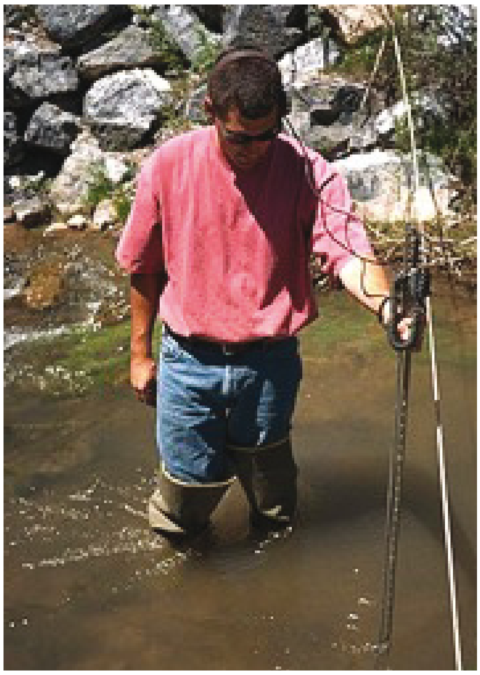


Figure I.5.67. Wading measurement using a top-setting rod

Stand in a position that least affects the velocity of the water passing the current meter. This position is usually obtained by facing the bank with the water flowing against the side of the leg. Holding the wading rod at the tag line stand from 0.03 to 0.08 m downstream from the tag line and 0.5 m or more from the wading rod. Avoid standing in the water if feet and legs would occupy a considerable percentage of the cross section of a narrow stream. In small streams where the width permits stand on a plank or other support above the water rather than in the water.

Keep the wading rod in a vertical position. Vertical axis current meters should be held parallel to the direction of flow while observing the velocity. Horizontal axis and electromagnetic current meters should be held perpendicular to the cross section. If the flow is not at right angles to the tag line, measure the angle coefficient carefully.

During measurements of streams with shifting beds, the scoured depressions left by the hydrographer's feet can affect soundings or velocities. Generally, place the meter ahead of and upstream from the feet. Record an accurate description of streambed and water-surface configuration each time a discharge measurement is made in a sand-channel stream.

For discharge measurements of flow too small to measure with a current meter use a volumetric method, Parshall flume or weir plate. Those methods are described in subsequent sections of this Manual.

5.6.8 Current-meter measurements from cableways

A horizontal axis or vertical axis current meter is generally used in conjunction with sounding weights and a sounding reel, as described in previous sections, when making discharge measurements from a cableway. Stationing for width measurements is usually determined from marks painted on the cableway. The velocity is measured by setting the meter at the proper position in the vertical, as indicated in Table I.5.11. Table I.5.11 is designed so that no velocity observations will be made with the meter closer than 0.15 m to the water surface. In the zone from the water surface to a depth of 0.15 m the current meter is known to give erratic results.

One problem found while observing velocities from a cableway is that the movement of the cable car from one station to the next makes the car oscillate for a short time after coming to a stop. Wait until this oscillation has dampened to a negligible amount before counting the revolutions.

By using tags, the meter can be kept under water at all times to prevent freezing the meter in cold air. Tags are also used in measurements of deep, swift streams. See the section of this report on "Measurement of depth".

If large amounts of debris are flowing in the stream, raise the meter up to the cable car several times during the measurement to be certain the pivot and rotor of the meter are free of debris. However, keep the meter in the water during the measurement if the air temperature is considerably below freezing.

During floods there is always a danger of catching a submerged or floating object, such as a tree or log, which can endanger the sounding equipment, meter and most importantly the hydrographer. Always be sure that the sounding cable has been installed on the sounding reel according to the procedure described in the section of "Sounding cables" and according to the breaking loads specified in Table I.5.4. This assures that the sounding cable will break when it reaches it's end, thereby preventing a potentially serious accident where the cable car and hydrographer could be spilled into the stream. Also, for added safety, always carry a pair of lineman's side-cutter pliers when making measurements from a cableway. If the sounding cable becomes hopelessly hung and does not break, as it should, cut the sounding line to insure safety. Sometimes the cable car can be pulled to the edge of the water and the debris can be released.

When measurements are made from cableways where the stream is deep and swift, measure the angle that the meter suspension cable makes with the vertical due to the drag. The vertical angle, measured by protractor, is needed to correct the soundings to obtain the actual vertical depth, as described in the section on "Depth corrections for downstream drift of current meter and weight".

5.6.9 Current-meter measurements from bridges

When a stream cannot be waded bridges may be used to obtain current-meter measurements. Many measuring sections under bridges are satisfactory for current-meter measurements but cableway sections are usually better. No set rule can be given for choosing between the upstream or downstream side of the bridge when making a discharge measurement.

The advantages of using the upstream side of the bridge are:

(a) Hydraulic characteristics at the upstream side of bridge openings usually are more favorable. Flow is usually more smooth and there is less turbulence than at the downstream side of the bridge;
(b) Approaching drift can be seen and be more easily avoided;
(c) The streambed at the upstream side of the bridge is not likely to scour as badly as at the downstream side.

The advantages of using the downstream side of the bridge are:

(a) Vertical angles are more easily measured because the sounding line will move away from the bridge;
(b) The flow lines of the stream may be straightened out by passing through a bridge opening with piers.

The choice of using the upstream side or the downstream side of a bridge for a current-meter measurement should be decided individually for each bridge. Consideration should be given to the factors mentioned above and the physical conditions at the bridge, such as location of the walkway, traffic hazards and accumulation of trash on piles and piers.

Use either a handline, a sounding reel supported by a bridge board or a portable crane to suspend the current meter and sounding weight from bridges. Depth measurements should be made as described in the section Measurement of depth. Measure the velocity by setting the meter at the position in the vertical as indicated in Table I.5.11. Keep equipment several feet from piers and abutments if velocities are high. Estimate the depth and velocity next to the pier or abutment on the basis of the observations at the nearest vertical.

If there are piers in the cross section it is usually necessary to use more than 25-30 partial sections to get results as reliable as those from a similar section without piers. Piers will often cause horizontal angles that must be carefully measured. Piers also cause rapid changes in the horizontal velocity distribution in the section.

The bridge pier might be excluded from the area of the measurement cross section depending primarily on the relative locations of the measurement section and the end of the pier. If measurements are made from the upstream side of the bridge it is the relative location of the upstream end (nose) of the pier that is relevant. For measurements made from the downstream side it is the location of the downstream end (tail) of the pier that is relevant. If any part of the pier extends into the measurement cross section the area of the pier is excluded. However, bridges quite commonly have cantilevered walkways from which discharge measurements are made. In that case the measurement cross section lies beyond the end of the pier (upstream from the nose or downstream from the tail, depending on which side of the bridge is used). In that situation it is the position and direction of the streamlines that determines whether or not the pier area is to be excluded. The hydrographer, if he or she had not previously noted the stationing of the sides of the pier when projected to the measurement cross section, does so now. If there is negligible or no downstream flow in that width interval (pier subsection) then the pier is excluded. If only stagnation and/or eddying exists upstream from the nose or downstream from the tail, whichever is relevant, the area of the pier is excluded. If there is significant downstream flow in the pier subsection the area of the pier is included in the area of the measurement cross section. The horizontal angles of the streamlines in and near the pier subsection will usually be quite large in that circumstance.

Footbridges are sometimes used for measuring canals, tailraces and small streams. Rod suspension can be used from many footbridges. The procedure for determining depth in low velocities is the same as for wading measurements. For higher velocities obtain the depth by the difference in readings at an index point on the bridge when the base plate of the rod is at the water surface and on the streambed. Measuring the depth in this manner will eliminate errors caused by the water piling up on the upstream face of the rod. Handlines, bridge cranes and bridge boards are also used from footbridges.

The handline can be disconnected from the headphone wire and passed around a truss member with the sounding weight on the bottom. This eliminates the need for raising the weight and meter to the bridge each time a move is made from one vertical to another and is the principal advantage of a handline.

Safety is a primary consideration when making discharge measurements from bridges. High-speed

Table I.5.11. Veclocity-measurement method for various suspensions and depths

Suspension[1]	*Minimum depth, in m*	
	0.6 method	*0.2 and 0.8 method*
7 kg 0.15 m, 15 kg 0.15 m	0.37	0.76
23 kg 0.17 m	0.43	0.86
23 kg 0.27 m	0.67	1.37
34 kg 0.30 m, 45 kg 0.30 m, 68 kg 0.30 m	0.76	1.52
90 kg 0.45 m, 136 kg 0.45 m	1.16[2]	2.29

1 Suspension shown indicates the size of the sounding weight and the distance from the bottom of the weight to the current meter axis. Thus 23 kg 0.27 m refers to a 23 kg weight, the suspension for which puts the meter 0.27 m above the bottom of the weight.

2 Use 0.2 method for depths 0.75 to 1 m with appropriate coefficient (estimated 0.88).

traffic can present a major safety hazard and it is no longer permissible to make discharge measurements from some bridges, such as on Interstate routes in the United States. All safety precautions, such as the use of cones, traffic signs and flag-persons should be employed.

The same safety precautions regarding the snagging of debris, such as floating or submerged trees or logs, should be observed, as described above for cableways.

5.6.10 **Current-meter measurements from ice cover**

Discharge measurements under ice cover, as shown in Figure I.5.68, are made under the most severe conditions but are extremely important because a large part of the discharge record during a winter period may depend on one or two measurements.

Select the possible locations of the cross section to be used for a measurement from ice cover during the open-water season when channel conditions can be evaluated. Commonly the most desirable measurement section will be just upstream from a riffle because slush ice that collects under the ice cover is usually thickest at the upstream end of the pools created by riffles.

Figure I.5.68. Ice rod being used to support current meter for a discharge measurement (left); Price AA current meter with polymer rotor used for ice measurements (right).

The equipment used for cutting or drilling the holes in the ice is described in a previous section of this Manual.

Never underestimate the danger of working on ice-covered streams. When crossing, test the strength of the ice with solid blows using a sharp ice chisel. Ice thickness may be irregular, especially late in the season when a thick snow cover may act as an insulator. Water just above freezing can slowly melt the underside of the ice, creating thin spots. Ice bridged above the water may be weak even though it is thick.

Cut the first three holes in the selected cross section at the quarter points to detect the presence of slush ice or poor distribution of the flow in the measuring section. If poor conditions are found investigate other sections to find one that is free of slush ice and that has good distribution of flow. After finding a suitable cross section make at least 20 holes in the ice for a current-meter measurement. Space the holes so that no partial section has more than 10 per cent of the total discharge in it. On narrow streams it may be simpler to remove all of the ice in the cross section.

The effective depth of the water, as shown in Figure I.5.69, is the total depth of water minus the distance from the water surface to the bottom of the ice. The vertical pulsation of water in the holes in the ice sometimes causes difficulty in determining the depths. The total depth of water is usually measured with an ice rod or with a sounding weight and reel, depending on the depth.

Measure the distance from the water surface to the bottom of the ice with an ice-measuring stick. If there is slush under the solid ice at a hole, the

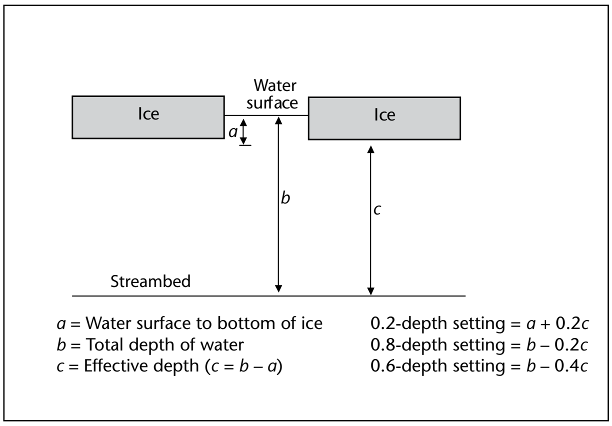


Figure I.5.69. Method of computing meter settings for measurements under ice cover

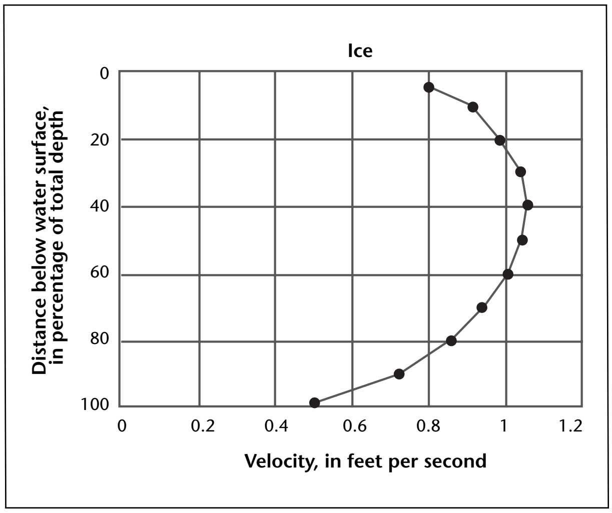
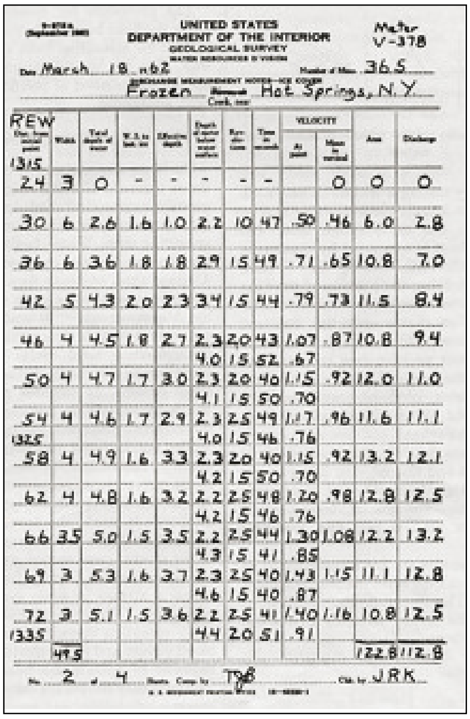


Figure I.5.70. Typical vertical-velocity curve under ice cover



UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

Meter V-378

Date March 18 1962 Number of Mea. 36.5

DISCHARGE MEASUREMENT NOTES—ICE COVER

Frozen Creek near Hot Springs, N.Y.

Dist. from initial point	Width	Total depth of water	W.S. to bot. ice	Effective depth	Depth of obser. below water surface	Revo-lutions	Time in seconds	Velocity At point	Velocity Mean in vertical	Area	Discharge
REW 1315											
24	3	0	-	-	-	-	-		0	0	0
30	6	2.6	1.6	1.0	2.2	10	47	.50	.46	6.0	2.8
36	6	3.6	1.8	1.8	2.9	15	49	.71	.65	10.8	7.0
42	5	4.3	2.0	2.3	3.4	15	44	.79	.73	11.5	8.4
46	4	4.5	1.8	2.7	2.3	20	43	1.07	.87	10.8	9.4
					4.0	15	52	.67			
50	4	4.7	1.7	3.0	2.3	20	40	1.15	.92	12.0	11.0
					4.1	15	50	.70			
54	4	4.6	1.7	2.9	2.3	25	49	1.17	.96	11.6	11.1
1325					4.0	15	46	.76			
58	4	4.9	1.6	3.3	2.3	20	40	1.15	.92	13.2	12.1
					4.2	15	50	.70			
62	4	4.8	1.6	3.2	2.2	25	48	1.20	.98	12.8	12.5
					4.2	15	46	.76			
66	3.5	5.0	1.5	3.5	2.2	25	44	1.30	1.08	12.2	13.2
					4.3	15	41	.85			
69	3	5.3	1.6	3.7	2.3	25	40	1.43	1.15	11.1	12.8
					4.6	15	40	.87			
72	3	5.1	1.5	3.6	2.2	25	41	1.40	1.16	10.8	12.5
1335					4.4	20	51	.91			
	47.5									122.8	112.8

No. 2 of 4 Sheets. Comp by TJB Ch. by JRK

Figure I.5.71. Part of note sheet for discharge measurement under ice cover

ice-measuring stick is not used. To find the depth at which the slush ice ends, suspend the current meter below the slush ice with the meter rotor turning freely. Raise the meter slowly until the rotor stops. This point is used as the depth of the interface between water and slush. After the effective depth of the water has been determined, compute the proper position of the meter in the vertical as shown in Figure I.5.69.

The Price winter WSC current meter yoke, with a polymer rotor, is recommended for use under ice cover when slush ice is present because the cups are solid and cannot become filled with slush ice as the cups of the regular Price meter often does. For situations where slush ice is not present, the Price winter WSC current meter yoke with regular Price metal cups is recommended. The old style vane ice meter is no longer recommended, primarily because of its poor performance in slow velocities.

The velocity distribution under ice cover, when the water is in contact with the underside of the ice, is similar to that in a pipe with a lower velocity nearer the underside of the ice. This is illustrated in Figure I.5.70. The 0.2 and 0.8-depth method is recommended for effective depths of 0.75 m or greater and the 0.6-depth method is recommended for effective depths of less than 0.75 m. It is recommended that two vertical-velocity curves be defined when ice measurements are made to determine whether any coefficients are necessary to convert the velocity obtained by the 0.2 and 0.8-depth method or the 0.6-depth method to the mean velocity. Normally the average of the velocities obtained by the 0.2 and 0.8-depth method gives the mean velocity but a coefficient of about 0.92 usually is applicable to the velocity obtained by the 0.6-depth method.

When measuring the velocity keep the meter as far upstream as possible to avoid any effect that the vertical pulsation of water in the hole might have on the meter. Eliminate as much as possible the exposure of the meter to the cold air during the measurement. The meter must be free of ice when the velocity is being observed. If there is partial ice cover at a cross section use the procedure described above where there is ice cover and use open-water methods elsewhere.

A sample sheet of discharge-measurement notes under ice cover is shown in Figure I.5.71. In this measurement the vertical-velocity curves indicate that the 0.2 and 0.8-depth method gives the mean velocity and that the 0.6-depth method requires a coefficient of 0.92.

5.6.11 Current-meter measurements from stationary boats

Discharge measurements are made from boats where no cableways or suitable bridges are available and where the stream is too deep to wade. Personal safety is the limiting factor in the use of boats on streams having high velocity.

For boat measurements, the cross section should be selected so that it has attributes similar to those described in a previous section "Selection of site", except for those listed in item 3 concerning depth and velocity. Depth is of no consideration in a boat measurement because if the stream is too shallow to float a boat the stream can usually be waded. However, velocity in the measurement section is an important concern. If velocities are too slow, meter registration may be affected by an oscillatory movement of the boat in which the boat (even though fastened to a tag line) moves upstream and downstream as a result of wind action. Also, vertical movement of the boat as a result of wave action may affect a vertical-axis current meter. If velocities are too fast it becomes difficult to string a tag line across the stream.

If a tagline is feasible for use in making a boat measurement, string the tag line at the measuring section by unreeling the line as the boat moves across the stream. Some tag-line reels are equipped with brakes to control the line tension while unreeling. After a tag line without a brake has been stretched across the stream, take up the slack by means of a block and tackle attached to the reel and to an anchored support on the bank. If there is traffic on the river one person must be stationed on the bank to lower and raise the tag line to allow the river traffic to pass. Place streamers on the tag line so that it is visible to boat pilots. If there is a continual flow of traffic on the river, or if the width of the river is too great to stretch a tag line, other means will be needed to position the boat. Night measurements by boat are not recommended because of the safety aspect.

When no tag line is used, the boat can be kept in the cross section by lining up with flags positioned on each end of the cross section, as illustrated in Figure I.5.72. Flags on one bank would suffice but it is better to have them on both banks. The position of the boat in the cross section can be determined by a transit on the shore and a stadia rod held in the boat.

Another method of determining the position of the boat is by setting a transit on one bank some convenient known distance from and at right angles to the cross-section line. The position of the boat is determined by measuring the angle α to the boat, measuring the distance CE and computing the distance MC as shown in Figure I.5.73.

A third method of determining the position of the boat is done with a sextant read from the boat. Position a flag on the cross-section line and another at a known distance perpendicular to the line. The boat position can be computed by measuring the angle β with the sextant, as shown in Figure I.5.73.

Boat position can also be determined by using the Global Positioning System (GPS) described in the section Global Positioning System with Differential Corrections (DGPS). This method is especially useful on very wide streams and in flood plains where other methods of determining boat position are not applicable.

Unless anchoring is more convenient, the motor must hold the boat stationary when depth and velocity readings are being taken.

Boat measurements are not recommended at velocities less than 0.3 m/s when the boat is subject to wave action. The up-and-down movement of the boat (and the meter) seriously affects the velocity observations. If the maximum depth in the cross

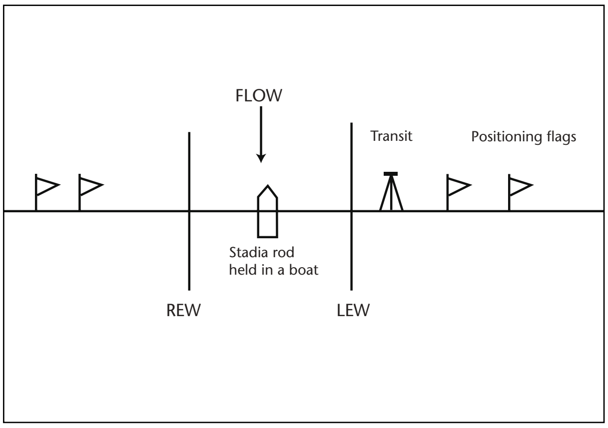


Figure I.5.72. Determining the position in a cross section using the stadia method

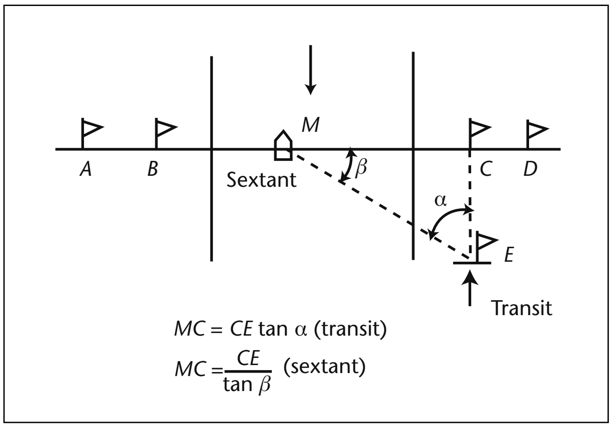


Figure I.5.73. Determining the position in a cross section using the angular method

section is less than 3 m and the velocity is low, use a rod for measuring the depth and for supporting the current meter. For greater depths and velocities use a cable suspension with a reel and sounding weight.

The procedure for measuring from a stationary boat using the boat boom and crosspiece is the same as that for measuring from a bridge or a cableway, as described in previous sections of this Manual.

The methods of measuring discharge from a moving boat, including the Acoustic Doppler Current Profiler method, will be described in Chapter 6.

5.6.12 **Networks of current meters**

Occasional special measurements require simultaneous velocities at several points in a cross section, distributed either laterally or vertically. For example, it may be necessary to measure a vertical-velocity profile quickly in unsteady flows and to check it frequently to determine the changes in shape of the vertical profile as well as the rates of these changes. In another example, for the measurement of tide-affected streams, it is desirable to measure the total discharge continuously during at least a full tidal cycle, approximately 13 hours. The need for so many simultaneous velocity determinations (one at each vertical in the cross section) for so long a period could be an expensive and laborious process using conventional techniques of discharge measurement.

A grouping of 21 current meters and special instrumentation has been devised by the USGS to facilitate measurements of the types just described. Only a few persons are required. The 21 meters are connected together so that the spacing between any two adjacent meters can be varied up to 60 m. Furthermore, each meter has sufficient handline cable to be suspended vertically from a bridge as much as 60 m. The meters have a uniform calibration. Revolutions of the rotors are recorded by electronic counters that are grouped compactly in one box at the center of the bank of meters. By flipping one switch the operator starts all 21 counters simultaneously, and after an interval of several minutes stops all counters. The indicated number of revolutions for the elapsed time interval is converted to a velocity for each meter. The distance between meters is known; a record of stage is maintained to evaluate depth; prior information at the site is obtained to convert point velocities in the verticals to mean velocities in those verticals. All of the information necessary to compute discharge in the cross section is therefore available, and is tabulated for easy conversion to discharge.

Similar group metering systems have been used in the United Kingdom, and possibly elsewhere. In one such study in the United Kingdom up to 40 current meters were operated simultaneously for a period of 24 hours at three different river sections (Herschy 1975, Herschy et al.,1978).

5.6.13 **Measurement of deep, swift streams**

Discharge measurements of deep, swift streams usually present no serious problems when adequate sounding weights are used and when floating drift or ice is not excessive. Normal procedures must sometimes be altered, however, when measuring these streams. The four most common circumstances are:

(a) It is possible to sound, but the weight and meter drift downstream;
(b) It is not possible to sound, but a standard cross section is available;
(c) It is not possible to sound, and a standard cross section is not available;
(d) It is not possible to put the weight and meter in the water.

Procedures are described below for use during measurements made under these conditions. The procedures for items (b), (c) and (d) are used where there is a stable cross section. The procedure to be used in unstable channels must be determined by conditions at each location.

Possible to sound; weight and meter drift downstream

For some streams it may be possible to sound the streambed, but because of the force of very high velocities the weight and meter are carried downstream. This may be a condition for only a few verticals near the centre of the stream, or it may affect many of the verticals. Corrections to the observed depths and meter settings must be made to account for the downstream drift. These corrections are commonly referred to as vertical angle corrections. The procedure for computing vertical angle corrections is described in a previous section of this report entitled “Depth corrections for downstream drift of current meter and weight”. The corrections can be computed manually or they may be computed automatically through the use of an electronic notebook such as the Aquacalc or Sutron DMX.

Not possible to sound; standard cross section available

When it is not possible to sound the streambed, a standard cross section from previous measurements at the bridge or cableway may be useful for determining depth. Such a cross section is useful only if all discharge measurements use the same permanent initial point for the stationing of verticals across the width of the stream. Also there should be an outside reference gauge or reference point on the bank or bridge to which the water-surface elevation at the measurement cross section may be referred. If these conditions are met, the following procedure can be used to make a discharge measurement:

(a) Determine the depths from the standard cross section, based on the water-surface elevation;
(b) Measure the velocity at 0.2 of the depth at each vertical;
(c) Compute the measurement in the normal manner using the measured 0.2-depth velocities as though they were the mean velocities in the vertical. Apply horizontal angle corrections if necessary. Use depths as determined in step 1 above;
(d) Determine the coefficient to adjust the 0.2-depth velocity to the mean velocity on the basis of previous measurements at the site by the two-point method. See a previous section of this report "Two-tenths-depth method";
(e) Apply the coefficient from step 4 to the computed discharge from step 3.

Not possible to sound; standard cross section not available

When it is not possible to sound the streambed and a standard cross section is not available, the following procedure should be used:

(a) Refer the water-surface elevation before and after the measurement to an elevation reference point on a bridge, on a driven stake or on a tree at the water's edge. It is assumed here that no outside reference gauge is available at the measurement cross section;
(b) Estimate the depth and observe the velocity at 0.2 of the estimated depth. The meter should be at least 0.6 m below the water surface. Record in the notes the actual depth the meter was placed below the water surface. If an estimate of the depth is impossible, place the meter 0.6 m below the water surface and observe the velocity at that point;
(c) Make a complete measurement, including some vertical-velocity curves, at a lower stage when the streambed can be sounded;
(d) Use the complete measurement and difference in stage between the two measurements to determine the cross section of the first measurement. To determine whether the streambed has shifted, the cross section should be compared with one taken for a previous measurement at that site;
(e) Use vertical-velocity curves or the relationship between mean velocity and 0.2-depth velocity to adjust the velocities observed in step 2 to mean velocity. Apply horizontal angle corrections as necessary;
(f) Compute the measurement in the normal manner using the depths from step 4 and the velocities from step 5.

Not possible to put the weight and meter in water

If it is impossible to put the weight and meter in the water because of high velocities and/or floating drift, the following procedure should be used:

(a) Obtain depths at the measurement verticals from a standard cross section if one is available. If a standard cross section is not available, determine depths by the method explained above in the section "Not possible to sound; standard cross section not available";
(b) Measure surface velocities by timing floating drift or by use of an optical flowmeter;
(c) Compute the measurement in the normal manner using the surface velocities as though they were the mean velocities in the vertical, and using the depths from step 1;
(d) Apply the appropriate velocity coefficient to the discharge computed in step 3. Use a coefficient of 0.86 for a natural channel and 0.90 for an artificial channel.

The optical current meter is described in a previous section. It is portable, battery operated and requires no great skill for quick and accurate readings of the surface rate of flow. It is not immersed, so it does not disturb the flow, and it is in no danger of damage from floating debris or ice.

It is well to note that just after the crest, the amount of floating drift or ice is usually greatly reduced, and it may be possible to obtain velocity observations with a current meter. These observations might be useful in defining the velocity coefficient mentioned in step 4 above.

5.6.14 Computation of mean gauge height of a discharge measurement

The mean gauge height of a discharge measurement represents the mean height of the stream during the period the measurement was made and is referenced to the datum of the gauging station. Just as an accurate determination of the discharge is important, an accurate determination of the mean gauge height is likewise important because it is one of the coordinates used in plotting the discharge measurement to establish the stage-discharge relation. The computation of the mean gauge height presents no problem when the change in stage is small, such as 0.05 m or less, for then the mean may be obtained by averaging the gauge heights at the beginning and end of the measurement. However, measurements must sometimes be made during floods or regulation when the stage changes significantly more than 0.05 m.

To compute an accurate mean gauge height for a discharge measurement the gauge must be read at the beginning and end of the discharge measurement and several times during the measurement if the gauge height is changing significantly. If the station is equipped with a digital recorder or data logger that automatically records at intervals of 15-minutes or less the intermediate gauge-height readings may be taken from those instruments after the measurement is completed. The hydrographer should accurately synchronize watch time and recorder time and should record watch time for selected verticals at intervals during the discharge measurement. If the recording interval is greater than 15-minutes (that is 30-minutes or 1-hour), intermediate gauge height readings should be obtained by reading the gauge once or twice during the discharge measurement.

If the change in stage during the measurement is greater than about 0.05 m, the mean gauge height should be computed by weighting the gauge-height readings. In the past the mean gauge height was computed by weighting the gauge readings with partial discharges from the discharge measurement. Later studies show that this method tends to over-estimate the mean gauge height. Time weighting has also been used to compute a weighted mean gauge height but this method tends to under-estimate the mean gauge height. Therefore it is recommended that both methods of weighting be used for discharge measurements having stage changes of 0.05 m or more and that an average of the two results be used for the mean gauge height.

It is usually best to plot the gauge height readings so that intermediate readings can be interpolated where necessary. Particular attention should be given to breaks in the slope of the gauge height graph. Figure I.5.74 illustrates a plot of gauge heights and the computation of mean gauge height for a discharge measurement. Gauge heights for this measurement were determined from the stage recorder at 15-minute intervals.

In the discharge-weighting procedure the partial discharges measured between recorded watch times are used with the mean gauge height for that same time period. The equation used to compute the weighted mean gauge height is:

$$H = \frac{q_1h_1 + q_2h_2 + q_3h_3 + ...q_nh_n}{Q} \qquad (5.17)$$

where H = weighted mean gauge height, in m, Q = total discharge measured, in m^3/s = $q_1 + q_2 + q_3$ ....+ q_n, q_1, q_2, q_3,...q_n = amount of discharge measured during time interval 1, 2, 3...n, in m^3/s^1. h_1, h_2, h_3,...h_n = average gauge height during time interval 1, 2, 3...n, in m.

Figure I.5.74 shows the computation of a discharge-weighted mean gauge height based on equation 5.17, the graph at the bottom of the figure and

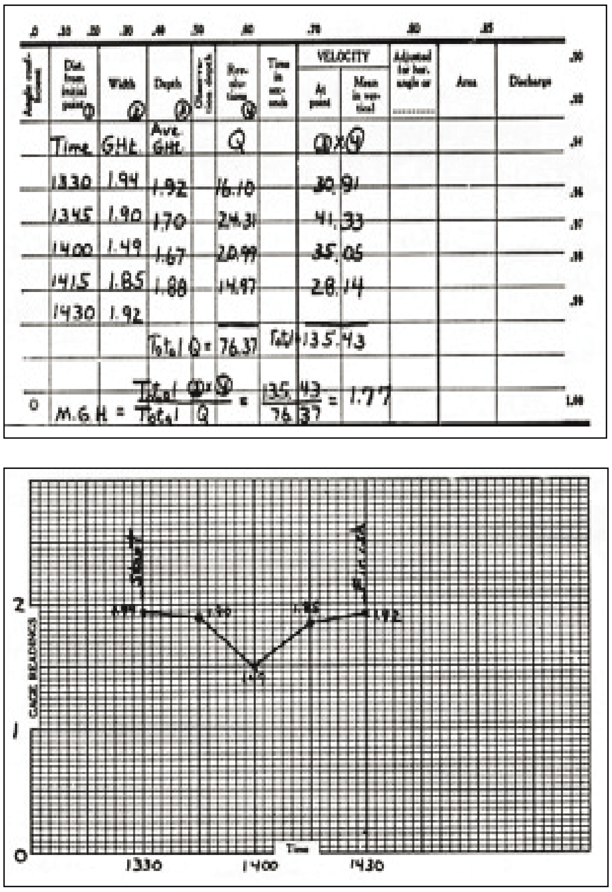


Figure I.5.74. Graph of readings used to compute a weighted mean gauge height

the discharges shown in the top of the figure. The discharges are taken from the original current-meter measurement.

In the time-weighting procedure the arithmetic mean gauge height for time intervals between breaks in the slope of the gauge-height graph are used with the duration of those time periods. The equation used to compute mean gauge height is:

$$H = \frac{t_1h_1 + t_2h_2 + t_3h_3 + \ldots\ldots t_nh_n}{T} \tag{5.18}$$

where H = weighted mean gauge height, in m, T = total time for the measurement, in minutes = $t_1 + t_2 + t_3 + \ldots\ldots t_n$, $t_1, t_2, t_3\ldots\ldots t_n$ = duration of time intervals between breaks in the slope of the gauge height graph, in minutes, and $h_1, h_2, h_3\ldots\ldots h_n$ = average gauge height, in m, during time interval 1, 2, 3, ...n.

Using the data from Figure I.5.74, the computation of the time-weighted mean gauge height is as follows:

Average gauge height, h, in m	*Time interval, t, in minutes*	*h x t*
1.92	15	28.80
1.70	15	25.50
1.67	15	25.05
1.88	15	28.20
Totals	60	107.55

The mean gauge height is computed as, H = 107.55/60 = 1.79 m.

In this example there is little difference between the discharge-weighted mean gauge height (1.77 m) and the time-weighted mean gauge height (1.79 m). The average of the two values, 1.78 m, is the preferred mean gauge height for the discharge measurement.

When extremely rapid changes in stage occur during a measurement, the weighted mean gauge height is not truly applicable to the discharge measured. To reduce the range in stage during the measurement, efforts should be made to reduce the time required for making the discharge measurement. It should be realized that shortcuts in the measurement procedure usually reduce the accuracy of the measured discharge. Therefore, measurement procedures during rapidly changing stage must be optimized to produce a minimal combined error in measured discharge and computed mean gauge height. The following section of this report describes procedures for making discharge measurements during periods of rapidly changing stage.

5.6.15 Discharge measurements during rapidly changing stage

Discharge measurements during periods of rapidly changing stage are more difficult to obtain and accuracy is often not as good as for measurements made when the stage is fairly constant. The computation of total discharge and the corresponding stage are both subject to more uncertainty when the stage changes significantly during the period of the measurement. Two procedures are suggested for shortening the time required for a discharge measurement. The first procedure is applicable more for large streams where the stage changes are usually not as great as for small streams. The second procedure is designed more for the flash flood conditions experienced on small streams, where peaks are almost of momentary duration, and where the rising and falling stage is very rapid.

Large stream measurements, rapidly changing stage

During periods of rapidly changing stage on a large stream measurements should be made as quickly as possible to keep the change in stage to a minimum. This will minimize discharge errors caused by shifting of flow patterns and other variables as the stage changes and will provide a more accurate stage computation for the measurement. To reduce the time required for making a discharge measurement it is necessary to make fewer than the usual number of observations and to shorten the time of those observations. This is sometimes referred to as a shortcut method. Following is a list of some of the things that can be done to reduce the time:

(a) Use the 0.6-depth method rather than the 0.2 and 0.8 method. If the 0.6 method cannot be used because the flow is too swift or if debris makes it too hazardous then the 0.2-depth method or the subsurface method could be used. If an optical meter is available, surface velocity measurements can by used;
(b) Reduce the velocity observation time to about 20-30 seconds. This is referred to as half-counts;
(c) Reduce the number of sections taken to about 15-18. For some conditions even less than 15 sections may be used;
(d) Observe and record the watch time at about every third vertical. If possible, observe and record the stage once or twice during the measurement.

By incorporating the above practices a measurement can usually be made in 15-20 minutes. If the surface or subsurface method for observing velocities is used then some vertical-velocity curves will be needed later to establish coefficients to convert observed velocity to mean velocity. A weighted mean gauge height should be computed for the discharge measurement, as described in a previous section.

Small stream measurements, rapidly changing stage

A series of instantaneous discharge measurements can be made during flash flood conditions on small streams by rating individual sub-sections, or verticals. This method requires repeated observations of gauge height, depth and velocity at selected verticals during the rise and fall of the flood wave. Two procedures are described herein with the primary difference being the method of determining depth at each vertical. In the first procedure the streambed elevation referenced to gauge datum is pre-determined for each selected vertical. Depth is then determined as the difference between the gauge height and the streambed elevation. In the second procedure depth is measured at the selected verticals by sounding each time velocity is observed. The first procedure is faster, however it may not be suitable if the streambed is unstable.

The method of computing the discharge measurements is also slightly different for the two procedures. For both procedures a rating of gauge height versus mean velocity is required for each sub-section, or vertical. For the second procedure a rating of gauge height versus depth is required for each vertical. The two procedures are described below:

(a) Procedure 1, depth is computed from pre-determined streambed elevations at each vertical:
   (i) Select about 10 verticals, or sub-sections. For very small streams fewer verticals may be used. Mark the selected verticals in some way so that repeated observations can be made at the same vertical each time;
   (ii) Determine the streambed elevation referenced to gauge datum for each selected vertical prior to making the series of discharge measurements. After the flood recedes, the streambed elevations at each vertical should be determined again to see if changes occurred during the flood. If the streambed is not stable it will be necessary to interpolate the changes based on time or on the basis of the best judgement of the hydrographer. Depth is determined at each vertical as the difference between this elevation and the gauge height;
   (iii) Take velocity observations at each vertical using the 0.6 depth method. Full counts of 40 seconds or more are recommended, but half-counts may be used if the stream is rising or falling extremely fast. If the 0.6 method cannot be used then take velocity observations at the 0.2 depth or the subsurface depth. If an optical flow meter is available it may be used to take surface velocity readings. For surface and subsurface velocity readings it will be necessary to determine the coefficient required for converting the readings to a mean velocity. Meter positions should be based on the depth, as computed in item (ii) above;
   (iv) Make observations of other factors that would affect the computation of discharge, such as horizontal angle coefficients;
   (v) Repeat the velocity and other observations at each of the selected verticals several times over the duration of the flood wave;
   (vi) Record the watch time that each vertical is measured and make corresponding gauge height observations frequently during the period of the flood wave;
   (vii) Develop a rating of gauge height versus mean velocity for each of the selected verticals. If surface or subsurface velocity observations were made, adjustments should be applied so that the rating represents mean velocity in the vertical. In some cases it may be necessary to develop more than one rating for each vertical. For instance, a rating for the rising side of the flood wave and a separate rating for the falling side of the flood wave may be necessary;
   (viii) Select a gauge height for which a discharge measurement is to be computed. Use a standard discharge measurement note sheet for computing the discharge measurement. Enter the stationing for the edge of water and for each of the selected verticals. Enter the depths at each vertical, computed on the basis of the selected gauge height minus the streambed elevation. Enter the mean velocity at each vertical on the basis of the gauge height versus mean velocity ratings. Enter other adjustments, such as horizontal angle coefficients, as observed during the

observation of velocities. Compute the discharge measurement similar to a regular discharge measurement;

(ix) Repeat the process described in item (viii) above for other selected gauge heights. If the ratings of gauge height versus mean velocity change, such as for rising and falling stage, then separate discharge measurements should be computed for the rising and falling limbs of the flood wave.

(b) Procedure 2, depth is measured by sounding at each vertical:

(i) Select about 10 verticals, or sub-sections, as described in the first procedure above. For very small streams, fewer verticals may be used. Mark the selected verticals in some way so that repeated observations can be made at the same vertical each time;

(ii) Determine the depth for each selected vertical by sounding the streambed each time a vertical is measured. This method is used primarily when it is possible to make soundings easily and when there is likelihood of streambed elevation changes caused by scour or fill during the course of the measurement;

(iii) Take velocity observations at each vertical using the 0.6 depth method. Full counts of 40 seconds or more are recommended but half-counts may be used if the stream is rising or falling extremely fast. It is not likely that surface or sub-surface observations will be required because depth soundings are possible with this procedure. Meter positions should be based on the sounded depth;

(iv) Make observations of other factors that would affect the computation of discharge, such as horizontal angle coefficients;

(v) Repeat the observations of depth, velocity and other variables at each of the selected verticals several times over the duration of the flood wave;

(vi) Record the watch time that each vertical is measured and make corresponding gauge height observations frequently during the period of the flood wave;

(vii) Develop a rating of gauge height versus mean velocity for each of the selected verticals. As described in the first procedure it may be necessary to develop more than one rating, such as for the rising and falling sides of the flood wave;

(viii) Develop a rating of gauge height versus depth for each of the selected verticals. If streambed changes occur during the measurements it will be necessary to take these into account by making appropriate corrections;

(ix) Select a gauge height for which a discharge measurement is to be computed. Use a standard discharge measurement note sheet for computing the discharge measurement. Enter the stationing for the edge of water and for each of the selected verticals. Enter the depths at each vertical, based on the selected gauge height and the ratings of gauge height versus depth. Enter the mean velocity at each vertical on the basis of the gauge height versus mean velocity ratings. Enter other adjustments, such as horizontal angle coefficients, as observed during the observation of depths and velocities. Compute the discharge measurement similar to a regular discharge measurement;

(x) Repeat the process described in item (ix) above for other selected gauge heights. If the depth and/or mean velocity ratings change, such as for rising and falling stage, or for streambed scour or fill, then separate discharge measurements should be computed for conditions before and after the changes.

5.6.16 Correction of discharge for storage during measurement

Most discharge measurements are made at or near the gauging station and the gauge control. At some gauges it may be necessary to make discharge measurements at a significant distance away from the gauge and/or control. For instance, during a flood the only place to measure may be at a bridge located some distance from the gauge. Or for some sites the low-water section control may be located at a significant distance downstream from the gauge. If a discharge measurement is made at a significant distance from the gauge control during a change in stage the discharge passing the control during the measurement will not be the same as the discharge at the measurement section. In these situations an adjustment must be applied to the measured discharge to account for the change in channel storage that occurs between the measurement section and the control during the period of the measurement. The adjustment for channel storage is computed by multiplying the channel surface area by the average rate of change in stage in the reach between the measurement section and the control. The equation is:

$$Q_G = Q_m \pm WL\frac{\Delta h}{\Delta t} \qquad (5.19)$$

where Q_G = discharge going over the control, in m^3/s, Q_m = measured discharge, in m^3/s, W = average width of stream between measuring section and control, in m, L = length of reach between measuring section and control, in m, Δh = average change in stage in the reach L during the measurement, in m, and Δt = elapsed time during measurement, in seconds.

The change in stage should be determined at each end of the reach (that is at the control and at the measuring section) and an average of these two values used. Generally, the gauge height at the gauge is used at one end of the reach, and a reference point (RP) or a temporary gauge is set at the other end of the reach. The water-surface elevation at each end of the reach is determined before and after the measurement to compute Δh. If the measurement is made upstream from the control the adjustment will be plus for falling stages and minus for rising stages. If the measurement is made downstream from the control it will be minus for falling stages and plus for rising stages.

An example computation for a flood measurement that was made 966 m upstream from the gauge (and control) during a period of changing stage is shown below:

- Measurement made 0.6 mile upstream, L = 966 m. Average width between measuring section and control, W = 45.7 m.
- Gauge height at beginning of measurement, at the gauge (and control), = 1.780 m.
  Gauge height at end of measurement, at the gauge (and control), = 2.054 m.
  Change in stage at gauge (and control), 2.054 – 1.780 = + 0.274 m.
- Gauge height at beginning of measurement, at measuring section, = 3.877 m.
  Gauge height at end of measurement, at measuring section, = 4.191 m.
  Change in stage at measuring section, 4.191 – 3.877 = + 0.314 m. Readings taken at measuring section from a reference point before and after measurement.
- Average change in stage in the reach, Δh = (0.274 + 0.314)/2 = 0..305 m.
  Elapsed time during measurement, Δt = 1.25 hours = 4,500 seconds.
  Measured discharge, Q_m = 240.6 m^3/s.

$$Q_G = 240.6 - 45.7\ (966)\ \frac{0.305}{4500} = 240.6 - 3.0 = 237.6\ m^3/s \qquad (5.20)$$

This discharge should then be rounded to 238 m^3/s, which represents the discharge at the gauge, or control.

The adjustment of the measured discharge for storage between the gauge or control and measuring site, as described above, is a separate and distinct problem from that of making adjustments owing to variable water-surface slopes caused by changing discharge. Those adjustments are related to stage-discharge rating analysis and are described by Kennedy (1984), and by Rantz (1982). The storage adjustment should be made immediately following the completion of the discharge measurement and the resulting adjusted discharge is later used for rating analysis.

5.6.17 Uncertainty of current-meter discharge measurements

The accuracy, or uncertainty, of a discharge measurement is dependent on many factors, including the equipment used, the location and characteristics of the measuring section, the number and spacing of verticals, the rate of change in stage, the measurement of depth and velocity, ice and/or debris in the measuring section, wind, experience and carefulness of the hydrographer and various other factors that can occur during the process of making the measurement. In some countries the evaluation of the accuracy of a measurement is based on a qualitative assessment that takes into account some or all of these factors.

A quantitative calculation of uncertainty for discharge measurements can also be made based on established uncertainties in the measurement of width, depth, velocity and other related factors. The collection and processing of data for determination of uncertainties in flow measurements are set forth in ISO 1088 (2007). The application of both the quantitative and qualitative methods is described in Chapter 10 of this Manual.

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CHAPTER 6

MEASUREMENT OF DISCHARGE BY ACOUSTIC AND ELECTROMAGNETIC METHODS

6.1 INTRODUCTION

At sites with significant backwater stable stage-discharge relations are not possible and on large streams and estuaries conventional methods of measuring discharge by current meter are impractical or very costly. Gauging sites may be inundated or inaccessible during floods. During unsteady flow conditions measurements need to be made as rapidly as possible. Measurements on tide-affected rivers must not only be made rapidly, but often continuously, throughout a tidal cycle.

This chapter deals with electronic methods of discharge measurement developed to overcome some of the above difficulties. The following four methods are described:

(a) Moving-boat discharge measurements using Acoustic Doppler Current Profilers (ADCP) described in section 6.2;
(b) River gauging using Acoustic Doppler Velocity Meters (ADVM) described in section 6.3;
(c) River gauging using ultrasonic (Acoustic Velocity Meter (AVM)) described in section 6.4; and
(d) Total discharge method using an electromagnetic full-channel-width coil described in section 6.5.

It should be noted that the terms ultrasonic, sonic and acoustic are sometimes used interchangeably even though by definition they are distinct. In the United States of America the term acoustic is prevalent, whereas in Europe the terms sonic and ultrasonic are preferred.

Acoustic discharge measurement methods have become widespread. There are some places that use acoustic current meters for nearly all discharge measurements. The technologies that have enabled such use are the Acoustic Doppler Velocimeters (ADVs) discussed in Chapter 5 that are used in wadeable streams and ADCPs that are to be discussed in this chapter. New deployment methods such as unmanned tethered boats allow hydrographers to make ADCP measurements from a bridge or cableway making ADCP measurements practical at sites where it is difficult to launch a manned boat. Acoustic measurement technologies continue to evolve rapidly. There has been a proliferation of instruments with new features. For example, until recently ADCPs could only be used in depths greater than about 1 m. ADCPs available in the last several years can be used in water as shallow as about 0.3 m.

6.2 MOVING BOAT METHOD USING ACOUSTIC DOPPLER CURRENT PROFILERS

There are three moving boat discharge measurement methods: the manual moving boat method, the automatic moving boat method and the Acoustic Doppler Current Profiler (ADCP) method. The first two of these, the manual and automatic methods, are virtually the same except for the method of data acquisition and recording. Equipment requirements are essentially the same for both methods; however the manual method requires direct reading of instruments and manual recording of observations by on-board personnel. The automatic method uses a computerized system whereby all instruments are automatically read and recorded by an on-board computer. The manual moving boat method and the automatic moving boat method are not used today, being supplanted by the ADCP moving boat method. A detailed description of the manual moving boat method can be found in Smoot and Novak (1969) and ISO 4369 (1979). These descriptions will not be repeated in this Manual.

The ADCP method uses a moving boat to traverse the stream, just as the manual and automatic methods, however instrumentation is different. The ADCP method measures velocity magnitude and direction using the Doppler shift of acoustic energy reflected by material suspended in the water column providing essentially a complete vertical velocity profile. It also tracks the bottom providing stream depth and boat positioning. A complete description of this method will be given in the following paragraphs.

6.2.1 Basic principle of the moving boat method

The moving-boat technique is similar to the conventional current-meter measurement in that both use the velocity-area technique to determine discharge (see Chapter 5). In each method a measurement is the summation of the products of the subsections of the stream cross-section and their respective average velocities. Both techniques require that the following information be obtained:

(a) Location of sampling verticals 1, 2, 3,...n across the stream in reference to the distance from an initial point;
(b) Stream depth, *d*, at each observation vertical;
(c) Stream velocity, *V*, perpendicular to the cross-section at each observation vertical.

During the traverse of the boat across the stream, a sonic sounder or ADCP records the profile of the cross-section, and a continuously operating current meter or ADCP senses the combined stream and boat velocities. Direction of flow is determined either by a vertical vane and angle indicator or by the ADCP measurements. These instruments collect the data necessary for computing discharge for the cross-section. Normally data are collected at 20 to 40 observation points (or verticals in the case of the ADCP method) in the cross-section for each run. Experience and testing have shown that discharges determined by the moving-boat technique match, within 5 per cent, discharges determined by conventional means.

The principal difference between the conventional measurement and the moving-boat measurement lies in the method of data collection. The conventional current-meter method uses what might be called a static approach in its manner of sampling; that is, the data are collected at each observation point in the cross-section while the observer is in a stationary position. This is in contrast to the dynamic approach to data collection utilized in the moving-boat method. Here, data are collected at each observation point (or vertical) while the observer is aboard a boat that is rapidly traversing the cross-section.

6.2.2 Theory of Acoustic Doppler Current Profiler moving boat method

The preceding paragraphs provide a brief introduction to moving boat discharge measurements. Moving boat methods of measuring stream flow have been under development for the past 35 years and have been successful. The ADCP moving boat method has reached a point in its development that it is clearly superior to the original manual and automatic methods. Use of ADCPs for river discharge measurements began in the mid-1980s. At that time there was one manufacturer and one model.

Internationally the use of ADCPs for measurements of discharge is increasing rapidly. For example the ADCP has been widely introduced in the United States Geological Survey (USGS) through an aggressive training and field support program. Rapid developments in ADCP technology have allowed for increasing use within the USGS while also affording many opportunities for demonstrating its utility. For example, during the summer of 2003 use of ADCPs permitted USGS field crews in Indiana to make 55 flood measurements – two to three times the number of measurements that would have been possible during the same time. The same number of personnel was used as were traditionally used for conventional current meters. Some measurements obtained in Indiana during the floods using the ADCP would not have been possible using the conventional current meter method (Hirsch and Costa, 2004).

Currently, there are two manufacturers of bottom-tracking ADCPs designed specifically for discharge measurements: Teledyne RD Instruments and SonTek. Both manufacturers offer instruments with various features. The basic theory of operation is the same for instruments from both manufacturers: the measurement of the Doppler shift of sound waves in the water to compute water velocities and the speed of the vessel in relation to the streambed. While the basic theory of operation is the same, the measurement of the Doppler shift is inherently noisy and RDI and SonTek/YSI have taken different approaches to averaging this noise (Mueller, 2003). For example, RDI uses broad-band (BB) signal processing while SonTek relies on very fast ping rates to reduce measurement uncertainty. Both manufacturers offer a variety of models with different acoustic frequencies. As of this writing, SonTek offers RiverSurveyor ADCPs with acoustic frequencies of 0.5, 1.0, 1.5, 3.0 megahertz (MHz). RDI offers the Rio Grande model of BB-ADCP with acoustic frequencies of 600 and 1 200 kilohertz (kHz) and the StreamPro ADCP that has an acoustic frequency of 2.0 MHz. Generally speaking, lower frequencies will work in greater depths, while higher frequencies will operate in shallower depth ranges. Mueller (2003) field tested RDI and SonTek ADCPs and found that systems from both manufacturers performed similarly, determining that discharge measurements were, on average, within 5 per cent of Price meter measurements or discharges computed from gauging station stage-discharge ratings.

ADCPs are powerful yet complex tools. Because ADCPs are complex it is very important that users understand the theory of operation and nuances of techniques before measurements are made. Training classes and follow-up exercises should be provided to field personnel before they make ADCP measurements. Because of the complexity of ADCPs and the rapid pace of changes to hardware and software, this Manual will not detail all aspects of their operation. Basic theory and good measurement practices are discussed and the reader is encouraged to read the references listed in subsequent sections for more detailed information.

The following sections of this Manual describe the theory and operation of ADCPs. The theory and operation is the same for instruments from both manufacturers. Recommended guidelines and techniques to obtain good discharge data also are much the same. Much of the information in the following sections was taken from Simpson (2002), Oberg, Morlock, and Caldwell (2005), Morlock (1996), Simpson and Oltmann (1993) and Lipscomb (1995).

Before the operational aspects of an ADCP measurement can be understood some of the basic physical properties of sound and sound propagation through different mediums must be examined. This section introduces basic Acoustic Doppler Velocity measurement principles and some of the problems associated with such measurements. Much of the information in this section is taken from R.D. Instruments, Inc. (1989, 1996).

The Physics of Sound

What commonly is perceived as sound is a vibration of our eardrums caused by the arrival of pressure waves. The eardrum transfers the pressure-wave information to parts of the inner ear where the mechanical energy of the pressure waves is converted to an electrical signal. The brain interprets this electrical signal as sound.

Sound waves can occur in most media (water, air and solids) and are similar to water waves. Sound waves have crests and troughs that correspond to bands of high and low air pressure, just as water waves have crests and troughs that correspond to high and low water-level elevations. Pitch (frequency) of sound waves increases as the wavelength (the distance between the wave peaks) becomes shorter. This frequency usually is expressed in hertz (Hz). One hertz equals one wave per second. The human ear can hear frequencies from about 40 Hz to about 24 kilohertz (kHz) (24 000 Hz). These frequencies are called the sonic frequencies. Sound frequencies below about 40 Hz are called subsonic, and sound frequencies above 25 000 Hz are called ultrasonic.

The Doppler Principle Applied to Moving Objects

The ADCP uses sound to measure water velocity. The sound transmitted by the ADCP is in the ultrasonic range, well above the range of the human ear. The lowest frequency used by commercial ADCPs is around 30 kHz, and the common range used by the USGS for riverine measurements is between 300-3 000 kHz.

The ADCP measures water velocity using a principle of physics discovered by Christian Johann Doppler (1842). Doppler's principle relates the change in frequency of a source to the relative velocities of the source and the observer. Doppler (1842) stated his principle in the article, "Concerning the Coloured Light of Double Stars and Some Other Constellations in the heavens," while working in Prague, Czechoslovakia. The Doppler principle can be described best using the water-wave analogy.

Consider a stationary observer watching a series of waves that are passing at a rate of one wave per second. This rate is analogous to a transmit frequency of 1 Hz. Now consider that the observer is boating toward the wave source at a rate of four wavelengths per second. Because the waves are passing at a rate of one wave per second, the observer notices the passage of five waves during each second of his boating trip. He senses that the rate of the passing waves is 5 Hz, though the wave source is still emitting waves at 1 Hz. This phenomenon is known as the Doppler effect.

Many people have experienced the Doppler effect while on a busy street. The sound of a car horn seems to drop in frequency as the car passes and recedes from the observer. The apparent lowering of frequency is called the Doppler shift. The car is a moving sound-wave source; therefore, when the car is approaching an observer, the frequency of the sound waves striking the observer's ear drums is proportional to the speed of the car (in wavelengths per second) plus the frequency of the car horn in hertz. When the car is receding from the observer, the frequency of the sound waves striking the observer's ear drums is proportional to the frequency of the car horn (in hertz) minus the speed of the car (in wavelengths per second). If the exact source frequency is known and the observed frequency can be calculated, equation 6.1 can be used to calculate the Doppler shift due to the relative velocities of the source and observer:

$$F_D = F_S \left(\frac{V}{C} \right) \tag{6.1}$$

where F_D = the Doppler shift frequency, in hertz; F_S = the transmitted frequency of the sound from a stationary source, in hertz; V = relative velocity between the sound source and the sound wave receiver (the speed at which the observer is walking toward the sound source), in meters per second; and C = the speed of sound, in meters per second.

Note that:
- If the observer walks faster (V increases), the Doppler shift (F_D) increases.
- If the observer walks away from the sound (V is negative), the Doppler shift (F_D) is negative.
- If the frequency of the sound (F_S) increases, the Doppler shift (F_D) increases.
- If the speed of sound (C) increases, the Doppler shift (F_D) decreases.

Measuring Doppler shifts using acoustic backscatter

An ADCP applies the Doppler principle by bouncing an ultrasonic sound pulse off small particles of sediment and other material which collectively are referred to as backscatterers that are present, to some extent, even in optically clear water. There are, of course, exceptions to every rule and in tow tanks and some natural rivers backscatterer density can be too low for the proper operation of an ADCP.

The ADCP transmits an acoustic ping, into the water column and then listens for the return echo from the acoustic backscatterers in the water column. Upon receiving the return echo the ADCPs onboard signal-processing unit calculates the Doppler shift using a form of autocorrelation wherein the signal is compared with itself later. A schematic diagram of a transmitted acoustic ping and the resulting reflected acoustic energy are shown in Figure I.6.1. Note that very little reflected acoustic energy is backscattered towards the transducer in Figure I.6.1; most of the acoustic energy is absorbed or reflected in other directions.

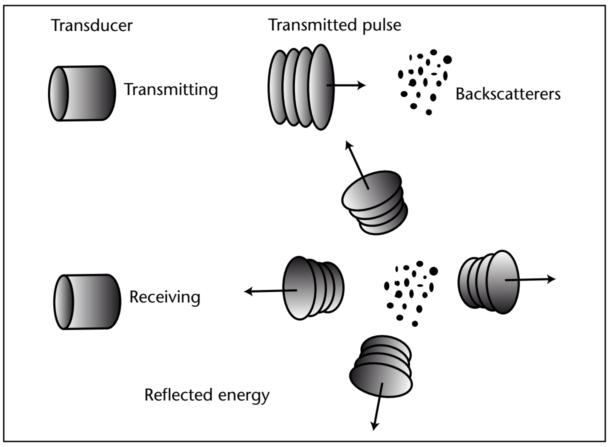

Figure I.6.1. An acoustic pulse being backscattered

Measuring Doppler shifts from a moving platform

When the scatterers are moving away from the ADCP, the sound (if it could be perceived by the scatterers) shifts to a lower frequency. This shift is proportional to the relative velocity between the ADCP and the scatterers, Figure I.6.1. Part of this Doppler-shifted sound is backscattered towards the ADCP as if the scatterers were the sound source, Figure I.6.2. The sound is shifted one time as perceived by the backscatterer and a second time as perceived by the ADCP transducer.

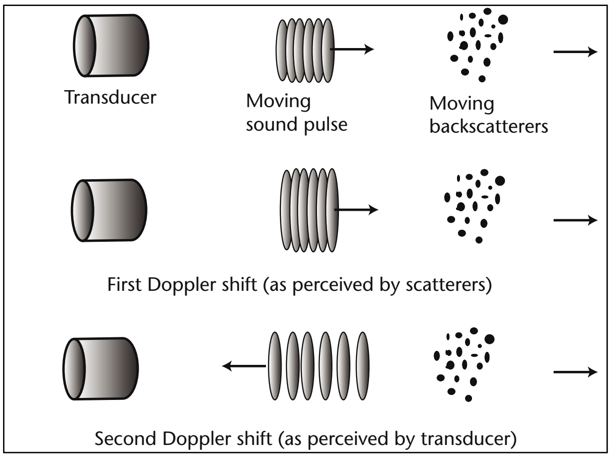

Figure I.6.2. Reflected pulses showing two Doppler shifts

Because there are two Doppler shifts, equation 6.1 becomes equation 6.2:

$$F_D = 2F_S\left(\frac{V}{C}\right) \tag{6.2}$$

If the sound source and receiver move, relative to the Earth, but stay at a fixed distance from one another, there is no Doppler shift. Only a change in distance between the source and receiver will cause a Doppler shift. The Doppler shift exists only when sound sources and receivers move in relation to each other. The Doppler shift between the source and the Earth exactly cancels the opposite shift between the Earth and the receiver.

Mathematically, this means the Doppler shift results from the velocity component in the direction of the line between the source and receiver as shown in equation 6.3:

$$F_D = 2F_S\left(\frac{V}{C}\right)\cos(\theta) \qquad (6.3)$$

where θ is the angle between the relative velocity vector and the line between the ADCP and scatterers.

ADCP three-dimensional velocity components

In a vessel mounted system the transducers are mounted near the water surface and aimed downward. Figure I.6.3 shows a typical ADCP. Note that there are four independently working acoustic beams with each beam angled 20-30 degrees from the vertical axis of the transducer assembly. This configuration of beams is the so-called Janus configuration, named after the Greek God, Janus, who could simultaneously look forward and backward.

To visualize the three-dimensional capabilities of the Janus configuration, refer to Figure I.6.4 while reading the following scenarios:

(a) If the starboard (90 degrees left of forward) beam has a positive Doppler shift, the port (90 degrees right of forward) beam has a negative Doppler shift, and the forward and aft beams have no Doppler shift, then the water is flowing from starboard to port (or the water is still and the boat is sliding starboard);
(b) If the forward beam has a negative Doppler shift, the aft beam has a positive Doppler shift, and the starboard and port beams have no shift, then the water is flowing under the boat from aft to forward (or the water is still and the boat is backing);

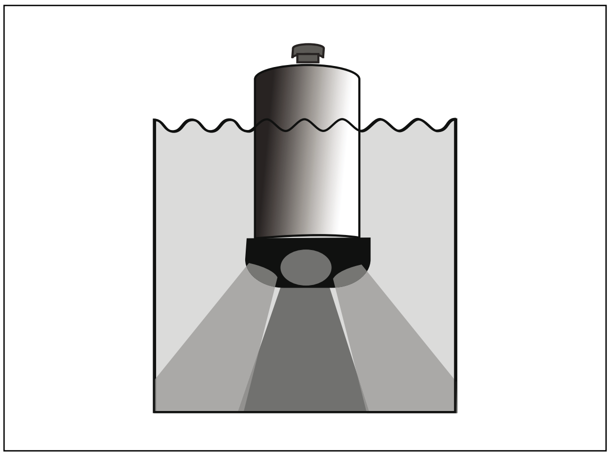

Figure I.6.3. Downward looking, convex-head Acoustic Doppler Current Profiler.

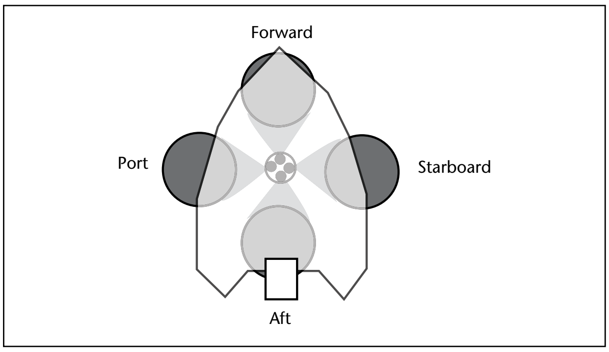

Figure I.6.4. Boat-mounted Acoustic Doppler Current Profiler with the "Janus" configuration

(c) If the forward and port beams have a positive Doppler shift of magnitude 1 and the aft and starboard beams have a negative Doppler shift of magnitude 1, then water is passing under the boat from a point halfway between forward and port with a magnitude of the square root of 2, or 1.414 (or the water is still and the boat is crabbing toward the forward port quarter at a magnitude of 1.414);
(d) If all four beams have a positive Doppler shift, the water is flowing upward toward the hull of the boat (or the water is still).

The computation of velocity in three dimensions (x, y and z) requires at least three acoustic beams. Some ADCP manufacturers use a four-beam configuration and the fourth redundant beam is used to compute an error velocity. This error velocity can be used to test the assumption that flow volume of water bounded by the four measurement beams is homogeneous. Velocity homogeneity means that the water velocities do not change significantly in magnitude or direction within the confines of the acoustic beam footprint.

Error velocity is defined as the difference between a velocity measured by one set of three beams versus a velocity measured by another set of three beams during the same time frame. A difference between these two measurements could be caused by a bad or corrupt beam velocity or by non-homogeneity. In practice, a small error velocity almost always exists because complete homogeneity of the velocity field rarely occurs during field measurements.

6.2.3 ADCP water velocity profile measurements

The ADCP is best known for its capability to measure profiles of water velocity. A velocity profile can be compared roughly to using a number of point-velocity meters that are suspended in the vertical axis of a water-velocity field. Theoretically, the velocity measured by each conventional current meter is analogous to the velocities measured at the centre of ADCP depth cells. However, the analogy between a string of current meters and an ADCP profile is not perfect. Current meters measure water velocity at individual points in the vertical profile, whereas velocities that are measured by the ADCP and assigned to individual depth cells are really the centre-weighted mean of velocities that are measured throughout the sample.

The ADCP profiling capability is accomplished by time gating and sampling the received echo at increasingly longer time intervals as the acoustic-beam wave fronts vertically traverse the water column. The ADCP transmits a ping along each acoustic beam and then time gates the reception of the returned echo on each beam into depth cells, or bins. Speed and direction are then calculated (using a centre-weighted mean of the velocities measured in the depth cell) and assigned to the centre of each depth cell over the measured vertical.

To measure absolute water velocities (water velocities relative to the Earth), the ADCP must sense and measure the velocity of the ADCP, relative to the river bottom. If the velocity of the water is known relative to the ADCP, and the velocity of the ADCP is known relative to the river bottom, then the water velocity, relative to the bottom, can be calculated. The bottom-track pulse is somewhat longer than the water-track pulse to properly ensonify the bottom. All ADCP instruments that are designed to measure discharge have the ability to calculate vessel velocity using a bottom-track pulse. The bottom-track ping also is used to measure the profiled depth range from each beam. These depth- range measurements are averaged to obtain a depth for the measured velocity profile.

6.2.4 ADCP discharge measurement principles

In the previous sections the theory and basic principles of how the ADCP is used to measure velocity was discussed. This section will describe the principle of calculating discharge from data collected using an ADCP. A basic knowledge of conventional river discharge-measurement techniques is necessary to understand how an ADCP measures discharge. Conventional discharge-measurement techniques are covered in Chapter 5 of this Manual. All of the principles described in this section are incorporated into computer programs designed to collate ADCP data collected during a cross-section traverse and compute discharge automatically and in real time.

ADCP discharge calculation methods

The ADCP could be used as a conventional current meter. If an ADCP were mounted to a boat the operator could position the boat at 30 or more verticals in the cross section. Velocities and depths could then be taken at each vertical, and the discharge calculated using the area/velocity method ($Q = AV$). Such a method would be only a slight improvement over the conventional stationary boat discharge-measurement technique described in Chapter 5.

The unique ability of the ADCP to continuously collect water-velocity profile data and ADCP-velocity (boat-velocity) data, relative to the bottom, lends itself to the use of a more sophisticated method of discharge-measurement integration. A velocity vector cross product at each depth bin in a vertical profile is calculated using ADCP-collected data. This cross product is first integrated over the water depth measured by the ADCP and then integrated, by time, over the width of the cross section. The following equations illustrate the integration method. The reader should try to envision them as descriptions of the discharge-measurement algorithm and mechanics.

Equation 6.4 is the general equation for determining river discharge through an arbitrary surface, s:

$$Q_t = \int_s \bar{V}_f \cdot \bar{n} \cdot ds \qquad (6.4)$$

where Q_t = total river discharge, in cubic meters per second; $\bar{V}_f$ = mean water-velocity vector, in meters per second; $\bar{n}$ = a unit vector normal to ds at a general point; and ds = differential area; in square meters.

The above is just a form of the familiar equation $Q = AV$ integrated over a cross section. For moving-boat discharge applications, the area s is defined by the vertical surface beneath the path along which the vessel travels. The dot product of $\bar{V}_f \cdot \bar{n}$ will equal zero when the vessel is moving directly upstream or downstream, and will equal $\bar{V}_f$ when the vessel is moving normal to $\bar{V}_f$. Both vectors are in the horizontal plane.

Because the ADCP provides vessel-velocity and water-velocity data in the vessels coordinate system, it is convenient to recast equation 6.4 in the following form (equation 6.5) (Christensen and Herrick, 1982).

$$Qt = \int_0^T\int_0^d \left(\left(\bar{V}_f \cdot \bar{V}_b\right) \cdot \bar{k}\right) dz \cdot dt \qquad (6.5)$$

where T = total cross-section traverse time, in seconds; d = total depth, in meters; $\bar{V}_b$ = mean vessel-velocity vector, in meters per second; $\bar{k}$= a unit vector in the vertical direction; dz = vertical differential depth, in meters; and dt = differential time, in seconds.

The derivation of this equation by Christensen and Herrick (1982) is summarized in Simpson and Oltmann (1993). The equation originally was formulated by Kent Dienes (R.D. Instruments, Inc., oral communication, 1986).

The cross-product algorithm is well suited to ADCP discharge-measurement systems. Translated into non-math terms, the above can be described as the cross product of the boat velocity and the water velocity first integrated over the cross-section depth and then integrated, by time, over the cross-section width.

The cross product part of equation 6.5, $\left(\bar{V}_f \cdot \bar{V}_b\right) \cdot \bar{k}$, can be converted to rectangular coordinates to facilitate plugging in boat- and vessel-velocity vectors:

$$\left(\bar{V}_f \cdot \bar{V}_b\right) \cdot \bar{k} = a_1 b_2 - a_2 b_1 \qquad (6.6)$$

where a_1 = cross component of the mean water-velocity vector, in meters per second; a_2 = fore/aft component of the mean water-velocity vector, in meters per second; b_1 = cross component of the mean vessel-velocity vector, in meters per second; and b_2 = tore/aft component of the mean vessel-velocity vector, in meters per second.

This is simply called the velocity cross product, which can be represented by the following equation:

$$f = a_1 b_2 - a_2 b_1 \qquad (6.7)$$

where f = the cross product of the water-velocity and boat-velocity vectors.

When the cross product is integrated over depth, the resulting value is in cubic meters per second per second. By substituting in the values for the beginning and ending times of each ensemble a discharge value is determined for each measured ensemble. The discharges for each ensemble then are summed to obtain total channel discharge. The mechanics of this operation require casting equation 6.5 into a form usable by the ADCP measurement software (equation 6.8):

$$Q_m = \sum_{i=1}^{N_s}\left[\int_0^d f_z dz\right] t_i \qquad (6.8)$$

where Q_m = measured channel discharge (doesn't include the unmeasured near-shore discharge), in cubic meters per second; N_s = number of measured discharge subsections; i = index of a subsection; d = depth of a subsection, in meters; f_z = value of cross product at depth z; dz = integrated vertical depth of subsection i, in meters; and t_i = elapsed travel-time between the ends of subsections i and $i - 1$, in seconds.

Estimating discharge in unmeasured parts of the ADCP profile

Problems arise when trying to implement the above summation for several reasons. To understand those reasons, the limitations of an ADCP water-velocity measurement must be considered. A zone near the upper part of the velocity profile, referred to as the blanking distance, is an unmeasured part of the velocity profile. The small ceramic element in the transducer is like a miniature gong. The large voltage spike of the transmit pulse bangs it like a hammer and the vibrations, as well as the residual signal, must die off before the transducer can be used to receive incoming signals. This means that incoming signals are not used if they are received during a short period after the signal is transmitted. This time period is equivalent to a distance travelled by the signal known as the blanking distance. The blanking distance plus the transducer draft (depth of the transducer face below the water surface) compose a part of the near-surface profile that is not measured by the ADCP.

Most transducers emit unwanted side lobes of acoustic energy at 30 to 40 degree angles off the main beam. Because of interference from these side lobes, up to 15 per cent (depending on the beam angle) of the water column near the bottom cannot be measured.

The combination of the effects of transducer draft, blanking distance and side-lobe interference yields a velocity profile that is incomplete. To properly compute discharge for each subsection, the cross-product values near the water surface and near the bottom must be estimated. As shown in Figure I.6.5, f values are not provided at or near the water surface and below a point equal to about 85 to 94 per cent of the total depth. The percent of unmeasured profile area depends on the beam angle. The unknown f values are labelled f_1 at the water surface and f_n at the channel bottom. The simplest method of estimating these f values would be to let f_1 at the

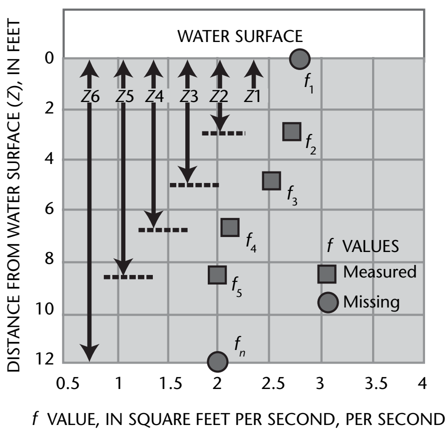

Figure I.6.5. Example velocity profile showing measured and missing *f* values

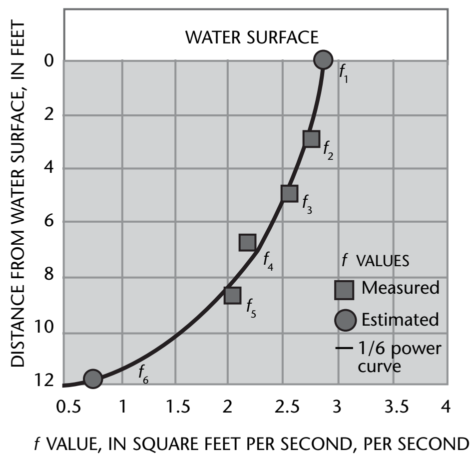

Figure I.6.6. Example velocity profile of one-sixth power-curve fit and typical *f* values

surface equal f_2 and to let f_n at the bottom equal the last measured value ($f_n - 1$) and approximate the integral using a logarithmic or power function. Attempts to use such methods did not prove satisfactory because of noisiness of the ADCP profile data.

A method using a one-sixth power law (Chen, 1989) eventually was chosen because of its robust noise rejection capability during most streamflow conditions. A full discussion of the one-sixth power law and its derivation can be found in Simpson and Oltmann (1993). The power law estimation scheme is an approximation only and emulates a Manning-like vertical distribution of horizontal water velocities. Different power coefficients can be used (one-half to one-tenth) to adjust the shape of the curve fit to emulate profiles measured in an estuarine environment or in areas that have bed forms that produce non-standard hydrologic conditions and provide alternate estimation schemes under those circumstances. Figure I.6.6 shows a one-sixth power curve drawn through *f* values that were illustrated in Figure I.6.5.

In cases where bidirectional flow exists (water is moving one direction at the surface and is moving the opposite direction near the channel bottom), the power-curve estimation scheme will not work. In these cases the unmeasured areas must be estimated using other methods.

In most cases, points at the top and the bottom of the profile can be estimated using the one-sixth power-curve estimation scheme. The estimated points then are used with the actual measured points to calculate a depth-weighted mean cross product for each ensemble. Discharge then can be calculated for each ensemble pair because the time between each ensemble is known.

The summation in equation 6.8 is accomplished by the following equation:

$$q_i = g_i t_i \tag{6.9}$$

where q_i = midsection discharge between measurement subsection i and subsection $i - 1$, in cubic meters per second; g_i = depth-weighted mean f value in measurement subsection i, in square meters per second per second; and t_i = vessel travel time between measurement subsection i and $i - 1$, in seconds.

Equation 6.8 is used to calculate the main channel discharge by summing all Q values (equation 6.9) collected during the cross-section traverse. Unfortunately, before the total channel discharge can be calculated, two other areas need estimation schemes (Figure I.6.7).

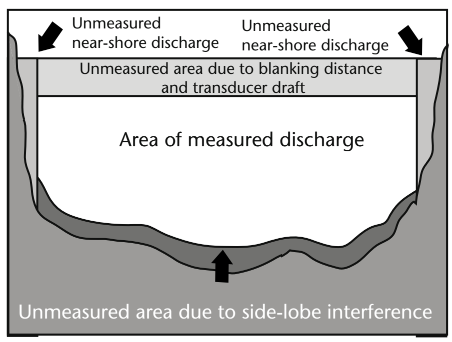

Figure I.6.7. Unmeasured areas in a typical ADCP discharge measurement cross section

Estimating discharge near the channel banks

The power-curve fitting scheme estimates values in the areas at the top and bottom of the profile (blanking/draft distance and side-lobe interference area), but, because of these and other ADCP depth limitations, shallow areas near the edges of the riverbank cannot be measured. The near-shore areas are estimated using a ratio interpolation method presented by Fulford and Sauer (1986), which can be used to estimate a velocity at an unmeasured location between the riverbank and the first or last measured velocity in a cross section. The equation for the estimate is equation 6.10:

$$\frac{V_e}{\sqrt{d_e}} = \frac{V_m}{\sqrt{d_m}} \qquad (6.10)$$

where e = a location midway between the riverbank and first or last ADCP-measured subsection; V_e = estimated mean velocity at location e, in meters per second; V_m = measured mean velocity at the first or last ADCP-measured subsection, in meters per second; d_e = depth at subsection e, in meters; and d_m = depth at the first or last ADCP-measured subsection, in meters.

Fulford and Sauer (1986) defined d_m and V_m as depth and velocity at the centre of the first or last measured subsection and not the near-shore edge of the subsection, as presented in equation 6.10. However, because the ADCP subsections purposely are kept very narrow at the start and finish of each measurement, the differences between the two applications are not significant (Simpson and Oltmann, 1993). With this method, discharge can be estimated by assuming a triangular discharge area between subsection m and the riverbank, and the equation 6.11:

$$V_e = 0.707 V_m \qquad (6.11)$$

Remembering that Q = AV, discharge in the estimated area can be calculated by equation 6.12:

$$Q_e = \frac{0.707 V_m L d_m}{2} \qquad (6.12)$$

where Q_e = estimated edge discharge, in cubic meters per second; and L = distance to the riverbank from the first or last ADCP-measured subsection, in meters.

The discharge-measurement software calculates depth, d_m, from the average of the depths measured on all four beams. The distance ,L, to the riverbank from the first or last discharge measurement subsection is provided by the system operator using estimation techniques described in the section on discharge-measurement techniques.

The triangular ratio-interpolation method works well in parabolic-shaped natural channels. However, it does not work well in rectangular concrete channels or natural channels with non-standard slopes near the banks. In these cases, a bank-slope coefficient can be used to properly depict the channel-bank geometry.

Determination of total river discharge

Using all the methods and equations described in the previous sections, total river discharge, Q_t, can be calculated by the following equation:

$$Q_t = Q_m + Q_{el} + Q_{er} \qquad (6.13)$$

where Q_m = total channel discharge [the sum of all q_i values collected during the discharge measurement traverse (equation 6.9)], in cubic meters per second; Q_{el} = near-shore discharge estimate on the left side of the channel, in cubic meters per second; and Q_{er} = near-shore discharge estimate on the right side of the channel, in cubic meters per second.

Based on the above principles a computer program is designed to collate ADCP data collected during a cross-section traverse and compute discharge in real time. In summary, the ADCP software calculates a water/boat velocity vector cross product in each bin before integrating the cross products over the sub-section depth. The resulting subsection discharges then are summed over the width of the cross section. Because the discharge cannot be measured near the water surface and near the channel bed, cross products in these unmeasured portions of the channel cross section usually are estimated using a one-sixth power-curve estimation scheme. Discharges in the unmeasured portions of the cross section near the edges of the river bank are estimated using a ratio-interpolation method.

ADCP bottom tracking

An ADCP must accurately compute boat velocity to compute water velocity, channel area and discharge. The ADCP does this by using the Doppler shift of reflected pings from the streambed – commonly referred to as bottom tracking. Bottom tracking is done by proprietary firmware schemes built into the ADCP by the manufacturer; however, all such schemes must rely on the assumption that the bottom reflection obeys basic laws of physics. These principles of bottom tracking should be known by the ADCP operator to properly evaluate bottom tracking under variable field conditions.

When an acoustic signal strikes the river bottom, the reflected signal is normally orders of magnitude stronger than the reflected signal from scatterers in the water mass. Because of this, the standard deviation of the bottom-track velocity measurement is about 10 times less than the water-mass velocity measurement.

In the case of heavy sediment load, which causes high absorption and scattering of the acoustic signal, the weakened bottom reflection from deep water cannot activate a detection threshold. In some cases, when this happens, the ADCP firmware is programmed to try other, more robust bottom-tracking modes before flagging the data as bad.

Another problem occurs during periods of high flow when heavy sediment loads are moving on or near the channel bed. A systematic bias in discharge measurements made with an ADCP is attributed to the movement of sediment near the streambed – an issue widely acknowledged by the scientific community (Mueller, 2006). This systematic bias leads to an underestimation of measured velocity and discharge. The integration of a Differential Global Positioning System (DGPS) to track the movement of the ADCP can be used to avoid the systematic bias associated with a moving bed. DGPS systems, however, cannot provide consistently accurate positions because of multipath errors and satellite signal reception problems on waterways with dense tree canopy along the banks, in deep valleys or canyons and near bridges. An alternative method of correcting for the moving-bed bias, called the loop method, based on the closure error resulting from a two-way crossing of the river was investigated by the Mueller (2006). The loop method basically involves starting the ADCP at a known, marked starting point, then crossing the stream completely twice – at the end of the second crossing the ADCP is returned to its starting point. A moving streambed will make it appear that the ADCP has moved upstream from the starting point. This apparent distance can be used to estimate the total magnitude of the moving bed for the measurement site and is used to adjust the discharge measurement accordingly. The uncertainty in the measured mean moving-bed velocity caused by non-uniformly distributed sediment transport, failure to return to the starting location, variable boat speed and compass errors were evaluated using both theoretical and field-based analyses. The uncertainty in the mean moving-bed velocity measured by the loop method is approximately 0.6 cm s^{-1}. Use of this alternative method to correct the measured discharge was evaluated using both mean and distributed correction techniques. Application of both correction methods to 13 field measurements resulted in corrected discharges that were typically within 5 per cent of discharges measured using DGPS.

6.2.5 **ADCP measurement configuration**

Before an ADCP is used to make a discharge measurement the user must configure the instrument for the environment expected at the site, such as water velocities and maximum depths. For example, the depth cell size and number of depth cells to be collected must be set such that the ADCP profiles all the way to the bottom at the point of maximum depth. The ADCP manufacturers offer software that is used to configure the ADCP and collect and evaluate measurement data.

Broad-band ADCPs manufactured by Teledyne RD Instruments have several water-measurement and bottom-measurement modes. SonTek ADCPs utilize a single mode for all measurements. The RD Instruments ADCP modes are chosen based on how fast, slow, shallow, or deep the water is and current shear. Several environmental factors may play a part in the choice of measurement modes. Before the advent of the current generation of ADCP software, the broadband ADCP user had to configure an ADCP using direct commands which required knowledge of the command format and allowed for the possibility of configuration errors because of mistyping or transposing numbers. The current generation of broadband ADCP software has a configuration wizard option that will configure the ADCP to be optimized for the conditions expected during the measurement. The user enters parameters such as expected maximum depth, maximum expected water velocity and maximum expected boat speed. The wizard verifies that the required fields have been entered by the user and uses these values to generate a configuration file for the measurement. The appropriate direct commands are specified based on rules supplied to RD Instruments, including depth cell size, number of depth cells, ambiguity velocity and water mode. The wizard also scales the data collection chart properties accordingly based upon the entered values. It is recommended that the configuration wizard always be used to configure RD Instruments ADCPs to prevent configuration errors. SonTek ADCPs utilize a single water track and bottom track mode, thus the configuration of these units is not as complex and a configuration wizard is not necessary.

6.2.6 ADCP hardware and equipment

The following equipment is needed to measure river discharge with an ADCP:

(a) ADCP system:
  - (i) Pressure case and transducer assembly;
  - (ii) Power supply and communications interface;
  - (iii) Discharge-measurement software and documentation.

(b) Ancillary equipment:
  - (i) Measurement platform or vessel;
  - (ii) ADCP mounting assembly;
  - (iii) Laptop computer;
  - (iv) Range finder or method for estimating distance to shore;
  - (v) Computer data-storage media (such as flash-memory card or CD-ROM) with sufficient storage space for making temporary backup copies of all field data files;
  - (vi) Temperature probe for measuring water temperature;
  - (vii) Radio modems and associated accessories for tethered and remote boat deployments.

The following sections will describe the various equipment components.

ADCP pressure case and transducer assembly

ADCPs of both manufacturers have similar physical characteristics. The transducers and electronics are contained in the same cylindrical enclosure. The diameter and length depends upon the model of the ADCP. ADCPs with lower acoustic frequencies are usually larger and heavier as they employ larger transducers. A convex or concave transducer assembly on one end of the cylindrical canister employs an orthogonal Janus beam aiming pattern with the three or four transducer beams angled 20° (or optionally 30°) outward from the centre axis of the assembly. The transducer assembly diameter depends on the frequency of operation and configuration. Figures I.6.8 and I.6.9 shows typical ADCPs.

The newest ADCP as of this writing, the RD Instruments Streampro, has perhaps the most radical departure from the typical ADCP physical configuration. The Streampro has a very small transducer head that is separated from the electronics case by a short cable. The entire Streampro is designed to be deployed from a small floating platform.

Figure I.6.8. Two Acoustic Doppler Current Profilers, an RD Instruments Rio Grande (left) and a SonTek model (right)

Figure I.6.9. An RD Instruments Streampro ADCP mounted on a small tethered boat

All ADCPs require a communications and power cable that attaches to the instrument pressure case. The cable will normally terminate in power connection terminals and also in an RS-232 connector for PC computer connection. Typically the cable has an inline fuse to prevent power surge damage to the ADCP. Older model narrow band and BB-ADCPs required the use of a deck box interface for power and communications; today's commonly used models do not require a deck box.

ADCP discharge measurement software and documentation

To measure river discharge the ADCP system is controlled by discharge-measurement software. The manufacturer generally provides software and documentation that are essential for the proper operation and setup of the BB-ADCP. Software and some user manuals are provided as computer diskettes.

Measurement platforms and vessel requirements

Every measurement site has unique features that dictate the type of ADCP deployment platform. Site features may include hydraulic characteristics such as water depth and access considerations such as the presence of boat ramps, bridges or cableways. The three types of ADCP deployment platforms are manned boats, tethered boats and remote-controlled boats. This section provides recommendations for deployment platform mounts, waterproof enclosures and radio-modem telemetry.

Manned Boats

Vessel requirements for manned boats will vary, depending on the size and flow rates of the rivers and streams. For example, if measurements are in large rivers or estuaries, the BB-ADCP may be mounted on a 9- to 15-m vessel with an enclosed cabin. If, on the other hand, discharge measurements are on small, slow-moving rivers, the vessel of choice may be a 5- to 6-m skiff. For very small streams with minimal wave action the BB-ADCP may be mounted on an inflatable boat, such as a Zodiac.

The proper boat choice will depend on the topography and hydrology of the area of interest; however, it is best to have several alternative vessels for discharge measurement use. The USGS in California uses three boats for ADCP discharge measurements. Figure I.6.10 shows a 30-m vessel with a side-swing mount. Using this configuration, measurements in the estuary can be obtained under all but the worst of conditions.

Figure I.6.11 shows an ADCP mounted on a 6-metre Boston Whaler. This vessel can be used in estuaries, rivers, and in river tributaries. A large 150-horsepower engine is used to get from place to place quickly when making discharge measurements in an estuary or river delta. A smaller engine is used when making discharge measurements in small rivers and slow-moving water. This vessel can be rigged with canvas for inclement weather.

Figure I.6.12 shows a side-swing mount on a 4.5-metre Boston whaler. This vessel is easily towed on a trailer and is used when measuring small rivers. The main engine is a four-cycle, 45-horsepower unit that can be idled at low speeds for discharge measurement. This configuration is used mainly in fair weather; however, canvass also can be rigged to make the vessel usable in inclement weather.

When making discharge measurements in narrow rivers, a trolling plate can be used on the main

Figure I.6.11. Side-swing mount on a 6-metre Boston Whaler for an ADCP

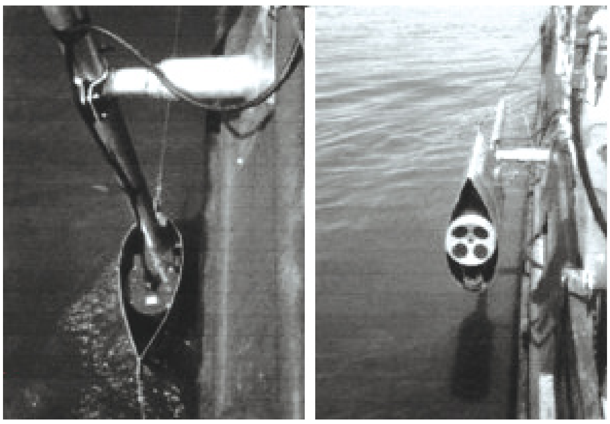

Figure I.6.10. Two views of an ADCP profiler side-swing mount on a 30-metre vessel

Figure I.6.12. Side-swing mount on a 4.5-metre Boston Whaler for an ADCP

engine or an electric trolling motor can be used to slow the vessel. For accurate measurements in very slow-velocity water, this vessel can be pulled on a tagline.

In Sweden, BB-ADCP operators have used a small, inflatable dinghy when making discharge measurements. The advantage of this type of vessel is that launch ramps are not needed. The dinghy can be inflated on the riverbank and the equipment set up for use in less than 30 minutes.

It is imperative that not only the correct boat be used for any given set of river and weather conditions, but also that boat operators be properly trained. Correct operation of the boat is vital to obtaining high-quality discharge and velocity measurements.

Another method for deploying an ADCP from a manned boat is to float the ADCP in a tethered boat (see next section) alongside the manned boat. The tethered boat can be attached via ropes to a boom that extends away from the manned boat hull. This arrangement has the advantage that the ADCP draft is not affected by the manned boat dynamic or static loading. It has a disadvantage in that rough conditions or quick manoeuvres can cause the tethered boat to swing quickly from side to side, which could adversely affect measurement quality.

Tethered Boats

A tethered boat is a small boat usually less than 5 ft long attached to a rope that can be deployed from a bridge, a fixed cableway or a temporary bank-operated cableway. The tethered boat should be equipped with an ADCP mount that meets all of the specifications outlined in the previous section on manned boats. The tethered boat should also contain a waterproof enclosure capable of housing a power supply and wireless radio modem for data telemetry. A second wireless radio modem attached to the field computer enables communication between the ADCP and field computer without requiring a direct cable connection. The radio modems should reliably communicate with the ADCP using the ADCP data-acquisition software, have a rugged, environmental housing, operate on a 12-volt direct current (DC) power supply and have at least 38 400 baud data-communication capability to maximize ADCP data throughput (Rehmel and others, 2002). Rehmel and others (2002) describe the development of a prototype tethered platform, a project to refine the platform into a commercially available product and tethered-platform measurement procedures.

Tethered ADCP boats have become a common deployment method (Figure I.6.13). Certain considerations need to be made when making tethered ADCP boat measurements. Tethered boats are used in a variety of settings, but primarily from the downstream side of bridges for convenience. Bridge piers can cause excessive turbulence during high streamflow, especially if debris accumulations are present on the piers and the piers are skewed to the flow. The effect of bridge pier-induced turbulence may be reduced by lengthening the tether to increase the distance between the bridge and the tethered boat. Attention should be paid to the cross section to ensure that there are no large eddies that could cause flow to be non-homogeneous. Possible alternatives to measuring off the downstream side of bridges include bank-operated cableways or having personnel on each bank with a rope attached to the platform, pulling it back and forth across the river. Bank-operated cableways may be as simple as a temporary rope and pulley apparatus (Figure I.6.14) or may involve the use of a small temporary cableway with

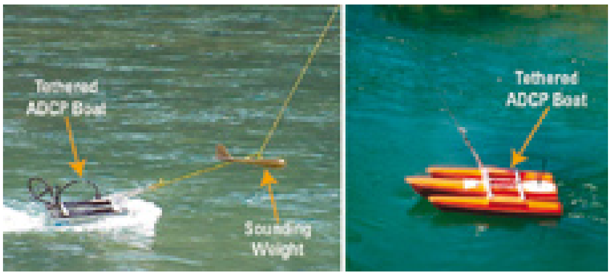

Figure I.6.13. Examples of tethered ADCP boats used for making discharge measurements

Figure I.6.14. A temporary bank-operated cableway for making ADCP measurements with a tethered ADCP boat

a motorized drive for towing the tethered boat back and forth across the stream. In 2004 remote-controlled rovers have been developed for cableways. These rovers can be carried from one streamflow-gauging station to another and, once mounted on the cableway, can be used to winch up the tethered boat and drive the boat back and forth at a user-controlled speed.

When the water velocity is slow (usually less than 0.15 m s^{-1}) it may become difficult to control the tethered boat. This lack of control may be exacerbated by wind, which could push the boat in an undesirable direction. Boat handling can be improved by attaching a floating sea anchor to the back side of the boat to increase the effect of the current and its pull on the tether. Make sure that this anchor is far enough behind the boat so as to not disturb the flow and potentially bias the velocity measurements. An example of a sea anchor deployed on a tethered boat is shown in Figure I.6.15.

When the water velocity is fast (usually greater than 1.5 m s^{-1}) it is not uncommon for a tethered boat to be pitched upward at the bow. This increased pitch is caused by increased vertical tension on the tether in faster flows, hull dynamics and an incorrect setting of the angle for the bail, for those boats equipped with a rigid bail. The bail connects the tether to the boat and can be either a rigid design or a more flexible rope bail. Large pitch angles may introduce some bias in depth measurements and should be minimized as much as possible. Experience in handling tethered boats has shown that adding a sounding weight on the tether near the location where it is tied to the boat (see Figure I.6.13) will help decrease the pitch angle. In addition, increasing the length of the tether helps reduce the pitch angle.

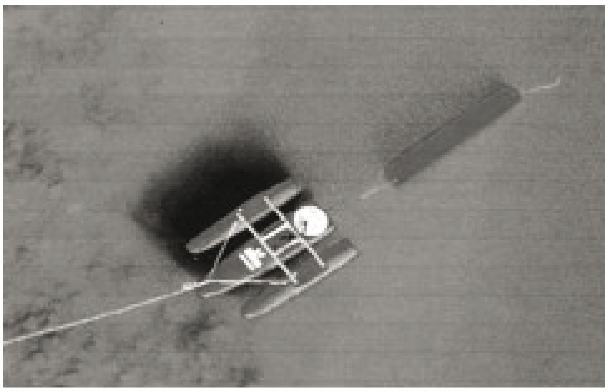

Figure I.6.15. Tethered ADCP boat with a sea anchor attached

The tether line should be visible from the water surface to minimize the risk of collision with river traffic. Orange plastic flags tied along the tether will enhance its visibility. The operator should also be capable of releasing the tether quickly in case the boat becomes entangled in debris or collides with river traffic. Do not wind the tether around the hand to hold the boat as this action is a safety hazard. Standard safety practices, site-specific traffic safety plans and local highway traffic regulations should be followed.

For tethered and remote-controlled boats it is possible to lose control of the boat. For example, a boat tether or tether attachment point could break. For this reason it is good practice to carry a retrieval system such as a small grappling hook attached to a length of rope. It may be possible to catch a wayward boat with this system from a bridge or from shore. It is recommended that ADCP operators using tethered and remote-controlled boat deployments have a contingency plan for retrieving the boat in the event of a failure that causes a loss of boat control. An example of a contingency plan would be to carry a small manned boat that could be quickly and safely launched to retrieve the tethered or remote-controlled boat.

Remote controlled boats

Unmanned, remote-controlled ADCP boats allow the deployment of ADCPs where deployment with either a manned boat or tethered boat may not be feasible or ideal. Similar to (but smaller than) manned boats, a remote-controlled boat has self-contained motors and a remote-control system for manoeuvring the boat across the river. Unlike the tethered boat, the remote-controlled boat has no tether restraints. Although remote-controlled boats have an increased risk of equipment loss because of potential loss of boat control they provide the ability to launch a boat without a boat ramp and to collect data away from bridge effects (for example, upstream of a bridge) or at sites where no bridge or cableway is present. See Figure I.6.16 for an example of a remote controlled boat and ADCP.

A remote-controlled boat ADCP mount should meet all mount specifications listed for manned boats. The remote-controlled boat also should contain a waterproof enclosure capable of housing a power supply, a radio modem and the control radio. Radio modems are used for data telemetry between the remote-controlled boat and field computer; the radio modems should have the capabilities described for tethered boat deployments.

Figure I.6.16. Radio controlled ADCP platform, 3.6 metres boat

The same operational guidelines regarding speed and manoeuvring for manned boats also apply to remote-controlled boats. Proper control of a remote-controlled boat requires practice. The operator should be familiar with remote-controlled boat operation prior to using this deployment technique in high flows. Regular maintenance of the boat and control radios is critical to ensure reliable operation.

ADCP mounting assemblies

ADCPs are typically mounted on either side of manned boats, off the bow or in a well through the hull. The ideal mounting location for an ADCP is a well through the hull midway between the gunwales and approximately three-fifths of the distance from the bow to the stern. Boat and ADCP operators, however, are often reluctant to install a well in many of the boats that are commonly used for river discharge measurements. Advantages and disadvantages for mounting locations on manned boats are listed in Table I.6.1.

The ADCP should not be mounted in close proximity to any object containing ferrous metal or sources of strong electromagnetic fields, such as portable generators, in order to minimize ADCP compass errors. A good rule of thumb is that an ADCP should not be mounted any closer to a steel object than the largest dimension of that object. However, and there are large variations in the magnetic fields generated by different metals. Even stainless steel varies appreciably in the amount of ferrous material contained in the steel.

ADCP mounts for manned boats should:
(a) Allow the ADCP transducers to be positioned free and clear of the boat hull and mount;
(b) Hold the ADCP in a fixed, vertical position so that the transducers are submerged at all times while minimizing air entrainment under the transducers;
(c) Allow the user to adjust the ADCP depth easily;

Table I.6.1. Advantages and disadvantages of ADCP mounting locations on manned boats

Mounting Location	*Advantages*	*Disadvantages*
Side of boat	Easy to deploy	Moderate chance of directional bias in measured discharges with some boats and flows
	Mounts are easy to construct and are adaptable to a variety of boats	Possibly closer to ferrous metal (engines) or other sources of electromagnetic fields (EMF)
	ADCP depth measurement can be easily obtained	Moderate-low risk of damage to ADCP from debris or obstructions in the water
		Susceptible to roll-induced bias in ADCP depths
Bow of boat	Minimizes chance of directional bias in measured discharges	Increased risk of damage to ADCP from debris or obstructions in the water
	Mounts relatively easy to construct	More difficult to measure ADCP depth
	Usually far from ferrous metal or electromagnetic fields	Less susceptible to pitch/roll-induced bias in ADCP depths, except at high speeds or during rough conditions (waves)
Well in center of boat	Protected from debris and obstructions	Often requires special modifications to boat
	Accurate depth measurements possible	
	Least susceptible to pitch/roll-induced bias in ADCP depths	

(d) Be rigid enough to withstand the force of water caused by the combined water and boat speed;
(e) Be constructed of non-ferrous parts;
(f) Be adjustable for boat pitch-and-roll; and
(g) Be equipped with a safety cable to hold the ADCP in the event of a mount failure.

Versatile ADCP mounting brackets are illustrated in Figure I.6.17, a side-mounted ADCP on a 5.5 metre long work boat, and in Figure I.6.18, a bow-mounted ADCP on 4.9 metre work boat. Note that these are just two examples of mounting an ADCP. There are numerous and innovative other methods that have been and are being used.

Figure I.6.17. Side-mounted ADCP mounting brackets

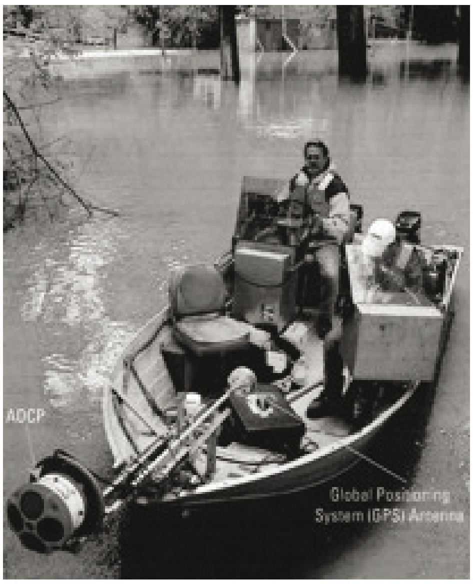

Figure I.6.18. Bow mounted ADCP mounting brackets and ADCP

Laptop computer

The computer selection for running the ADCP software is very important. Because of the amount of data processing required, the computer must have an i286 Central Processing Unit (CPU) or equivalent (an i386, i486, or Pentium CPU is desirable) and also must be IBM compatible. The computer at least must be capable of displaying Extended Graphics Array (EGA) compatible graphics. Because of the amount of data storage required, a hard drive (non-volatile ram drive) should be used with at least 20 megabytes (Mb) of available storage space.

The computer screen display should be visible in direct and diffuse sunlight. For routine collection of streamflow data, a rugged laptop computer is desirable. Several manufacturers now produce laptops with antiglare screen coatings, shock-mounted hard drives and water-resistant keyboards and access panels. Standard laptops have minimal protection from the elements; rain and dust protection are not provided. Do not rely on the internal laptop battery to provide power for an all-day measurement session because most laptop batteries will not last beyond about 3 hours and many will not last 1 hour before requiring recharge.

The data-processing computer is connected to the ADCP through a serial connection on the back of the deck unit. Normally, this serial connection is RS-232c; however, for cables longer than about 60 metres RS-422 protocol should be used. RS-422 protocol requires the use of a separate converter box as well as changes in the internal ADCP switch settings (R.D. Instruments, Inc., 1995). The RS-232c serial connection is provided by way of a standard IBM personal computer 9-pin female to 25-pin male adaptor cable which is available in most computer shops. A null-modem adaptor is not needed.

The computer must be protected from direct sunlight and heat during data collection. The LCD screen turns dark and unusable if it remains in direct sunlight too long. The computer also can be damaged by the heat of direct exposure to the sun. One solution is to place the computer inside an empty cooler that is turned on its side. The cooler shades the computer and LCD from direct sunlight and also protects the computer from splashing water.

To protect the measurement data from computer loss, malfunction or damage separate electronic storage media should be used to backup all measurement files immediately following a measurement. Examples of media include DVD-ROMs and USB memory sticks.

Range finder or method for estimating distance to shore

Edge discharges are estimated in the transect software using a technique similar to that used when making conventional discharge measurements. The unmeasured area between the boat and the river edge is estimated using the last measured mean velocity, the last measured depth, and the distance from the boat to shore. The algorithm assumes a triangular-shaped area for this estimate, as described in a previous section of this Manual. If the river channel is rectangular, these edge estimates can be doubled and an adjustment made for the roughness of the edges.

Estimating the distance to shore can be done with the naked eye; however, such distance estimates are almost always short of the true distance. The reason for this is unclear, but is possibly due to lack of visual clues between the boat and shore. The most reliable way of ensuring accurate edge estimates is to set buoys out from the shore at known distances measured with a tape or distance meter. The transect software is then started and stopped at these buoys. This method is not always possible when large numbers of discharge measurements are needed at different locations within a short time period.

There are several types of distance-measurement devices, such as optical, sonic and infrared lasers that have been used to increase the accuracy of the edge estimates without the need to set buoys or onshore devices. Good results have been obtained with inexpensive, optical range finders that use parallax and a focusing device to estimate distance. The operator identifies a rock or object at the stream edge and then rotates a knob to converge two images of the object. The distance is then read from a scale on the device. This method requires a little practice but with properly calibrated range finders acceptable accuracy can be obtained up to about 180 metres.

Sonic devices usually require a vertical wall for a signal return. Riverbank edges generally do not have topographies that enhance acoustic reflections. To be usable these devices need special sonic targets (corner reflectors) placed on the riverbanks. These devices can be useful if the operator is able to deploy the sonic targets at the cross-section edges.

Laser devices are cumbersome and more delicate than the other range finders but can be more accurate over longer distances. They can be used without targets up to about 75 metres and up to hundreds of meters with targets. The major drawback to these devices is their cost and durability. They require precision optics, which are delicate and easily damaged.

6.2.7 ADCP discharge measurement procedure

The following sections describe the pre-measurement, measurement traverse and post-measurement requirements for making an ADCP discharge measurement.

Site selection

General guidelines for selection of an ADCP measurement section are listed below:

(a) Desirable measurement sections are roughly parabolic, trapezoidal or rectangular. Asymmetric channel geometries (for example, deep on one side and shallow on the other) should be avoided if possible, as should cross sections with abrupt changes in channel-bottom slope. The streambed cross section should be as uniform as possible and free from debris and vegetation;
(b) Measurement sections with velocities less than 0.10 m s^{-1} should be avoided if it is possible to do so, and an alternative measurement location is available. Although measurements can be made in low velocities, boat speeds must be kept extremely slow (if possible, less than or equal to the average water velocity), requiring special techniques for boat control;
(c) Depth at the measurement site should allow for the measurement of velocity in two or more depth cells at the start and stop points near the left and right edges of water;
(d) A site with very turbulent flow, for example evidenced by standing waves, large eddies and non-uniform flow lines, should be avoided. This condition is often indicative of non-homogenous flow and violates one of the assumptions required for accurate ADCP velocity and discharge measurements;
(e) Measurement sections having local magnetic fields that are relatively large as compared to the Earth's magnetic field should be avoided. Large steel structures, such as overhead truss bridges, are a common source for these large local magnetic fields and may result in ADCP compass errors;
(f) When using DGPS avoid locations where multi-path interference is possible (signals from the

satellites bounce off structures and objects such as trees along the bank or nearby bridges or buildings) or where reception of signals from GPS satellites is blocked. It may be possible to make valid measurements in sections that violate one or more of the above guidelines, but whenever possible locate and use a better measurement section. If a better section is not available the loop method documented by Mueller (2006) will likely provide better results than DGPS for correcting moving streambed biases.

Pre-measurement field procedures

After the boat is launched and the ADCP equipment is set up there are a number of items that should be checked before beginning the actual traverse for the measurement:

(a) Care should be taken to obtain an accurate transducer depth measurement. If a side-swing mount is used the weight of the operator(s) can cause an unwanted pitch of the ADCP vessel. This pitch angle may cause an erroneous reading of the ADCP depth. Determine the unpitched transducer depth and enter it on the log sheet and in the configuration file;

(b) The operator should note any conditions relevant to the discharge measurement on the discharge-measurement log sheet. Wind, bidirectional flow, eddies, standing waves, passing boats and sediment conditions are just some of the things that should be noted for later analysis of the discharge measurements;

(c) Synchronize the computer, ADCP and operator watch times. In most cases the ADCP clock should be set to agree with the recorder time at the streamflow station;

(d) The ADCP internal magnetic compass should be calibrated. This is especially important if a DGPS is being used for the measurement;

(e) Perform a short reconnaissance of the cross section to determine shallow areas and the shape of the cross section so that unmeasured areas near the bank can be characterized. If the cross section is unsuitable for any reason (too shallow in places, for example), select another measurement cross section;

(f) If buoys are used to aid the estimation of edge distances they should be deployed and the distance to shore from each buoy should be measured and noted on the discharge-measurement log sheet;

(g) If range finders are used to determine edge distances they should be checked for proper calibration;

(h) Record weather, hydrological and other physical phenomena pertinent to the discharge measurement on the discharge-measurement note;

(i) Make an independent water temperature reading and compare this to the water temperature measured by the ADCP. The two temperature readings should agree within 2°C. Temperature errors greater than this can cause a bias in the ADCP computed discharge;

(j) Make a moving bed test unless it has been previously verified that the streambed is stationary. One method to make a moving bed test is to anchor the boat near mid-stream, and record ADCP data for 10 minutes, using bottom track as the boat-velocity reference. When a moving bed is present a stationary boat will appear to have moved upstream. If it is not possible or not safe to anchor near mid-stream, use two reference points on shore to keep aligned during the 10-minute test. In either case, if the moving bed velocity exceeds 1 per cent of the mean velocity of the stream then DGPS should be used instead of bottom tracking to compute boat velocity;

(k) If the average salinity in the measuring section is greater than zero it should be measured and entered into the ADCP data-collection software;

(l) Make sure that the right configuration file has been loaded properly into the transect software;

(m) Make sure that the power supply has been turned on and the ADCP has been awakened.

Starting the cross-section traverse

Boat-maneuvering techniques for discharge measurements when using the ADCP and the transect software do not require the precision once needed for conventional moving-boat discharge measurements. However, there are some basic maneuvers that improve accuracy and allow smooth transitions between measurements.

For a typical measurement the operator must maneuver the boat close to, and parallel with, the riverbank. The boat should be maneuvered in as close as possible to the bank without bottoming out the boat motor propeller or the ADCP transducer assembly. Performing this maneuver takes practice.

While the boat is somewhat stationary the operator should start the transect software and set the acquire display to tabular velocity mode (initial setting). The tabular mode setting is optional as the transect software will collect data in any display mode. However, the tabular mode enables the operator to determine if there are an adequate number of depth cells that have good discharge before starting the measurement traverse. This capability provides the greatest advantage over the other display modes, especially when starting and ending the discharge measurement.

At this point the operator is beginning the discharge measurement and must accomplish several tasks quickly:

(a) The distance to shore must be estimated by some means, as described previously, and recorded;
(b) The operator must turn the ADCP data recording off and must start the ADCP pinging;
(c) While looking at the tabular display the operator must verify that the ADCP is collecting at least two good bins of velocity data;
(d) When the operator is satisfied that accurate data are being collected and the boat is in the correct position to start the discharge measurement, he must start the recording and wait until two good ensembles have been collected. During this period (about 5 seconds), the boat should be barely moving toward center channel.

Procedures for the cross-section traverse

When the transect software begins collecting data, the operator should verify that the message ADCP PINGING appears near the upper right of the monitor display and that a transect recording file is opened (file name visible at the lower right of the transect software screen). At this time the boat should have just begun traversing the river cross section. The operator must quickly scan the initial ensemble display to determine if everything is operating correctly. Signs of improper operation are flagged "bad" in all columns and rows or "bad" in all rows of an individual column. The display columns correspond to north velocity, east velocity, vertical velocity, error velocity and percent good. If the incoming data appears correct the operator should continue the transect with the same course and a slightly increased speed (approximately that of the water or slightly less). As the boat enters faster flow, engine revolutions per minute and boat heading may have to be adjusted slightly to enable a smooth traverse.

Uniform boat speed during a transect is more important than steering a straight course. The course may be allowed to change slightly and slowly, if necessary, during the traverse. However, rapid course and boat heading changes can introduce errors into the measurement. The key element here is to DO EVERYTHING SLOWLY, including course changes, the speed of the vessel itself and even the speed of persons moving around onboard the measurement vessel. Sharp accelerations of the measurement vessel in any direction should be minimized or eliminated.

Ending the cross-section traverse

As the vessel approaches the opposite edge of the measuring section the boat should be slowed by gradually changing the heading to a more upstream direction and slowing the boat motor. The boat then can creep toward the bank, a process called crabbing. When the operator decides that the approach cannot be continued further an edge value is determined and the transect is ended.

At the end of a cross-section traverse the boat heading is changed just enough so that the boat stops its bank-ward movement and begins to slowly creep in the direction of center channel. At this point the operator may begin another transect and obtain a starting distance value. Slow crabbing at the start and finish of each cross section works better than nosing the boat into the bank and then backing away. The ADCP should not be allowed to pass over the boat propeller vortex during the discharge measurement. Entrained air in the vortex will cause failure of the ADCP bottom track and result in lost ensembles.

The operator should practice the above described technique a few times before an actual transect session is begun so that all personnel become accustomed to the flow conditions at this location. The more practice you have in making these measurements the more uniform will be the measurement results. When maneuvering near the riverbank, a large heading adjustment away from the bank (swinging the bow away from the bank) should not be made because it will bring the stern (and, therefore, the engine prop and shaft) into contact with the bank or bottom. This maneuver can produce highly undesirable results.

Alternate techniques for low flow conditions

The above described technique works only when there is sufficient stream velocity to allow the boat to crab. At very low stream velocities (less than 10 cm/s); the boat must be turned VERY SLOWLY to enable a direct crossing of the stream. In some cases the best approach is to raise the engine and pull the boat slowly (at the stream velocity or less) across the stream with ropes or a tag-line. A winching system can also be devised to move the boat across a stream very slowly.

To gain reasonably accurate measurements in very slow moving water, special setup commands must be used to increase the measurement pulse lag times. The use of these long lag times causes the ambiguity velocity to be very low. Therefore, the boat must be moved across the stream very slowly.

Post-measurement requirements

An assessment of the discharge measurement should be made after completion of the transects composing the measurement. A thorough review of all measurement data often is not practical in the field but a cursory review of the measurement should be made in order to assign a preliminary quality rating to the measurement and to make certain that there are no critical data-quality problems with specific transects. If all transects were collected at the same measurement section, the transect widths and discharges in the measured (middle) and unmeasured (top, bottom, and edge) sections should be consistent. If transect widths or discharges are not consistent with the other transects the transect data should be scrutinized to determine if a critical data-quality problem occurred.

If a critical data-quality problem is identified data from that transect should not be used in the computation of discharge. A new transect should be collected, starting from the same side as the discarded transect, if flow conditions remained steady. If the flow has changed a new transect series should be collected (a minimum of four transects if the flow is stable when the new transects are collected). It is emphasized that a transect should be discarded only if a critical data-quality problem is identified and documented on the field note sheet.

The measured discharge should be plotted on the rating curve for that streamflow-gauging station and the percent difference from the stage-discharge rating computed. 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, a second discharge measurement to check the original measurement should be made.

For ADCPs, the following steps should be followed when making a check measurement:

(a) Start as if making a completely new discharge measurement;
(b) Turn the ADCP off and then power it back on;
(c) Re-measure the ADCP depth;
(d) Perform a diagnostic test;
(e) Calibrate the compass and evaluate the compass;
(f) Re-run the software for configuring the ADCP;
(g) If available, and conditions permit, consider making the measurement using a different mode and/or in a different cross-section location;
(h) Adhere to the preceding description for pre-measurement routine.

The measured discharge from the check measurement should then be plotted on the rating curve and the percent difference from the stage-discharge rating computed in the field.

Site-specific conditions, such as turbulence, eddies, reverse flows, surface waves, moving bed, high sediment concentration and proximity of the instrument to ferrous objects, should be noted under the appropriate sections on the ADCP measurement note sheet and used in assigning a quality rating for the measurement (Lipscomb, 1995).

All of the files in a discharge-measurement series should be identified uniquely. Immediately after completion of a measurement, all files including raw data files, configuration files, instrument test files, compass calibration files and any electronic measurement forms should be backed up on a non-volatile media such as CD-ROM, flash-memory cards or USB drives and stored separately from the field computer. The purpose of this backup is to preserve the data in the event of loss or failure of the field computer.

The ADCP should be dried after use and stored in its protective case for transport. When working in estuaries and other salt-water environments the ADCP should be rinsed off with fresh water and dried prior to storing the ADCP for transport. Failure to dry the ADCP may result in corrosion of the ADCP connectors, mounting brackets and any ADCP accessories stored in the protective case. This is especially important when working in saltwater environments.

6.2.8 ADCP stationary measurement methods

Recently ADCP manufacturers have developed software for making stationary discharge measurements with ADCPs. Rather than traversing the channel with a moving platform to make a measurement, in this method the ADCP is held in position at stations across the channel. At each station a vertical velocity profile is collected from which a mean velocity is computed. The mean velocity plus depth data and distances between stations is used to compute discharge, much as a conventional meter measurement would be made. The hydrographer generally will float the ADCP from a tethered platform and move the ADCP from station to station, keeping track of distances using bridge marks, measuring tapes or marked tag lines. The ADCP is held in position long enough to collect a representative velocity profile. Because bottom tracking is not used during a stationary ADCP measurement, this method could be useful in conditions where moving stream bedloads are significant. Another application of this method would be to use ADCPs to make measurements of ice-covered rivers. In this case holes would be cut in the ice and the ADCP suspended in the hole. At the time of this writing, this software was relatively new so no detailed recommendations or suggestions are presented here for making ADCP stationary measurements. In addition to the SonTek and Teledyne-RDI systems discussed in this section, Nortek Inc. manufactures an acoustic current profiler called a Qliner. This instrument was also sold with the trade name BoogieDopp, and has been used in the United States of America and other countries. The Qliner/BoogieDopp is deployed from a small tethered boat, and does not bottom track – thus it is designed to make discharge measurements using the ADCP stationary method. Other manufacturers of ADCPs that do not bottom track are Aanderaa Instruments and OTT.

6.2.9 ADCP discharge measurement accuracy

An extensive study of ADCP discharge measurement uncertainty is not yet available. Comparative studies of ADCP measurements with other types of measurements, such as current meter measurements, and with stage-discharge ratings, indicate that ADCP measurements provide acceptable accuracy. A more detailed discussion of ADCP measurement uncertainty is given in Chapter 10 of this Manual.

6.3 RIVER GAUGING USING ACOUSTIC DOPPLER VELOCITY METER

The Acoustic Doppler Velocity Meter (ADVM) is an instrument that operates in a manner very similar to an Acoustic Doppler Current Profiler (ADCP) that was described in previous sections of this Manual. An ADVM measures water velocity by using the Doppler principle applied to sound transmitted under water. The primary difference between the ADCP and the ADVM is that the ADVM is fixed to a stationary mount under the water surface instead of being mounted on a moving boat. Secondly, it transmits the acoustic beam horizontally (referred to as side-looking) through the water rather than vertically. General details of the instrumentation, installation, rating development and other aspects of the ADVM will be given in the following sections of this Manual.

This Manual describes side-looking ADVMs from two manufacturers. It should be noted that there are now a number of other ADVMs available with configurations and capabilities different than the two units described in this Manual. Some other ADVMs commonly used for index-velocity applications include:

(a) Up-looking instruments that are mounted on the channel bottom and provide vertical velocity data; these are useful at sites with unusual vertical velocity distributions such as occurs with vertical bi-directional flow;
(b) Lower-frequency units that have a far greater range than the units described in this Manual;
(c) Units from other manufacturers such as Teledyne RD Instruments and Aanderaa;
(d) Profiling instruments that measure velocities in multiple cells. Basically, these instruments are ADCPs that lack bottom-tracking capabilities and are designed for in-situ installations. There is some blurring between these profilers and ADVMs with multiple sample cells or volumes. They can for practical purposes be considered in the same class of instruments and the index-velocity method of producing discharge records would apply to any of these instruments.

The index-velocity method of defining a rating curve has been used for many years, primarily with vane gauges and point electromagnetic velocity meters (see Chapter 8). These instruments yield an index velocity at a single point in the stream. The ADVM, on the other hand, gives an index velocity for a significant part of the flowing stream which can more easily be related to the stream mean velocity. It is useful in streams affected by variable backwater, sluggish, slow-moving streams, ice-affected streams, tidal streams and streams where flow reversals may occur. It should be noted that the term index velocity and velocity index are used interchangeably in this Manual as well as other reports that discuss this method of developing discharge ratings where a rating parameter is water velocity.

The content and descriptions given in the following sections of this Manual are based on a report by

Morlock, Nguyan, and Ross (2002). Several parts of their report are used verbatim.

6.3.1 Basic principle of operation

An ADVM uses a pair of monostatic acoustic transducers set at a known orientation to measure water velocities. Monostatic refers to the capability of each transducer to transmit and receive sound (SonTek Corporation, 2000). Each ADVM transducer transmits sound pulses (pings) of a known frequency along a narrow acoustic beam (Figure I.6.2). As the pings travel along the acoustic beam they strike particulate matter suspended in the water. When the pings strike suspended matter, which acts as a sound scatterer, some of the sound is reflected along the acoustic beam to the transducer. The returned sound (echo) has a Doppler shift proportional to the velocities of the scatterers and water they are travelling in along the acoustic beam.

The two acoustic beams are set at a known angle (beam angle) in a two-dimensional plane that is parallel to the water surface (see Figure I.6.19) so if seen from above, they would be in a "V" configuration. From velocities measured along the individual acoustic beams, the ADVM uses trigonometry (because the beam angle is known) to compute velocity in a user-set part (sample volume) of the plane defined by the beams (see Figure I.6.19). An ADVM will compute and output a mean velocity for the sample volume; the velocity is output in terms of an x-component and a y-component. In a typical installation where the ADVM is mounted on the side of the river, the x-component is the component of velocity parallel to the main flow direction of the river (along flow) and the y-component of velocity is perpendicular to the main flow direction of the river (across flow). The ADVMs sample velocities over a period of time set by the user, the averaging interval and report the x- and y-components of the mean velocity sampled during the averaging interval.

An important ADVM parameter is signal strength, which is a measure of the strength of the echoes returned to the ADVM. Signal strength decreases with distance from the ADVM because of sound absorption and spread of the acoustic beams (SonTek Corporation, 2000). The maximum measurement range of an ADVM is dependent upon the distance at which the signal strength approaches the instrument noise level. For this discussion, the instrument noise level may be considered the signal strength of the ADVM measured while the ADVM is out of the water (SonTek Corporation, 2000). ADVM manufacturers sometimes use the terms signal strength and beam amplitude interchangeably.

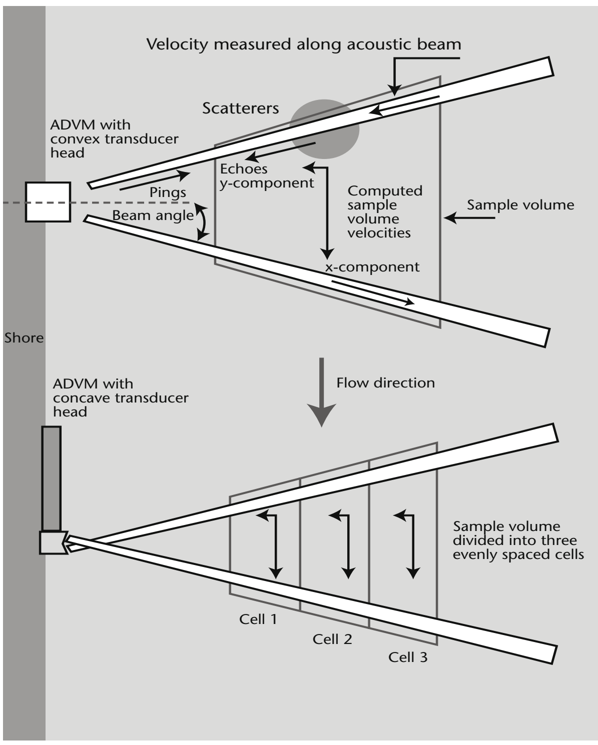

Figure I.6.19. Schematic of a typical Acoustic Doppler Velocity Meter setup

6.3.2 Description of Acoustic Doppler Velocity Meters

There are currently many models of ADVMs in use for index-velocity applications. Manufacturers of commonly used ADVMs include SonTek, Nortek, Teledyne RD Instruments and Aanderaa. These manufacturers produce ADVMs with widely varying configurations, acoustic frequencies and other features. Some of these instruments, such as the SonTek Argonaut-SW, are designed to be mounted on the channel bottom to look up through the water column. These are useful for sites with vertical bi-directional flow or vertical flow stratification. The index-velocity rating method is the same for up-looking instruments.

Two ADVMs that are commonly used, and that are similar in construction and features, are the Argonaut-SL and EasyQ (see Figure I.6.20). These two ADVMs will be used in the following paragraphs for purposes of describing ADVMs. Both consist of a transducer head attached to a canister-shaped housing that contains the instrument data-processing electronics. The Argonaut-SL has a convex transducer head while the EasyQ has a concave transducer head (see Figure I.6.20). Both

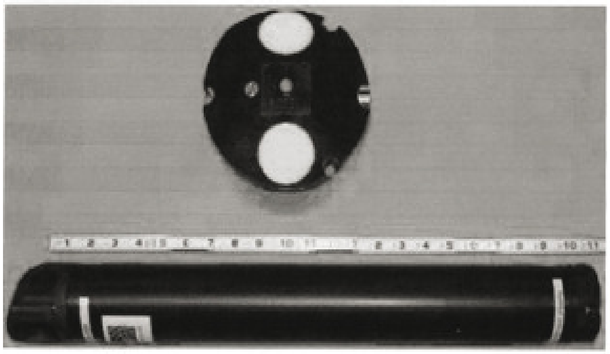

Figure I.6.20. Argonaut-SL (top) and EasyQ (bottom) Acoustic Doppler Velocity Meters

instruments have a watertight cable connector for the attachment of a communications/power cable. Selected specifications for the Argonaut-SL and EasyQ used in this study are given in Table I.6.2.

Table I.6.2. Selected specifications for the Argonaut-SL and EasyQ Acoustic Doppler Velocity Meters

Feature	*Argonaut-SL*	*EasyQ*
Length (cm)	18	58.9
Diameter (cm)	15.2	7.5
Weight in air (kg)	2.5	1.7
Transducer beam angle (degrees)	25	25
Sample volume minimum start distance (m)	0.5	0.2
Sample volume maximum distance (range) (m)	22	16
Transducer frequency (MHz)	1.5	2
Number of cells within sample volume	1	3

Note: centimeter (cm), kilogram (kg), metre (m), megahertz (MHz)

The EasyQ measures and outputs velocities from three separate, consecutive cells within the sample volume (see Figure I.6.20). Each cell's size can range from 0.4 to 2 m (the maximum distance to the beginning of the first cell is 6 to 8 m, giving a maximum range of 16 m). The EasyQ is also equipped with an upward looking transducer designed to measure stage acoustically.

Argonaut-SL's and EasyQ's are available with other features and options. For example, the Argonaut-SL is available in different form factors (shapes) and has an upward looking stage transducer and multi-cell (5 cells) capability, as well a 3.0-MHz-frequency unit intended for smaller rivers and streams. A long-range, 1-MHz EasyQ model is available with 4-m cells.

The Argonaut-SL and EasyQ manufacturers provide software used to program their instruments. Programmable parameters include sample-volume size and velocity-sampling interval. The software also is used to perform data-quality and instrument diagnostic checks.

The Argonaut-SL and the EasyQ can be interfaced with electronic data loggers (EDLs) to collect and store parameters from sensors. The parameters then can be retrieved remotely from the EDLs by using various telemetry methods for real-time-data applications. The ADVM can be interfaced with an EDL by using the SDI-12 (serial-digital interface at 1 200 baud) communication protocol. To use the SDI-12 protocol, a sensor is connected to a cable that consists of a data wire, power wire and ground wire. Using the SDI-12 protocol, a number of sensors can be connected to the same EDL through a single communications port. An EDL using the SDI-12 protocol issues a measurement command to the ADVM. The ADVM returns a data string to the EDL that tells the EDL the length of time the ADVM will sample and the number of parameters that will be returned to the EDL after the sampling is complete. After the sampling is complete the ADVM sends a data string to the EDL containing the sampled parameters.

The SDI-12 standard allows an EDL to collect up to nine parameters from the Argonaut-SL. The parameters relevant to using an Argonaut-SL for the production of river discharge include the following:

(a) Velocity x-component: the x-component of the mean velocity measured within the sampling cell (see Figure I.6.19);
(b) Velocity y-component: the y-component of the mean velocity measured within the sampling cell (see Figure I.6.19);
(c) Computed velocity vector: the resultant vector computed from the x- and y-velocity components, using the following formula:

$$V = \left(V_x^2 + V_y^2\right)^{0.5} \tag{6.14}$$

where V is the computed velocity vector, V_x is the velocity x-component, and V_y is the velocity y-component;
(d) Standard deviation: the mean standard deviation of the Argonaut-SL velocity measurement;
(e) Signal strength: the mean strength of the echoes returning to the Argonaut-SL;

(f) Temperature: the mean temperature measured by the Argonaut-SL.

The SDI-12 option allows an EDL to collect up to 18 parameters from an EasyQ. The parameters relevant to using an EasyQ for the production of river discharge include the following five:
(a) Velocity x-components for cells 1, 2, and 3: the x-components of the mean velocities measured within each of the three EasyQ cells (see Figure I.6.19);
(b) Velocity y-components for cells 1, 2, and 3: the y-components of the mean velocities measured within each of the three EasyQ cells (see Figure I.6.19);
(c) Amplitudes for cells 1, 2, and 3: the average echo strength within each of the three EasyQ cells;
(d) Stage: the stage output from the EasyQ upward looking transducer;
(e) Pitch and roll: the pitch and roll of the EasyQ computed by the EasyQ on-board tilt sensor.

There are a number of ADVMs available that can profile with many user-sized sample cells or volumes. These instruments in many cases can be considered to be ADCPs without bottom tracking because of the additional features they may have, such as:
(a) Internal tilt and roll sensor;
(b) Internal compass;
(c) Dynamic ranging, a feature that adjusts the range of the uppermost bin based on the distance to the water surface;
(d) A fourth beam that produces a redundant vertical velocity to test the assumption of homogenous flow in all beams.

Profiling ADVMs 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. Because of the complexity and number of output data, some profiling ADVMs may not have SDI-12 equipped for interface with DCPs. Data from these instruments is commonly logged internally and retrieved periodically using a field computer.

Profiling ADVMs have varying acoustic beam geometries and can be installed in several different orientations. Profiling ADVMs with two acoustic beams are usually used for horizontal orientations. The beams are set in a two-dimensional plane that is parallel to the water surface so if seen from above they would be in a V configuration. In the example orientation shown in Figure I.6.19, the profiler is mounted on the side of the river and velocities are output in Cartesian coordinates; the x-component is the component of velocity parallel to the main flow direction of the river (along flow) and the y-component of velocity is perpendicular to the main flow direction (across flow). The profiling ADVM outputs an x-component and y-component of velocity for each bin (the number and size of the bins is programmable).

Profiling ADVMs with three acoustic beams are usually used for vertical orientations with the three beams traversing the water column from the bottom up for a profiling ADVM mounted on the channel bottom or from the top down if the ADVM is mounted on a floating buoy. Profiling ADVMs with three beams can output velocities in three dimensions, where the x-component is along the flow, the y-component is across the flow and the z-component is the vertical-velocity component (up or down). All three components can be output for each bin.

The velocity measured by the profiling ADVM that is used for an index velocity can be a mean velocity measured in one or multiple bins. Profiling ADVM transducer frequency, maximum and minimum range, maximum and minimum bin size and features vary by manufacturer and model.

6.3.3 **Installation Considerations**

A primary ADVM-installation consideration is avoidance of acoustic-beam signal contamination caused by boundary reflections. Boundaries in a river can include the water surface, river bottom, structures such as bridge piers and objects such as rocks and logs. The ADVMs described in this Manual have narrow (about 2-degree) acoustic beams. The beams spread with range from the ADVM and, in a shallow river or stream, may strike the surface or bottom, causing beam contamination and biases in the ADVM velocity measurements. Boundary interference can be prevented with the use of aspect ratio, which provides an estimate of maximum ADVM range and is expressed as range/distance. Range is the maximum sample volume range and distance is the distance to the closest boundary (an example using an aspect ratio of 4: an ADVM that is 1 m deep and 2 m from a river bottom should be programmed so that the maximum sample volume range is no greater than 4 m). The aspect ratio indicates that an ADVM will have less range in shallow than in deep rivers and streams. In most conditions, aspect ratios of 8 to 10 will work and at many stations with high backscatter and smooth bottoms, aspect ratios could be greater than 15.

ADVM signal strength can be used to check for boundary interference within the sample volume. If no boundary interference is present, the signal strength of each beam should peak at the transducer and then gradually decrease with distance from the transducer. Boundary interference will cause the signal strength to increase markedly or spike. A spike in the ADVM sample volume means that boundary interference may produce unreliable velocity data. The size of the sample volume would need to be selected so the spike remains outside the sample volume. ADVM manufacturers recommend the end of the sample volume be placed no closer than 10 per cent of the total distance from the ADVM to the boundary (for example, if a boundary is discovered at 10 m, the end of the sample volume should be no farther than 9 m). The 10 per cent recommendation is designed to minimize a phenomenon known as sidelobe interference (SonTek Corporation, 2000).

If no boundary interference is detected within the maximum range of the ADVM, selection of the sample volume end distance should be based on the instrument noise level. The manufacturers recommend that the sample volume end be programmed so that the beam signal strengths at the sample volume end are about five counts above the instrument noise level. ADVM software will display the signal strengths in units of counts, where one count equals about 0.43 decibel.

The end of the sample volume is selected to prevent boundary interference and to be above the instrument noise level. The beginning of the sample volume should be selected to minimize turbulence from the structure on which the ADVM is mounted.

If beam-signal strengths drop below the five counts above the instrument noise level the Argonaut-SL has a feature that automatically will reduce the end of the sample volume. To ensure accuracy the user should consider setting the end of the sample volume as specified above so the Argonaut-SL does not reduce the end of the sample volume. A reduction would change the sample volume size and could change the index-velocity relation.

Another programming consideration is the relation of velocity measurement errors to the averaging interval. ADVM measurement errors will result from instrument and environmental sources. Environmental errors will likely dominate instrument errors. There will be random instrument errors associated with the measurement of velocity using a single ping. Errors are reduced by averaging pings, which increases the averaging interval. Some environmental errors will be caused by turbulence, the size of turbulent eddies as dictated by channel geometry and short-term flow pulsations. At stations where river turbulence is pronounced, a longer averaging interval may be needed to attain the level of accuracy a shorter interval would produce at stations with less turbulence. In general, the averaging interval for each station will need to be considered individually and some experimentation may be needed to find an optimal interval.

For ADVM installation it is important to consider flow disturbances caused by structures on which an ADVM is mounted. It is often desirable to mount instruments on the downstream face of a bridge pier, which provides some protection from debris. Mounting to the downstream face of a pier, however, could cause the ADVM to sample within the vortices caused by flow separation from the upstream face of the pier (wake turbulence). The beginning of the sample volume would need to be beyond the wake-turbulence zone. The following equation, derived from Hughes and Brighton (1991), can be used to estimate the extent of the wake-turbulence zone:

$$b = c(dx)^{0.5} \tag{6.15}$$

where b is the lateral distance from the pier centre line to the approximate edge of the wake-turbulence zone, d is the width of the pier, x is the distance to the upstream face of the pier, c is a factor accounting for pier shape: c is 0.62 for circular or round-nosed piers; c is 0.81 for rectangular piers.

For stations with long piers or narrow channels it may not be practical to set the sample volume totally outside the wake-turbulence zone. If a part of the sample volume is within the zone it may be necessary to use a longer averaging interval to compensate for the additional turbulence. If possible, the sample volume should be set to be outside the wake-turbulence zone.

Some other installation considerations include the following:

(a) Protection – The ADVM should be protected from debris and vandalism. Its mount should be durable and rigid and the instrument should be accessible for maintenance;
(b) Power – The longer the averaging interval the greater the power consumption (ADVM manufacturers can be consulted concerning computation of power consumption);
(c) Cable lengths – The maximum recommended cable length for SDI-12 operation is 250 ft (for

distances above 250 ft, it may be possible to provide SDI-12 communications using SDI-12 radios);
(d) Bio-fouling – Instruments in waters that contain mussels or barnacles can be subject to fouling of the transducers by these organisms. Heavy growth over the transducers will attenuate the strength of transmitted and hence received acoustic signals. Eventually the signal strength will approach the instrument noise limit and the velocity data will become corrupt. ADVMs that may be subject to bio-fouling should be mounted so that they are easily raised for cleaning. The application of anti-bio-fouling paint to the transducers can help retard the growth of organisms, but even instruments treated as such will require cleaning.

For up-looking ADVMs that are to be mounted on the channel bottom there are additional considerations as follows:
(a) The ADVM will need to be secured on the bottom in such a way that it does not move vertically or horizontally. If the instrument moves the orientation of the beams in relation to the current will change, compromising the accuracy of the index-velocity rating. Mounts can be fabricated or can be purchased from several manufacturers;
(b) Bottom mounted ADVMs can be vulnerable to snags by boat anchors or fishing lines. There are commercially available trawl-resistant mounts for ADVMs;
(c) For sites with heavy bed-load movement, the ADVM may become covered with sand or silt which will corrupt the velocity data and make it difficult to retrieve the unit;
(d) Instruments on the bottom will still need to be occasionally retrieved for cleaning, repair or replacement.

6.3.4 Basic concept of computing river discharge using ADVM

The following approach to the computation of river discharge is based on methods where cross-section area and stream velocity are used to compute discharge records from instruments that measure stage and index water velocities. These methods are similar to other methods that use an index velocity, such as AVM's and electromagnetic meters, for the purpose of computing discharge.

River discharge can be computed, as:

$$Q = \overline{V} A$$

where Q is the discharge in cubic metres per second, $\overline{V}$ is the mean velocity for a specified channel cross section, in metres per second, and A is the channel area for a specified cross section, in square metres.

The channel area, A, for a river can be determined by surveying the cross section of the river. Because a range of stage occurs in most rivers, it is necessary to develop a relation between stage and channel area, called a stage-area rating. The channel area for any given stage then can be computed from the stage-area rating.

The mean velocity, $\overline{V}$ for a river can be computed from the water velocity measured by an ADVM. To compute mean velocities from ADVM-measured velocities, the relation between mean channel velocity and ADVM-measured velocity must be determined. The method used to relate mean velocity and ADVM-measured velocity is the index-velocity method. The ADVM instrument measures velocity in a part of the stream and that measured velocity is used to compute the mean channel velocity from the index-velocity rating.

After stage-area and index-velocity ratings are developed, discharges can be computed for a streamflow gauging station equipped with an ADVM. Discharge can be computed from each ADVM velocity recorded by the station EDL. A measurement of stage also must be recorded so that channel area can be computed. Details for developing the stage-area rating, the index-velocity rating, and the computation of discharge are given in WMO-No. 1044, Volume II, Chapter 2 – Discharge Ratings using the velocity-index method.

6.4 RIVER GAUGING USING ULTRASONIC (ACOUSTIC) VELOCITY METER METHOD

This section describes a method of river gauging that utilizes the time of travel of a sound wave propagated across a stream for the purpose of measuring stream velocity. As explained at the beginning of this chapter, the terms ultrasonic, sonic and acoustic are sometimes used interchangeably. This method of river gauging has been in use for 30 years and has proven to be a reliable method.

The ultrasonic (acoustic) velocity meter (AVM) of river gauging was introduced to afford a means of gauging where existing methods were unsuitable. The velocity-area method, for example, requires conditions which produce a stable stage-discharge relation and measuring structures are generally confined to small rivers having sufficient afflux available and where a construction in the river is acceptable. Generally cost alone rules out the installation of a measuring structure in rivers over 50 m wide. The AVM is similar to the ADVM described in the previous section of this Manual in that both classes of instruments use hydro-acoustics, or sound propagated through the water to measure water velocities. The description of the AVM method, as presented in the following sections, is based primarily on ISO 6416 (1992). Other important references include Green and Herschy (1978) and Herschy (1974).

6.4.1 **Principles of measurement**

When a sound pulse is transmitted through water in motion, in a direction other than normal to the mean direction of movement, the time taken to travel a known distance will differ from that taken in stationary water of the same temperature, salinity, sediment concentration and depth. If the sound pulse is transmitted in the same direction as that in which the water is flowing, the time taken to cover the known distance will be shorter than in stationary water. If the pulse travels in a direction that is opposite to that in which the water is flowing the time of travel will be longer.

If the time taken for a sound pulse to travel a measured distance between two reference points in one direction is compared with the time taken to travel between the same two points in the opposite direction, the difference observed is directly related to the average velocity of the element of water in the flight path bounded by the two reference points. This is referred to as the path velocity.

This basic principle, in combination with appropriate instrumentation, allows accurate measurement of the mean velocity of the element of a body of water that is located in the line that joins the two reference points. A method of sampling flow velocity, which provides more information about the average condition of the entire body of flowing water than does a point measurement, but which still falls short of being a fully representative measurement of the total flow, is thus available.

However, just as a number of point samples of flow velocity can be integrated to provide an estimate of mean cross-sectional velocity, path velocity measurements can be mathematically transformed for the same purpose. The relation between the path velocity, v_{path}, and that along the line of flow in the channel, v_{line}, (known as "line velocity") is:

$$v_{line} = \frac{v_{path}}{\cos\phi} \tag{6.17}$$

where ϕ is the angle between the acoustic path and the direction of flow.

In open-channel flow measurement, practical considerations will normally dictate:

(a) That the reference points at either end of an acoustic flight path are located on opposite banks of the watercourse;

(b) That the line that joins them intersects a line that represents the mean direction of flow at a known angle which normally lies between 30° and 60°.

At intersection angles greater than 60° the time differences between sound pulses in opposite directions may become excessively small and difficult to measure. This problem may not be significant where high velocities are to be measured but if velocities are low (that is where time differences between forward and reverse sound pulses are themselves small), difficulties may arise.

At an angle of 90° there will be no time difference between forward and reverse pulses.

With large angles there is also an increase in the error in velocity computation that results from related errors in the measurement of the angle. This is due to the presence of the cosine function in the equation relating time difference to velocity. Table I.6.3 demonstrates this effect.

Table I.6.3. Systematic errors incurred if the assumed direction of flow is not parallel to the channel axis

Path angle, ϕ	*Velocity error for 1° difference between actual and assumed flow direction*
Degrees	*%*
30	1
45	2
60	3

To allow discharge to be calculated, not only should an estimate of mean velocity in the gauge cross-section be available, but the channel cross-sectional area should also be known. A system for flow determination using the ultrasonic principle will be capable of making sample measurements of velocity, water stage and computing channel area from a stage-area rating. Considerations for stage-area ratings are the same as those discussed under the previous Acoustic Doppler Velocity Meter section.

6.4.2 Characteristics of sound propagation in water

The sound spectrum encompasses a wide range of frequencies. The audible range lies between approximately 50 Hz and 15 000 Hz and is generally referred to as sonic. Frequencies less than 50 Hz are usually termed subsonic. At frequencies above 15 000 Hz, the term ultrasonic is normally applied.

Speed of sound in water

The speed of sound in fresh water varies from about 1 400 m/s to a little above 1 500 m/s, over the normal ambient temperature range. This represents a variation of approximately 7%. In water containing dissolved salts the speed is somewhat higher than in fresh water. The speed of sound depends on the density and elasticity of the medium and is independent of frequency.

Transmission of sound in water

Only a portion of the acoustic energy transmitted reaches the target. The remainder is lost for various reasons, as follows:

(a) Spreading loss is the reduction in acoustic intensity due to the increase in area over which the given acoustic energy is distributed. Losses due to this cause depend upon the relation between the path length, the diameter of the ultrasonic transducer and its characteristics frequency;

(b) Attenuation loss is the reduction in acoustic intensity due to the resistance of the medium to the transmission of acoustic energy. It is analogous to the loss of electric energy in a wire, where there is no spreading loss. Attenuation loss is directly proportional to the square of the frequency;

(c) Scattering is the modification of the direction in which acoustic energy is propagated caused by reflections from things such as microscopic air bubbles and suspended matter. For ADVM and ADCP measurements, these reflections are the basis for velocity measurements as those instruments use the Doppler shift of the reflections to compute velocities. For AVMs these reflections are a source of signal loss;

(d) Absorption is the process by which acoustic energy is converted into heat by friction between the water molecules as a sound wave is subjected to repeated compressions and expansions of the medium. In general, this loss is a function of frequency squared.

Reverberation

Reverberation is the energy returned by reflectors other than target reflectors. Reverberation of sound in water is analogous to the familiar optical effect which impairs the utility of automobile headlights on a foggy night.

Refraction

The path taken by an acoustic pulse will be bent if the water through which it is propagating varies significantly in either temperature or density. In slow moving rivers with poor vertical mixing, the effect of the sun upon the surface may produce a vertically distributed temperature gradient. This will cause the acoustic path to bend towards the bed. With a temperature gradient of 0.5°C per metre of depth over a path length of 50 m, the vertical deflection will be about 2 m. In contrast, the effect of vertical density gradients (such as may be associated with salt water intrusion into the gauged reach) is to bend the path towards the surface. Similar effects may be produced by horizontally distributed temperature or density gradients such as may be associated with partial shading of the shading of the water surface or with the confluence of tributary waters of contrasting characteristics.

Reflection

Sound is reflected from the water surface and, to a lesser extent, from the channel bed. The bed may even be a net absorber of sound. As an acoustic wave propagates across a river (generally as a cone of around 5° width) it will intersect with the water surface and be reflected, suffering a 180° phase change in the process. The secondary wave will proceed across the river and arrive at the opposite bank. Its arrival will be sensed by the target transducer later than the direct wave and the difference in arrival time will be a function of the difference in the respective lengths of the direct and indirect paths.

Errors in signal timing will occur if the secondary signal interferes with the first cycle of the direct signal. To avoid this effect the difference in the two paths should exceed one acoustic wavelength (speed of sound/frequency). This will be achieved if the depth of water above the acoustic path exceeds that given by the equation:

$$D_{min} = 27\sqrt{\frac{L}{f}} \quad (6.18)$$

where D_{min} is the minimum depth above the acoustic path, in metres, L is the path length, in metres, and f is the transducer frequency, in hertz.

A similar restriction may apply to the channel bed, particularly if it is smooth, and reflects rather than absorbs an acoustic signal.

6.4.3 **Application**

Like all variants of the basic velocity-area method, the ultrasonic method is suitable for use in some situations, and unsuitable in others. The following paragraphs describe some of the positive attributes of the method.

Open channels

The method is suitable for use in general purpose river flow measurement, a significant advantage being some additional freedom from constraints applicable to other available techniques. In particular, the method does not demand the presence of a natural control or the creation of a man-made control at the proposed gauge location, as it does not have to rely upon the establishment of a unique relation between water level and discharge.

The method is capable of providing high accuracy of flow determination over a wide range of flows contained within a defined gauge cross section. Flow determination of predictable accuracy can be available from the time the gauge is first established.

Use of the method creates no obstruction to navigation or to the free passage of fish. It creates no significant hazard or loss of amenity for other river users or riparian interests. If carefully designed, the gauge can be physically unobtrusive.

Backwater effects

The method is generally tolerant of the backwater effects created by tides, tributary discharges, reservoir water level manipulation, periodic channel obstruction and downstream weed growth.

Multiple channels

At locations where total flow is divided between two or more physically separate channels, the technique allows instrumentation to be used to determine individual channel flows separately and then to combine these basic data to create a single unified determination.

Flood plain flow measurement

Flow may not readily be contained within a single, well-defined cross-section, but may have significant flow that by-pass the main gauge cross-section by way of an extensive flood plain. In such cases it may be possible to subdivide the flood plain by means of minor civil engineering works into a series of channels in which the flow can be measured separately.

A station may be designed to provide a comprehensive flood-plain measurement capability by this means or may simply provide a flow or velocity sampling facility. In the latter situation, gauged cross-sections may be constructed in the flood-plain. These should not provide total coverage but merely provide locations at which flood-plain flow can be sampled for subsequent examination and analysis.

6.4.4 **Gauge configuration**

The ultrasonic method can be set up in several different ways depending on site characteristics, accuracy requirements and resources available. Several possible gauge configurations can be used as described in the following sections.

Single path systems

In its most basic form, the ultrasonic gauge can operate satisfactorily with a single pair of transducers, giving only a single line velocity determination. Provided that a relation can be established between this sample and the mean cross-sectional velocity, discharge can be computed as readily by this simple means as by a more complex method. Figure I.6.21 illustrates a single path system.

Transducer mountings may be moved vertically. A vertical velocity profile may be determined in a manner analogous to the rotating element current meter. An easier approach to determining vertical velocity profiles would be to employ an ADCP. The transducers may then be set at an elevation that provides a valid estimate of the mean cross-sectional velocity.

Transducer settings may be altered to account for seasonal flow regime changes. For the single path

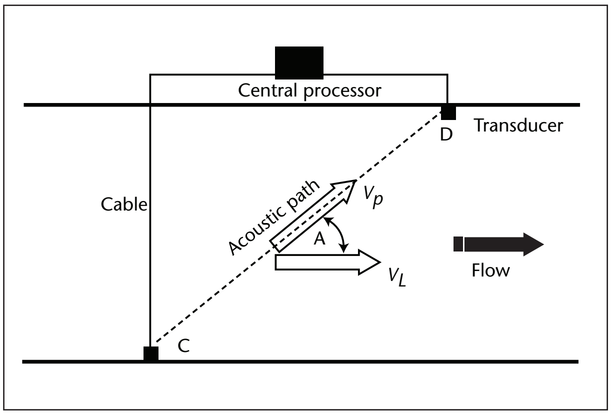

Figure I.6.21. Single-path ultrasonic gauge configuration

gauge with movable transducers, the range of water levels at the gauge site should be small or change slowly. Wide variations in water level can sometimes be accommodated if they occur slowly. For example, in a groundwater-fed stream, discharges vary slowly from day to day.

The single path gauge relies on a relatively stable velocity profile. It may not be suited to locations that experience a significant backwater effect. The single path gauge is inherently vulnerable to transducer damage or malfunction. There is no built-in component redundancy capability.

Multipath systems

At some sites it will normally be necessary to install two or more paths to provide a more accurate estimation of mean velocity in the cross-section than is possible with a single path only. These sites may include one or more of the following flow conditions:

(a) There is wide and frequent variation in water level and/or flow;
(b) Velocity distribution in the vertical deviates significantly from the theoretical;
(c) There is significant risk of backwater effects acting upon an otherwise stable stage/discharge relation.

The number of paths that may be installed is limited only by the design of the gauge instrumentation chosen to meet the required constraints of accuracy, reliability and cost. The aim is to achieve an acceptable representation of the vertical velocity profile in the gauge cross-section, at all levels or flows, from the highest to the lowest likely to be experienced.

Where a high level of performance security (that is freedom from operational interruption or degradation) is also a goal in the system, it may be desirable to provide an additional number of redundant paths, such that physical damage to or malfunction of one or more paths has a minimal effect upon the overall accuracy of measurement.

Multipath gauge configuration may also be appropriate as a means of accommodating complex cross-sectional geometry. Figure I.6.22 illustrates four types of multipath ultrasonic gauge configurations.

Crossed path systems

One of the fundamental principles of the ultrasonic technique is that the angle at which each individual flight path in a system intersects the line representing the mean direction of flow at that elevation should be known accurately. Errors in this angle are magnified in the discharge computation process (see Table I.6.3).

In practice, it may be difficult to determine precisely the mean direction of flow at a given site. The assumption that it is parallel to the banks may not always hold. It may be true at some parts of the flow/level range, but not at others. The gauge site itself may not be ideal and there may be directional effects associated with sub-optimal channel geometry or approach conditions. At low flows in particular, the effects of complex bed geometry may override the normal control of mean direction from the bank.

Where it is suspected that the flow is not parallel to the channel banks, and where the likely resulting error in the flow computation is thought to be significant, it may be possible to introduce an element of self correction. This can be done by configuring the gauge to have one or more sets of flight paths installed as pairs and set at the same elevation but laid out in the form of a symmetrical cross (see Figure I.6.22).

In this configuration each path oriented in an upstream direction from the left bank should be matched by an equivalent path at the same elevation oriented in a downstream direction from the same bank and aimed at a point on the right bank directly opposite the downstream, left-bank transducer. The twin paths should normally be disposed so as to intersect in midstream and to form the equal sides of a pair of congruent, isosceles triangles. Gross mismatch between path lengths should be avoided because of the

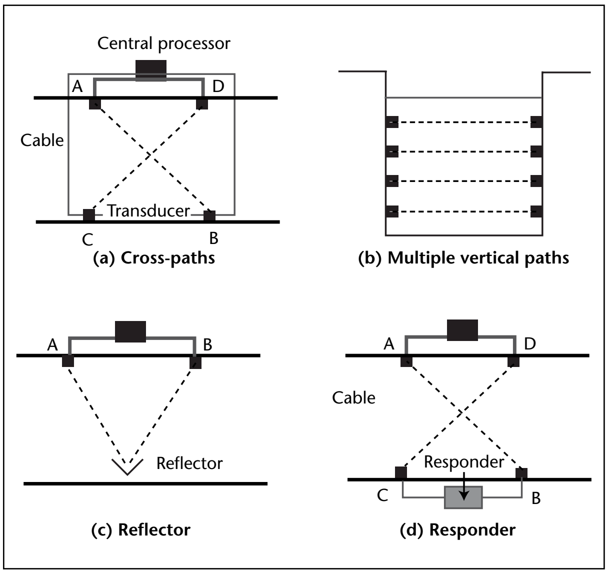

Figure I.6.22. Various multipath ultrasonic gauge configurations

likelihood of there being significant differences in cross-sectional geometry between the two paths.

Within the system instrumentation, each line velocity in a crossed pair should be computed separately. If the two velocities computed for a pair of crossed paths are identical (within computational and measurement error) then the path angle assumed by the system design may be taken to be correct. If the two velocities are significantly different then the assumed path angle is incorrect. Neither of the computed line velocities will be correct; one will be high and the other low.

Provided that the true mean direction of flow does not change significantly over the measured reach, then simple averaging of the two paired line velocities will produce a close approximation to the true mean water velocity at that elevation, the inherent errors in each being largely self-cancelling. The risk of error remaining, because of changing flow direction through the gauged reach, may be reduced by keeping the reach as short as possible.

At locations where high gauge reliability is required, measurement redundancy should be combined with the use of crossed-path geometry to reduce the risk of system failure through physical damage by having transducer arrays that are physically separated on the river bank.

Reflected path systems

The basic ultrasonic system normally requires that there are sets of transducers on both banks of the channel. It is required that signal and power cables should cross the channel either overhead, on the bed or trenched into the bed.

Alternatively, there may be situations in which it is inappropriate to provide live transducers on both banks. One bank may be inaccessible making system servicing difficult. In such situations, recourse may be made to a system configuration that has both transmitting and receiving transducers on the same bank, communicating via a passive reflector located on the opposite bank (see Figure I.6.22).

This configuration may also be used to achieve longer flight paths, where these are needed to improve measurement accuracy at low velocities without making the nominal path angle more acute. A further advantage is that the path angle does not need to be taken into account in the equation for computing line velocity thus eliminating a significant potential source of uncertainty.

Systems using divided cross sections

Modern instruments allow the adoption of very complex path configurations. The resulting system control and computational implications are relatively easy. A cross section can be divided into two or more parts, such as complex main channel geometry, a main channel with wide flood plain or a very wide main channel. Each channel part is then treated as a relatively simple gaugeable entity, and the individual results are added together. It should be considered, however, that wide flood plains are difficult to gauge using transit time systems because of problems with obtaining minimum depth.

6.4.5 Site selection

There are a number of both practical and physical considerations that must be evaluated in the process of selecting a suitable gauge site.

Site access

Sites using the ultrasonic technique should be readily accessible. During installation, significant civil engineering works may be required and heavy construction equipment may be needed on site. The technique requires use of electronic technology. The need to handle such equipment over long distances should be avoided.

Power supply

Short period (48 hours) battery operation of sophisticated systems and long term (3-6 months) operation for simple systems having a low sampling rate are feasible. Data loggers and telemetry devices can be operated with batteries for many years. However, for reliable, sustained operation of a multipath flow-meter a continuous source of electricity is required. Connection to a source of power at an acceptable cost should be possible or on-site generation capacity should be provided. Use of solar cells for battery charging is also a viable alternative.

Calibration and periodic verification measurements

The technique provides an absolute determination of velocity. However, in systems designed to have only a small number of separate flight-paths periodic calibration may be necessary to establish the relation between indicated velocity and some alternative determination of velocity in the cross-section. Even in multipath systems, where the velocity of the cross-section is inherently well sampled, there will often be a need for verification measurement by means of an alternative method. It is prudent to bear in mind the needs of acceptable alternative measuring methods when choosing a site for an ultrasonic gauge.

Geometry of the cross-section

The channel to be gauged should be straight with its banks parallel. The bank-to-bank bed profile should be nearly horizontal. There should be a minimum change in cross-section geometry between the upstream and downstream ends of the gauged section. Locations that are subject to significant bed level or profile instability should be avoided.

Channel aspect ratio

Sound pulses transmit through water as cones of projection. If the channel to be gauged is wide relative to its depth, the cone of projection of one or more transmitting transducers may intersect with the bed or the water surface before reaching the receiving transducer, resulting in signal reflection. Unless the system is designed carefully this may cause spurious measurements.

The ultrasonic system is not suitable for use in wide, shallow channels. Limiting conditions at a particular site will depend upon the number of paths to be installed and the number of paths remaining operational during low water. Limiting width/depth ratios can be computed readily and alternative design strategies are available.

Low-frequency sound attenuates less with distance than high frequency. Where the path length is short the use of low-frequency sound may result in unacceptable errors.

Aquatic vegetation

Aquatic vegetation in the cross section seriously attenuates the acoustic signal. Different types of aquatic plants may have different effects because it is the air included within the plant structure which produces the unwanted result.

Water temperature gradients

Temperature gradients in the water can result in signal loss.

Suspended sediment

The presence of solids suspended in the water may have a significant effect upon signal attenuation and can cause both reflection and scatter. At locations where suspended-sediment concentrations may be greater than 1500 mg/l for significant periods the ultrasonic technique may not be suitable.

Water density effects

Waters of differing density caused by salinity or other factors create problems similar to those associated with water temperature gradients. The key factor is the periodic nature of the salinity. If a density interface is present at the gauge location, signal loss due to refraction or reflection may occur. In wide estuaries, brackish water intrusions may cause cross-gradients and in such situations time may be required to allow the flow to stabilize and densities to become uniform before measurements can be taken.

Velocity profile effects

For effective operation of a gauge, velocity profiles in the gauged cross-section should show only minimal differences between the upstream and downstream reaches. Reaches that exhibit significantly changing velocity profiles should be avoided. If no alternative sites are available consideration should be given in the gauge design to:

(a) Making the gauged reach as short as possible;

(b) Incorporating additional paths to achieve better velocity sampling in the vertical; or
(c) Making provision to correct for non-parallel flow by means of a crossed-path system.

Entrained air

The presence of significant amounts of entrained air bubbles may cause problems due to reflection and scattering of the propagated acoustic wave. Locations which are downstream of dams, weirs, waterfalls, mills or power plant tail-races may suffer from this problem, which may persist for several kilometres downstream from the source. The ultrasonic gauge should be located at least 30 minutes flow travel time downstream of the source of aeration.

Tributary effects

The ultrasonic technique works most reliably where the physical properties of the water in the channel reach to be gauged are nearly homogeneous. In situations where a tributary is introducing water of a significantly different temperature or suspended-sediment concentration, difficulties may result. Full mixing of the two bodies of water may not be achieved for a considerable distance downstream of the confluence.

Ambient electrical noise

The effective functioning of the ultrasonic technique depends upon the reliability and sensitivity of the electronic equipment. Some instruments may be significantly affected by ambient electrical noise which may originate quite a distance from the gauge.. Powerful radio transmitters located many kilometres from the gauge may be a cause of difficulty. Most of these problems can be overcome.

Remotely-generated hydraulic effects

Velocity profiles that are far from ideal may be created by bed, bank or tributary confluence conditions at locations remote from the gauge, but may persist to have an effect at the gauge. They may exist during some flows but not others. Locations near severe bends in the channel, or close to tributaries of hydrological regimes different from that of the main stream should be avoided.

6.4.6 **Site survey**

Detailed site survey work should be carried out to evaluate the risks to system performance that might arise from each of the constraining factors outlined in the previous sections. Their likely effect upon overall system performance should be known before gauge design is undertaken.

Visual survey

A thorough visual survey should be undertaken on both banks of the watercourse, for an appreciable distance upstream and downstream of the potential site, to check that no obvious hazards to system performance are evident. This visual survey should be completed more than once, at times of both high and low river stages and during climatic and vegetation extremes. The factors of interest include aquatic vegetation, river traffic, the effects of the operation of navigation locks or power generation facilities, sediment concentration, access, security from unauthorized interference, land ownership, access for construction, operation and servicing, confluence locations, aeration effects, bed and bank condition, velocity profiles, location of bends, dams or weirs and water level range.

Survey of the cross-section

The cross-section of the proposed gauge should be surveyed thoroughly. If circumstances allow the survey should extend from as much as ten river widths upstream to two river widths downstream. A minimum of three cross-sections should be surveyed, although more are preferable.

The physical survey of the cross-section should be carried out more than once and results compared for evidence of bed and bank stability. Cross-sections should be selected that are representative of:
(a) Extreme low flow conditions;
(b) Conditions immediately after a significant flood or tidal event; and
(c) Conditions representative of any seasonality in the river regime.

In the ideal situation, a history of the geometry of the cross-section should be obtained over a number of years. If at all possible, two surveys should be undertaken, one before and one immediately after the high flow season.

Velocity distribution

At the earliest opportunity in the design process a detailed velocity profile survey should be completed at the potential gauge site. This will normally be done using conventional current meters and the greatest possible detail should be sought, commensurate with available resources.

The velocity profile survey should be designed to demonstrate the existence (or otherwise) of effective hydraulic uniformity in the channel reach to be gauged. Particular efforts should be made to obtain measurements that are representative of extreme low flow conditions since this is where the effects of non-uniformity are likely to be greatest.

At least three cross-sections in the reach should be profiled, one each at the upstream and downstream extremities and one at a central location. If resources allow, the velocity profile survey should match the detail of the physical survey of the cross-section and extend both upstream and downstream of the proposed gauge location.

Results of the velocity profile survey should be plotted graphically to show velocity isopleths in each surveyed cross-section and results inspected for evidence of significant change in the velocity distribution through the reach.

Signal propagation survey

If other indications facilitate the choice of site it is recommended that an acoustic survey should be carried out using portable equipment to determine whether sound propagation conditions are likely to be satisfactory. The equipment should consist of sets of transducers with support systems suitable for temporary mounting in the channel and an oscilloscope with dual time base capable of displaying the signal waveform. The waveform to be displayed may consist of a few cycles having a characteristic frequency between 100 kHz and 1 MHz, delayed from the transmit signal by many milliseconds (depending upon the path length) but typically about 70 ms for 100 m path length.

It is recommended that this survey be repeated at different states of flow and it should be designed specifically to test any areas of doubt that may have been identified in earlier physical and hydraulic surveys. The survey installation should consist of at least four transducers, deployed to form a pair of crossed paths, and capable of being moved vertically to sample velocity at different depths. Tests should be made to observe the strength and variability of signal and whether or not flow is parallel to the banks. Path comparisons should be made in terms of the path length and the line velocity for the path and not velocity alone.

Other survey activity

If possible (and certainly if no acoustic survey has been carried out), a water temperature survey should be performed at a time when there is a reasonable combination of low velocity and high insulation to determine if temperature gradients are likely to be present in the water.

In watercourses that are known to carry a high suspended-sediment concentration and at locations where there is a marked tidal effect, a suspended sediment survey should be undertaken. Such a survey should cover the full range of water depths. The ability of tidal flows to pick up sediment from the channel bed should be verified.

The site should be checked for excessive levels of electrical noise or radio interference. Potential sources include public broadcasting transmitters, emergency services communications or power installations that use a switched mode of operation. The characteristic frequency and amplitude of any interference, as seen by the transducers should be noted and incorporated in the system specification. Sources of electrical noise do not always operate continuously. It is prudent to carry out a control survey on more than one occasion and in addition, to inspect potential sources in the locality of the proposed site, whether or not any noise has been observed.

6.4.7 Operational measurement requirements

For successful operation an ultrasonic flow gauge requires information regarding the following:

Basic components of flow determination

The three essential components in the computation of flow are water velocity, water depth, and cross-section width. The first, water velocity, will always be provided by the specialized ultrasonic instrumentation itself.

Water depth determination is a function that may be incorporated within the ultrasonic instrumentation or may be derived from separate instruments. The actual determinant will normally be water level, relative to a fixed datum, which may be located either below the lowest possible or above the highest possible water level. The relationship between this datum and the assumed mean bed level in the gauge cross-section will normally be taken as a constant. This datum relationship should be checked periodically because its stability depends upon the stability of the channel bed.

In this context, mean bed level in the gauge cross-section, refers to the mean level of the channel bed within the area enclosed by the two banks at their lowest nominal locations and by lines through each of the most upstream and downstream transducer arrays of the gauge, drawn normal to the respective banks, as determined by the surveying of at least 50 uniformly distributed points on the bed within that area. Fewer points may be necessary for a uniform channel.

Channel width should be established by means of a conventional land survey technique, and checked periodically (although not as frequently as bed geometry). A high degree of accuracy (better than 0.1%) is readily attainable in the determination of distance, without resort to unusual methods or equipment, and this should be sought since acoustic path length is one of the fundamental system measurements. The basic ultrasonic flow measurement technique can accommodate quite varied channel bank geometry and individual flight paths in a multipath system can be of different lengths and can intersect the mean direction of flow at different angles, if the physical design of the gauge requires it.

6.4.8 **Computational requirements**

One of the features of the time-of-travel method is that the velocity of sound in still water component in the general equation for calculating water velocity from time difference is self cancelling when the separate equations representing the forward and backward flight path conditions are merged and simplified. This has the enormous advantage that the accuracy of the resulting value is independent of what the speed of sound in water actually is at the site in question, and at the time in question. The accuracy of the deduced measurement is therefore related solely to the accuracy with which time of flight and time difference can be measured and with modern electronics this accuracy is very high.

Direct flight path systems

The simplified equation for deriving line velocity in a direct flight path having transducers on both banks (see Figure I.6.22) is:

$$\overline{v_L} = \frac{L}{2cos\phi}\left(\frac{1}{t_2} - \frac{1}{t_1}\right) \tag{6.19}$$

where $\bar{v}_L$ = the average water velocity, at the elevation of the acoustic path, parallel to the axis of the channel, L = the length of the acoustic path AB, ϕ = the angle between the mean direction of flow and the acoustic path, t_1 = the travel time from transducer A to transducer B, and t_2 = the travel time from transducer B to transducer A.

Reflected flight path systems

The simplified equation for deriving line velocity in a reflected flight path having transducers on one bank only (see Figure I.6.22) is:

$$\overline{v_L} = \frac{L^2}{2L'}\left(\frac{1}{t_4} - \frac{1}{t_3}\right) \tag{6.20}$$

where $\bar{v}_L$ = the average water velocity, at the elevation of the acoustic path, parallel to the is the average water velocity, at the elevation of the acoustic path, parallel to the axis of the channel, L = the length of the reflected acoustic path ARB, L' = the projected distance in the line parallel to the mean direction of flow between upstream and downstream transducers, t_3 = the travel time from the downstream transducer A to the upstream transducer B via the reflector R on the opposite bank, t_4 = the travel time in the reverse direction.

In the reflected path variant of the time-of-flight system, it is not necessary to know the angle between paths and mean direction of flow.

Multipath sequencing

All the submerged paths in a multipath system (see Figure I.6.22) should be sampled sequentially at a rate sufficient to ensure that no significant changes in flow occur during the time taken to carry out one complete measurement cycle. It may be detrimental to transducers to fire them when they are out of the water, and system design should allow for those that are above water level to be inactive.

Each adequately submerged path should be sampled as frequently as possible, and over as long a period as possible, commensurate with the need to obtain a computed result before a significant change in flow can occur. A well-engineered system should be capable of executing at least 30 complete cycles of the entire transducer array per minute. Multiple sampling of individual flight-paths is an essential system feature, since there is a relatively high likelihood of any single sample failing. If reliable computation of individual path velocities is to be attained then a large number of samples should be available for averaging.

Flow computation for multipath systems

Flow measurement by the ultrasonic technique is analogous to flow measurement by a current meter. However, while the most commonly used current metering method is based on the estimation of mean velocity at a series of verticals dispersed across the gauged cross-section, in the ultrasonic method the velocity samples are horizontally orientated and vertically distributed.

Flow may be computed by exactly the same methods applied to a current meter discharge measurement. In practice, the methods which operate through the summation of a series of panel flows are likely to be most suitable. Panel dimensions can be derived from the fixed geometry of the ultrasonic measuring system. The flight paths themselves define the horizontals (analogous with the verticals of conventional gauging) and panel widths are defined by the differences in elevations of the acoustic flight paths.

Either the mid-section or the mean-section method of computation may be applied. In the mid-section method (see Figure I.6.23), each flight-path velocity should be taken to be the mean for the panel defined by the two lines mid-way between the path in question and the next highest and next lowest in the transducer array, and the panel length should be the river width at the elevation of the path in question. In the mean-section method, mean panel velocity should be the mean of the two velocities in the flight paths that limited the section, and mean panel length should be the mean of the two river widths at the elevations of the flight paths in question.

Special provisions need to be made to deal with the end panels in the computation. In the case of the mid-section method, the situation is straightforward. The top-most panel in the vertical stack is defined as being bounded below by the line that is halfway between the line of the top-most active flight path and the one immediately below it. It is bounded above by the water surface. The mean

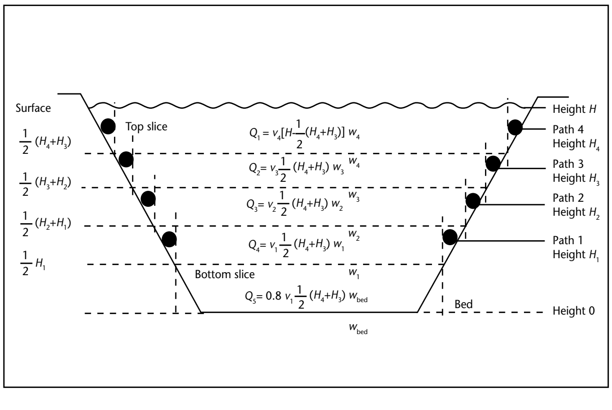

Figure I.6.23. Example of flow computation by mid-section method

velocity of the panel is the velocity measured in the top-most active flight path.

The mean length of the panel is the mean of the width of the channel in the plane half way between the top-most active flight path and the one immediately below it and at the water surface. The latter value may have to be derived by mathematical interpolation between the known widths of the channel in the plane of the highest submerged flight path, and in the plane of the lowest unsubmerged flight path.

The width of the panel is the difference in elevation between the highest active flight path and the water surface. Flow in the lowest panel in the vertical stack, the one nearest the channel bed, can be computed in analogous fashion by the mid-section method (see Figure I.6.23).

A flight path that is submerged, but which lies too close to the surface to be used because of the problem of signal reflection, will normally not be active and is ignored in the above computation, except as an end-point for the interpolation of channel width at the surface.

The treatment of end panels in the mean-section method requires the adoption of a different convention. Here, some assumptions have to be made as to what the surface and bed velocities are likely to be, since these values are required for the computation of mean panel velocity.

In practice, the more line velocity determinations there are available from active flight paths, and the closer they are to bed and surface, the smaller the resultant error associated with these aspects of velocity estimation. In station designs achieving closeness of flight path to bed, and narrow panel width, simple assumptions can be made concerning the estimation of bed and surface velocity, without significantly detracting from the overall accuracy.

Such assumptions include attributing the determined velocity in the highest active flight path to the whole of the top-most panel, and weighting of the determined velocity in the lowest active flight path to achieve an estimate of near-bed velocity. The latter factor may be determined empirically by current meter measurements, and will normally lie in the range 0.4 to 0.8.

Whatever steps are taken to prevent it, there will be times when individual flight paths in a multipath system are inoperative, either through physical damage to transducers or through failure of other parts of the instrumentation. In a well-designed system, flow determination should nevertheless continue to be possible, even if accuracy is somewhat reduced. The degree of reduction of accuracy will depend upon the ratio of failed paths to operational paths. If there is more than one failed path, residual accuracy will also depend upon the distribution of failed paths among the remaining operational ones. If they are adjacent, the resultant error will be greater. The most straightforward computational method is the mid-section method to calculate flow for the larger panel defined by the operational paths adjacent to the defective one, ignoring the latter's presence completely.

Flow computation for single path systems

In systems where only a single path determination of velocity is made, it may be necessary to establish a relation between this and the mean velocity in the cross section. Where this relation is stable, computation can be straightforward, with flow derived as:

$$Q = fvA \qquad (6.21)$$

where Q = the flow rate, f = an empirically derived function, v = the velocity determined in the single path, and A = the cross-sectional area.

The function f may be unity in systems where the single path is located at a depth that is representative of mean velocity in the cross-section. It may take some other value and still be stable, or it may itself vary with stage. It will normally be necessary to establish the value(s) of function f by velocity sampling at different depths and at different stages. This may be done by conventional current meter measurements, or by using the ultrasonic system itself, if provision has been made to move the transducers to different depths. Figure I.6.24 illustrates a typical relation between the velocity coefficient K [function f in equation 6.21] and stage. The velocity coefficient K is calculated by dividing mean velocity by line velocity.

6.4.9 **Concept of measurement redundancy**

The ultrasonic technique lends itself to the employment of design strategies that include over-provision of system components, so that a single component failure does not lead to a totally inoperative gauge. Large investments in instrumentation can make it appropriate to reconsider such design strategies. Most modern instrument models allow for over-provision of sensing elements. The incremental cost of over-providing velocity and level transducers will often be small compared to the overall system cost.

A multipath system incorporates some inherent component redundancy, in that the loss of one of the available transducer sets may only slightly reduce the overall accuracy of flow determination. It may, however, be necessary to consider carefully the minimum number of operational paths that is tolerable at minimum water levels, and to make some modest over-provision in this range.

The system element that will, in general, most benefit from replication is depth. In a multipath system, a missing or aberrant velocity determination has only a proportional effect upon overall system accuracy. If there is only one depth determining device, its loss is catastrophic to the system. It is recommended that provision be made for several of these devices, from which the instrumentation system selects the one giving the most probable determination of depth.

6.4.10 **System calibration**

Multipath and single path systems are both direct measuring systems if path angles and lengths are known. Multipath systems define the vertical velocity distribution and need little or no calibration. If the paths provided in the system design are

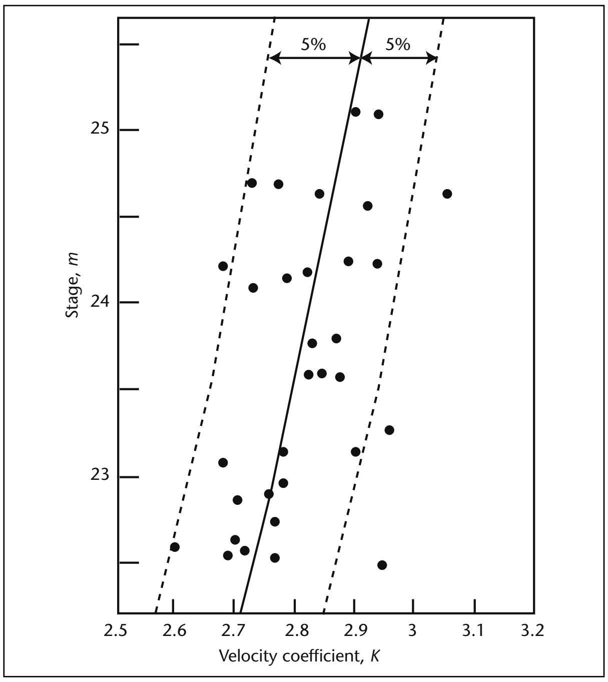

Figure I.6.24. Typical relation between velocity coefficient *K* and stage

sufficiently numerous, there should be no need for calibration.

With single path systems it will be necessary to calibrate to establish the relation between line velocity and mean velocity for the range of stages likely to be experienced and to conduct periodic checks to ensure that the relation remains stable. Calibration may be achieved independently using current meters or the ultrasonic system itself.

Single path systems with fixed transducers

The line (index) velocity method of calibration is used when a single pair of transducers is fixed in position and the system requires independent calibration. Figure I.6.24 shows an example of such calibration. This method yields a value of the coefficient K which relates line velocity, v_L to the average velocity, $\bar{v}$ and which also compensates for any error in the assumed values of path angle ϕ and path length L.

Single path systems with moveable transducers

This method relies upon the determination of line velocity provided by the system instrumentation being accurate and reliable. This should be checked by an independent determination using current meters.

Calibration of the gauge requires the transducers to be set at an elevation which gives the average velocity in the cross-section of that part of the flow range of particular interest to the user. When the stage changes the transducers will no longer give the average velocity. They will underestimate it for an increase in stage and overestimate it for a decrease in stage.

If the vertical velocity distribution is logarithmic, then the average velocity will be sampled by positioning the transducers at approximately 0.6 D from the surface, where D is the depth of flow above the mean bed level. The actual position will be found from the vertical velocity curve which, in turn, will be found by using the facility to move the system transducers in the vertical plane so as to make velocity determinations at different proportions of depth. In situations where the vertical velocity distribution is not logarithmic, appropriate analysis of the observed distribution will yield positioning information for the transducers.

Vertical velocity distributions should be determined for several stages of flow and values of $\bar{v}/v_d$ determined for respective values of d/D from 0.1 to 0.9, where $\bar{v}$ is the mean velocity for each curve, v_d is the line velocity at distance d from the surface, and D is the depth of flow.

A curve is then drawn to relate mean $\bar{v}/v_d$ values to d/D values (see Figure I.6.25). $\bar{v}/v_d$ is the adjustment factor or coefficient C_v by which the ultrasonic velocity at any distance d from the surface is greater or less than the average velocity in the cross-section.

The curve in Figure I.6.25 also indicates the optimum elevation of the transducers at the value of d/D when $C_v = 1$. The data points from which the curve is plotted will have some scatter, depending upon the geometric similarity of the vertical discharge curves. The standard deviation should be computed for each value of d/D at the 95% confidence level. When the stage changes, the appropriate values of C_v may be found from Figure I.6.25 by entering the curve at the new value of d/D. It should be noted that Figure I.6.25 is an example only, and new data need to be obtained and plotted as shown in the Figure.

6.4.11 Flow computation for single or multiple path systems using index-velocity methods

An alternative to flow determination by the methods discussed above is the use of the index-velocity methods discussed under the Acoustic Doppler

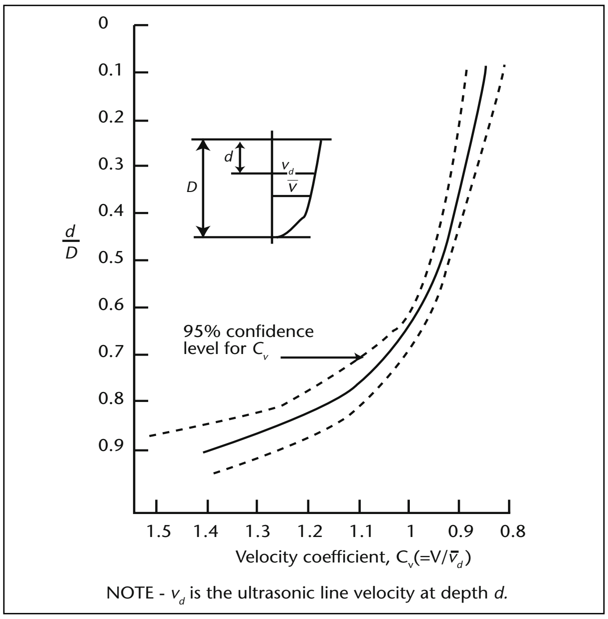

Figure I.6.25. Sample curve of *d/D* values against mean values of C_v

Velocity Meter (ADVM) section. For the index-velocity method, the AVM path numbers and configurations are not critical (except for the redundancy issue – more paths provide more redundancy). Also, the index-velocity method will work for fixed transducers, even for a range of stages provided that the stage is sufficient for operational purposes. To use the index-velocity method the AVM output path velocity is the index velocity (V_i). A stage-area rating is developed and discharge measurements are made to derive mean channel velocities. V_i is plotted versus mean channel velocity and best fit of the points is used as the index-velocity rating. Stage-area rating and index velocity rating methods and guidelines are identical to those discussed in the ADVM section and Chapter 2, Volume II. Note that a multipath system can be calibrated so that ratings can be developed for an average velocity from multiple paths and individual paths. This is useful if a path fails and flow computation can be accomplished simply by switching ratings so that the rating for the operational path is used. This method was used to develop ratings for AVMs used to compute discharge at four streamflow-gauging stations in the Chicago River system that are used in the Lake Michigan Diversion Project in Illinois as documented by Duncker and others (2006). The AVMs had a variety of configurations, including multiple paths in the vertical. The report describes for each station: (a) the AVM instrumentation, (b) stage-area and index-velocity ratings, (c) the methods used to compute discharge and (d) the methods used to estimate missing record. A method is derived using first-order error analysis for computing the total uncertainty of the discharge estimates at 5-minute to annual time scales and this method is applied to the discharges at the four stations.

6.5 TOTAL DISCHARGE METHOD USING ELECTROMAGNETIC FULL-CHANNEL-WIDTH COIL

The following sections describe a method of measuring total discharge in small channels by using the full-channel-width electromagnetic method. Although this method is somewhat limited because of the channel size for which it is practical, it has certain advantages which make it worthy of consideration. The following discussion of the total discharge method using an EM channel-width coil is based on ISO 9213 (1992). Several sections are taken directly from that reference. Other important references include Green and Herschy (1978) and Herschy (1977).

6.5.1 Principle of operation

The electromagnetic gauge operates on a principle similar to that of an electric dynamo. If a length of conductor moves through a magnetic field, a voltage is generated between the ends of the conductor. In the electromagnetic gauge, a vertical magnetic field is generated by means of an insulated coil which is located either above or beneath the channel. The conductor is formed by the water which moves through the magnetic field; the ends of the conductor are represented by the channel walls or river banks. The very small voltage generated is sensed by electrodes on the channel banks, and these are connected to the input of a sensitive voltage measuring device. The faster the velocity, the greater is the voltage which is generated.

The basic physical relationship between the variables is:

$$V = vbB \tag{6.22}$$

where V = the voltage generated, in volts, v = the average velocity of the conductor, in metres per second, b = the length of the conductor, in metres, and is equal to the width of the channel, and, B = the magnetic flux density, in teslas.

In the case of an operational gauge having an insulated bed and a square coil just wider than the channel, the voltage generated is approximately 0.8 times that given by equation 6.22. This reduction in voltage is caused by the shorting effect of the water upstream and downstream of the magnetic field.

Numerically, the empirical relationship (± 3%) is:

$$\varphi \approx vbH \tag{6.23}$$

where φ = the electrode potential, in microvolts, and H = the average magnetic field strength, in amperes per metre.

The physical relationship between B and H in free space, air or water is given numerically by:

$$B = H \times 4\pi \times 10^{-7} \tag{6.24}$$

where B and H are in different units.

In the case of an operational gauge having a non-insulated bed, the voltage generated is reduced by the shorting effect of the bed. The signal is reduced in proportion to the ratio of the bed to water conductivity. The higher the water conductivity, the less the reduction will be. The reduction should not be allowed to exceed a factor of 10, because the signal levels may be too low to be measured accurately and the reduction factor too variable to be determined with confidence. For water of high electrical conductivity (that is greater than 500 µS/m) (for example raw sewage flowing in a concrete or brick channel) the reduction is small, of the order of 10%. For a natural river, the reduction will be large and variable, governed by the bed and water conductivities, and the voltage generated may be only one-tenth that of a gauge having an insulated bed. The configuration is therefore normally only suitable for special situations.

For a typical case of an insulated channel having a full-width coil of 500 ampere turns, located beneath the channel, and a water velocity of 1 m/s, the voltage generated will be approximately $500\mu V$. It should be noted that this magnetic field decreases with vertical distance from the plane of the coil, in accordance with classic physical principles. The average field across the channel width should be calculated over the entire range of water depths if a theoretical calibration is to be obtained.

In the ideal case where the magnetic field strength is constant over the entire wetted section, then taking into account equation 6.23, the discharge, Q, is given by:

$$Q = vbH \tag{6.25}$$

and thus

$$Q = \frac{\varphi h}{H} \tag{6.26}$$

where h is the depth of water, in metres.

In an operational gauge in which the coil is mounted above the channel, the water near the bed will move in a less strong magnetic field relative to that near the surface, and so a non-linear relationship between Q and depth is necessary.

Normally this relationship is expressed by a simple equation of the form:

$$Q = \frac{(K_1 + h - K_2 h^2)\varphi}{H} \tag{6.27}$$

where K_1 and K_2 are constants.

The coil may be buried under the channel (see Figure I.6.26) or bridged across the channel above the highest water level (see Figure I.6.27).

A bridged coil configuration is normally used where the physical presence of the coil is aesthetically acceptable and not subject to vandalism. On wider channels, a bridged coil may, however, be impractical. A buried coil may be impractical to use in existing reinforced concrete channels. The choice of the type of coil will normally be made on a financial basis, as technically the only significant difference between the two types is the way in which the magnetic field varies with depth.

The channel cross-section may be rectangular, trapezoidal or circular. However, if there is a large range of depth and only a single coil located either above or beneath the channel, the magnetic field strength at different depths will differ. If this is the case, the contributions to the average generated voltage at various depths will not all have the same constant of proportionality and the average voltage will deviate from the ideal of $v \times b \times constant$ in

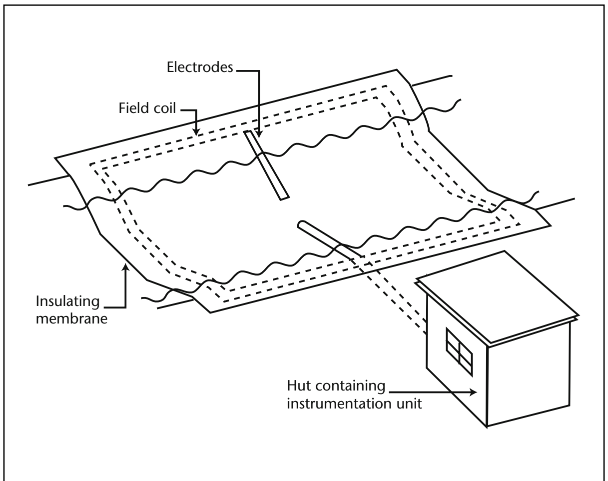

Figure I.6.26. Buried coil configuration for an electromagnetic gauge

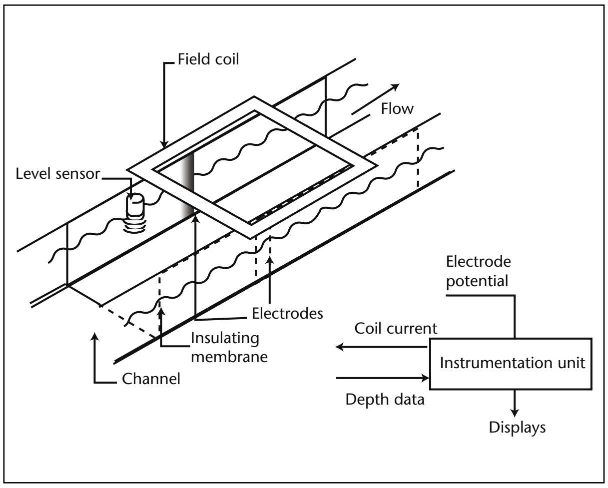

Figure I.6.27. Bridged coil configuration for an electromagnetic gauge

equation 6.25. If large vertical velocity gradients exist, the spatial integration will be erroneous. For a coil located beneath the bed, the magnetic field will be stronger near the bed than near the water surface.

The magnitude of the field is proportional to the electric current flowing through the coil. This current will normally remain relatively constant, but owing to ambient temperature fluctuations affecting the resistance of the coil cable and fluctuations of the mains voltage, it may vary slightly. It is necessary, therefore, to measure the current and to carry out a proportional correction to the flow calculation made.

Each volume of water flowing in the cross section will contribute to the electrode voltage, and for an ideal arrangement the voltage will be proportional to the true spatial integration of the velocity across the section. In practice, deviations from the ideal integration are small, which makes the method suitable for sites where the velocity profiles are irregular and variable The method is suitable, therefore, for sites where there is considerable aquatic vegetation, limited variable accretion, upstream bends and large obstructions in the reach.

With most natural channels the bed will be an electrical conductor and hence will reduce the induced voltage owing to electrical current leakage. It will usually be necessary to line the channel with an electrically insulating impervious membrane to reduce the current leakage to an acceptable level (see Figure I.6.26).

Problems are caused by the Earth’s magnetic field, the electrolytic effects of the electrodes in the water and external electrical interference. These may be overcome by reversing at regular intervals the magnetic field produced by the coil. This is achieved by reversing the current. Measurements are taken with the field first in one direction and then in the other.

6.5.2 **Selection of site**

A site survey should be carried out if necessary to measure any external electrical interference (for example power cables, radio stations or electric railways). Areas of high electrical interference should be avoided.

Owing to the high power consumption of the coil, equipment intended to measure flow continuously cannot reasonably be operated from its own power supply. Where the electrical power supply is derived from an external power source, the system should be arranged so as to restrict the amount of current passed through the ground. On sites where protective grounding is used, that is where the neutral line is grounded everywhere power is supplied, special permission to disconnect the neutral-to-ground link on the power supply may have to be requested. The power supply voltage should be within ± 20% of its nominal value. A 1 *kW* source of electrical energy should be available for river gauges.

The site should afford adequate on-bank working space for handling the membrane and cable during construction and good access for operation and maintenance.

Since the magnetic field reduces as distance from the coil increases, it is recommended that the ratio of coil width to the vertical distance between the coil and any water being measured be not less than 2. For narrow deep channels, this may mean that a coil many times wider than the channel is necessary.

For non-insulated channels the signal attenuation due to bed leakage increases as the width-to-depth ratio increases. In this case, it is recommended that the width-to-depth ratio does not exceed 10. However, for insulated channels, operation with width-to-depth ratios of 200 is possible.

The site characteristics should be such that the calibration of the station can be checked by an alternative method.

Sites should be selected where there is no spatial variation in water conductivity. Whether or not the channel is insulated, the accuracy of the method will be reduced if the conductivity is not uniform across the section. Gradual variations with time are unimportant provided that the spatial uniformity of the conductivity is maintained. This requirement makes an electromagnetic gauge unsuitable for channels in which fresh water flows over saline water, which often occurs in estuaries. Provided that these requirements are met, the quality of the water will not affect the operation of the gauge. Similarly the conductivity of the water will not affect the operation of a gauge in an insulated channel provided that it exceeds 50 $\mu S/m$.

Rapid changes in water quality with time will produce changes in the steady (or slowly changing) voltages between the electrodes. Such changes will bias the amplifier stages of the electronic system.

In a non-insulated channel the accuracy of measurement is reduced. However, in this situation a preliminary survey should be carried out to measure the conductivity of the water and bed to estimate the signal reduction before deciding whether the site is suitable.

6.5.3 **Calibration**

After a station has been installed, a rating equation, for example in the form of equation 6.27, based on the coil current, voltage, and range of stage should be established and checked by using a current meter or other method.

Care should be taken to ensure that the act of check gauging does not affect the working of the electromagnetic gauge. Any activity in the river should be carried out outside the limits of the coil and insulated section to reduce electrical interference from current-meters and support cables and to avoid wave action generated by wading and boats.

6.5.4 **Applications**

The electromagnetic gauge is particularly suited for measuring the flow of untreated domestic waste effluent and treated effluent discharge into rivers, the flow of potable water in a treatment works and the flow of cooling water in power stations. Different versions of the electromagnetic gauge are suitable for measuring flow in rivers, partly-filled pipes or culverts carrying storm water, raw effluent or sewage.

The advantages of the method include the following:
(a) Tolerates aquatic vegetation;
(b) Tolerates entrained air;
(c) Tolerates temperature stratification;
(d) Tolerates suspended sediment or floating debris in the water;
(e) Tolerates deposited sediment or other accretion on the channel bed;
(f) Tolerates variable backwater;
(g) Tolerates upstream inflows; however, if the inflow conductivity is significantly different from that of the main channel, there should be sufficient distance for adequate mixing;
(h) Can detect a minimum velocity of about 0.001 m/s;
(i) Tolerates irregular velocity profiles, including skew flow and severe eddy currents in the measurement area;
(j) Is suitable for gauging very shallow water;
(k) Inherently integrates the velocity profile over the entire channel cross-section;
(l) Affords a wide range of stage and discharge measurements;
(m) Provides adequate quality of measurement;
(n) Does not constrict the flow;
(o) Can measure reverse flow.

6.5.5 **Design and construction**

The electromagnetic gauging station should consist of the following elements (see Figures I.6.26 and I.6.27):
(a) A field coil installed beneath or above the channel;
(b) A pair of electrodes, one on each side of the channel;
(c) An insulating membrane, normally necessary;
(d) An instrumentation unit, including a coil power supply unit;
(e) Equipment housing;
(f) Water level measuring device.

The field coil

The sensitivity of the equipment to the flow is improved by increasing the strength of the field. This is proportional to the number of turns in the field coil and also to the current flowing through the coil. The energy required to produce the magnetic field in a coil of a certain size, number of turns and current is inversely proportional to the cross-sectional area of the conductors which make up the coil. It is also proportional to the electrical resistivity of the material used for the conductors. A compromise should be made, therefore, between the capital cost of the cable, electricity running costs and strength of electrical interference, and the resolution required in the determination of flow. In practice, a coil with a square configuration slightly larger than the channel width and of some 200 to 1 000 ampere turns, should cover most practical situations.

Any electrical leakage between the coil and the water in the channel will create voltages across the width of the channel. These voltages cannot be separated from those generated by the movement of water through the magnetic field and will produce an apparent offset in the readings of the equipment.

If the coil is located beneath the channel, the use of a polyethylene-insulated cable with a polyethylene outer sleeve is recommended. In all cases, the insulation between the coil and earth (or the water surrounding the coil) should exceed $5 \times 10^8\ \Omega$.

The coil should be installed in ducting (normally of about 250 mm diameter) to afford access for maintenance of the cable. Construction constraints normally require the coil to be square in plan.

For a bridged coil, a lesser grade of insulation such as Poly Vinyl Chloride (PVC) is acceptable. The coil should span the full width of the river above the maximum stage at which measurements are required. If the coil is likely to be submerged, it should be able to withstand impact by floating debris. If meaningful measurements are required in this condition, the insulation should exceed $5 \times 10^8\ \Omega$ when submerged, and no should shall be in contact with the water.

It is recommended that the coil be wound with a multi-core cable (for example 12 cores each of 4 mm cross-section, insulated from each other, sheathed overall) to simplify installation. It is recommended that the cable is not armoured with steel otherwise the field may be partially contained in the armouring. Non-ferrous armouring is permissible but the armouring should be insulated from the water to avoid leakage of the induced signal.

If the equipment is installed in a potentially explosive atmosphere, the coil should be of limited power and should be protected against accidental mechanical damage. For such duty, a typical coil is a 300 turn, 4 mm cross-section copper conductor, with a maximum possible current of 5 A. The coil should be encased in an approved plastic trunking, and surrounded by concrete 50 mm thick. Alternatively, double-insulated conductors wound inside an approved glass-reinforced plastic trunking with an approved junction box may be used.

The frequency at which the magnetic field is reversed should be low enough to permit a stable field to be established, but not so low as to permit polarization effects to become significant. Frequencies of the order of 1/2 or 1 cycle every second (0.5 Hz to 1 Hz) are recommended. The coil current should be either measured or, alternatively, stabilized at a fixed value.

The electrodes

It is recommended that the electrodes be made from stainless steel strip or tube. In clean water rivers, they should be covered by a mechanical filter to reduce varying oxidation potentials generated by wave action of the water. Typically, the width of flat electrodes may be in the range 50 mm to 100 mm. Tubular electrodes should be of the order of 10 mm to 20 mm diameter. The filter may take the form of a perforated plastic tube of 80 mm diameter placed around the electrode.

In channels containing polluted water which is liable to putrify, the electrode mounting should not permit such water to become trapped in pockets or crevices near the electrode, and no mechanical filter should be used.

The potential between the electrodes is likely to reach several hundred volts in the event of a lightning strike in the vicinity of the gauge. To protect the instrumentation from such an event a Zener barrier is essential between the electrodes and the input to the instrumentation. A Zener barrier is a voltage and current-limiting circuit which protects against high voltage inputs from lightning strikes. It also reduces the risk of hazardous voltages being presented to the electrodes by faulty electronic equipment.

The inductive coupling between the signal cable and the coil should be a minimum. This can be achieved by the feed from the electrode on the far bank passing in a straight line through the coil centre to bisect the plan area of the coil. An alternative arrangement is to take two signal cables from the far bank electrode. One cable passes through the same ducting as the upstream coil cable and the second electrode cable passes through the downstream coil ducting. The signals from these two cables are added together using a resistance network. Ducting for the electrode cables either should cross the channel beneath the insulating membrane (if used) or should be bridged across the channel.

In open channels the electrodes should be supported in guides mounted on the walls or banks on either side of the channel. Such mountings should extend throughout the full depth of flow. The guides may consist of slotted plastic rods for flat electrodes or perforated plastic tubing for tubular electrodes. Alternatively, the electrodes may be moulded into glass-reinforced plastic units, with only one face of the metal electrode exposed. The guides should be secured to the channel walls or banks, but the membrane should not be punctured (see Figure I.6.27).

Insulating membrane

If the channel is to be lined, an insulating membrane should be used which is tough enough to withstand the stresses involved. A high density polyethylene sheet 2 mm or 3 mm thick, or equivalent material, is recommended. The resistivity of the material should be greater than $10^{12}\ \Omega$m.

The membrane should be mechanically anchored and sealed at the leading and side edges to protect against local scour and seepage. The lining should be laid and secured in such a way as to prevent subsequent movement. The bed at the trailing edge should be protected against damage by local scour.

In practice the membrane may be covered by a variety of materials to protect it against damage. Acceptable protection on the river bed is a 100 mm thick layer of concrete (this should not be reinforced). The banks of the river may be protected by rock-filled non-metallic gabions or, in some instances, a layer of concrete. In a rectangular channel, the membrane may be set behind a vertical wall of concrete or similar material, such as concrete blocks or clay bricks. No metal reinforcement or wire rope should be used within the insulated reach.

The membrane should not be punctured, except along the edges for anchoring purposes. For this reason the take-off point to a stilling well should be beyond the limits of the membrane.

In a concrete channel the upstream leading edge and sides of the membrane should be battened to the concrete or fixed by similar means. In a river the edges of the membrane may be anchored by concrete bagging to trap the membrane in a trench.

It is recommended that the length of the lining is not less than 1.5 times the channel width at the maximum stage at which measurements are to be made. The lining should be centred with respect to the coil centre.

Instrumentation unit

The instrumentation should consist of a power supply to drive the coil, a sensitive detector to measure the electrode voltage and other electronic processing units to compute the discharge from the site parameters and water depth. The electronic system detects and measures the required signal in the presence of interference the magnitude of which may be many thousands of times greater. To obtain meaningful determinations of flow, measurements should be averaged over a period of several minutes.

An in situ data-logging system may be included with an instrument to record data on one or more of a variety of recording devices, such as an EDL.

To check the equipment, a digital output display should give a continuous display of discharge and depth with built-in indicator alarms to detect electronic faults. It should also be possible to display other fundamental variables, including the electrode potential, coil current and engineering parameters in the instrumentation, such as power supply voltages. Where a non-insulated channel is used, the display should be capable of indicating the water resistivity and the measured bed resistance.

The equipment should withstand without damage the disconnection or reconnection of any of its major assemblies. In the event of the mains power supply voltage falling temporarily to a low value, no damage should be sustained. The equipment should be capable of withstanding periods of 12 hours with no power before dampness causes a temporary deterioration in performance. No permanent damage should occur for power failures of less than 7 days. The equipment should automatically return to correct operation upon restoration of the power source.

Equipment housing

The electronic system should not be subjected to temperatures outside its design range when in operation. The housing should be secured against the ingress of corrosive or explosive gases, if these are likely to be present. Ventilation and sufficient working space should be provided to enable maintenance personnel and field staff to work in the housing for periods of several hours.

Water level gauge

A water level measuring device should be interfaced with the electromagnetic processor. The equipment datum should be at the mean level of the insulation at the bottom of the channel, below the level of the bottom of the electrodes. If the insulation is covered with a protective layer of concrete or other non-conducting material, then the datum is the mean level of the top of this covering. The zero point of the gauge should be at a datum preferably at or below the point of zero flow.

6.5.6 Measurement tolerances

If the equipment is established on a non-insulated channel, regular measurement of water conductivity and bed leakage is required. The ratio between these parameters should be determined to an uncertainty of within 5% and the value thus determined used in the flow determination.

The uncertainty in the measurement of coil current should not exceed 1% of the measured value. The uncertainty in the measurement of voltage generated by the movement of water in the magnetic field should be ± 0.5 μV or ± 1% of the actual value, whichever is greater.

The signals generated by the movement of water through the magnetic field produced by the coil will vary from a few to several hundred microvolts. Typical values by which the signals will be modified by interference from various sources are given in Table I.6.4. The direct current polarization potentials will change with changing water quality. When gauges are being designed for use in foul sewers, polarization changes of 0.01 V/min should be allowed for.

Table I.6.4. Typical magnetic field signal interference of full channel EM gauge

Source	*Interference*
Power frequency (50 Hz or 60 Hz)	± 1.5 V between electrodes and ground ± 5 mV between electrodes
Radio frequency	± 40 mV between electrodes and ground ± 5 mV between electrodes
Lightning	± 1 000 V between electrodes and ground ± 300 V between electrodes
Polarization	± 2 V between electrodes and ground ± 1 V between electrodes

6.5.7 Flow computation

The equipment should measure the difference in electrical potential between the electrodes, which is generated by the flow of water, and should reject the electrical interference. To achieve this, the electronic equipment should:

(a) Control accurately the switching of the coil;
(b) Measure the coil current when it is stable;
(c) Protect the electrode potential measuring circuits against the electrical surges induced in the electrodes and connecting cables when the coil current is reversed. This is necessary because the induced signal is large (perhaps equivalent to a potential of 5 000 μV) and cannot be distinguished from the required water-induced signal except by its time of occurrence;
(d) Measure the polarization potential between the electrodes and provide a bias to the potential measuring circuits so that they can operate within their linear region. The polarization potential is large, perhaps 10^6 times the required resolution of the electrode potential;
(e) Measure the potential between the electrodes (ignoring common-mode potentials between electrodes and ground) and obtain an average over each coil cycle;
(f) Calculate the component of the average potential between the electrodes which is in phase with the magnetic field. This component will be a measure of the signal generated by the flow of water. The calculation should take into account any changes in bias introduced to enable the circuits to operate in their linear range;
(g) From the average potential which is generated by the flow of water, calculate flow in accordance with equation 6.27. H can be calculated from the coil current (which may be a variable) and the dimensions of the coil and the channel.

The electrical interference is regarded as random, and so will in the longer term average to zero. In order to obtain consistent measurements of flow, the data should be averaged over periods generally between 2 min and 15 min, depending on the degree and frequency of the interference.

The averaging period should be accurately defined and should be of limited duration so that comparisons can be made between the flow computed using the electromagnetic technique and that computed using an alternative calibration technique. A moving average technique, updated every 0.5 min and representing flow averaged over 15 min, is recommended for river use.

6.5.8 Uncertainties in flow measurement

Generally for an insulated channel, the random uncertainty at the 95% confidence level in the value predicted from the calibration relation may be of the order of ± 2%. The corresponding uncertainty for a non-insulated channel may be much higher, depending on the site and the number of observations taken during the calibration, and may be of the order of ± 10%. These values are based on observations taken at a large number of sites.

The uncertainty in a single determination of discharge may be calculated by combining the component uncertainties using the root-mean-square method. Values for these component uncertainties should be estimated independently for each site.

The component uncertainties are:

(a) The dimensions of the coil, and its position relative to the channel bed;
(b) The variability of the water velocity profile. This is important on those sites where the coil

does not produce a near-constant magnetic field over the entire channel cross-section;
(c) The measurement of the coil current. The magnetic field strength H is proportional to the current and hence the computed flow is inversely proportional to the current;
(d) The measurement of depth, relative to the channel bed; and
(e) The measurement of the three components of the electrode potential:
    (i) Fixed offset, observable at zero flow. On most rivers this value can only be inferred from a graph of electrode potential and flow, extrapolated to zero flow. The value of the offset should not exceed 0.5 μV;
    (ii) Errors in the slope of the relation between the actual and the indicated electrode potential, which can be assessed using apparatus to generate simulated electrode potentials; and
    (iii) Variability with time, caused by electrical interference, which can be assessed by comparison of the variability's of computed flow for different lengths of integration time.

Generally, it may be stated that the uncertainty in a single determination of discharge will be of the same order as that of a current-meter measurement.

The minimum detectable velocity may be expected to be approximately 0.001 m/s. The ratio of the minimum detectable discharge to the maximum is approximately 1:1,000. For equipment connected to an insulated channel, an uncertainty of ± 5% of the measured value may be expected when the average velocity is in excess of 0.1 m/s.

For a more detailed discussion of uncertainties see Chapter 10 of this Manual. See also ISO 5168 (2005).

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CHAPTER 7

MEASUREMENT OF DISCHARGE BY PRECALIBRATED MEASURING STRUCTURES

7.1 GENERAL

The practice of measuring discharge by observing the head on measuring structures, or manmade controls, is one that is followed in many countries. This chapter provides only a brief discussion of this highly specialized subject because it is treated in detail in WMO Technical Note No. 117, *Use of Weirs and Flumes in Stream Gauging* (WMO-No. 280) (available under HWRP Publications), and in numerous ISO standards. See the references at the end of this chapter for a complete listing of ISO standards.

7.2 TYPES OF PRECALIBRATED MEASURING STRUCTURES

Laboratory ratings are presented for the following types of measuring structures that may be used as gauging stations:

(a) Thin-plate weirs:
   - (i) Triangular-notch weirs (Figure I.7.1);
   - (ii) Rectangular weirs (Figure I.7.2).

(b) Broad-crested weirs (any weir not of the thin-plate type):

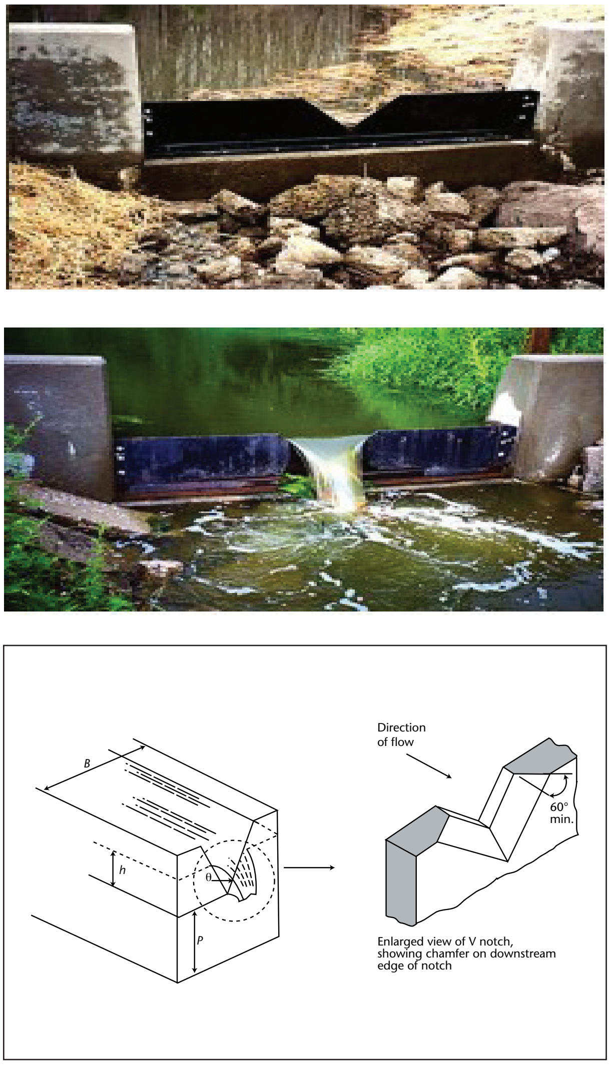


Figure I.7.1. The triangular thin-plate (squared-edged) weir (V-notch)

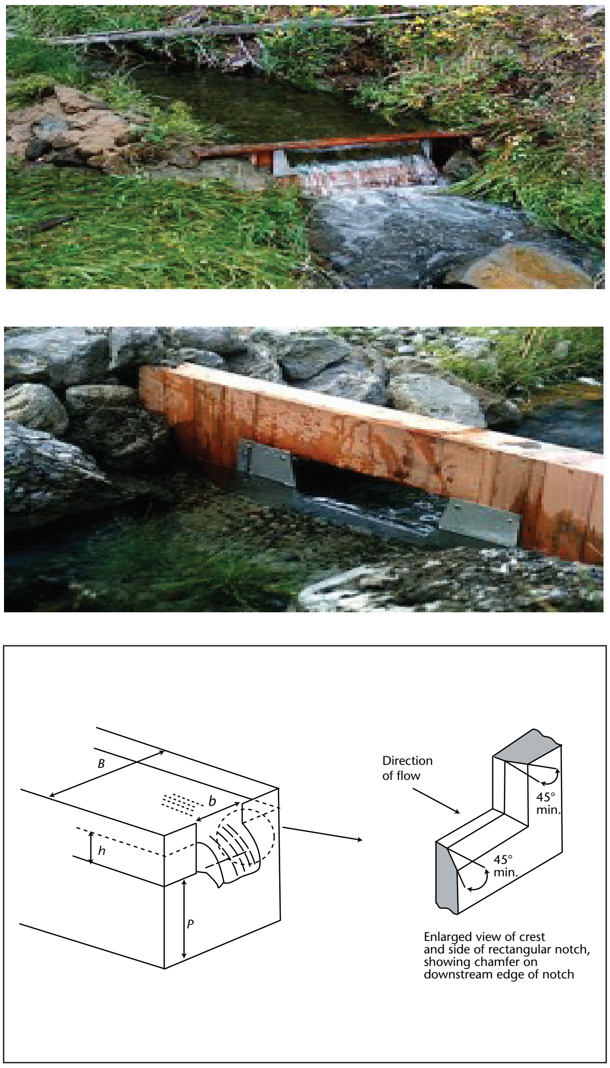
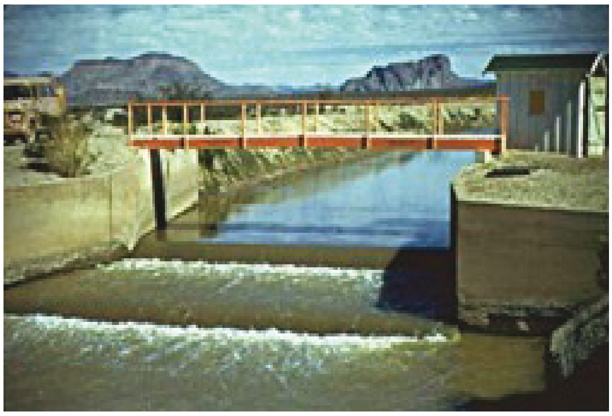


Figure I.7.2. The rectangular thin-plate (square-edged) weir

   (i) Round-nosed horizontal-crested weirs (Figure I.7.3);
   (ii) Triangular-profile Crump weirs (Figure I.7.4);
   (iii) Rectangular-profile weirs (Figure I.7.5);
   (iv) Flat-V weirs (Figure I.7.6).
(c) Standing-wave flumes:
   (i) Rectangular-throated standing-wave flumes (Figure I.7.7);
   (ii) Trapezoidal-throated flumes (Figure I.7.8);
   (iii) U-shaped flumes (Figure I.7.9);
   (iv) Parshall flumes (Figure I.7.10);



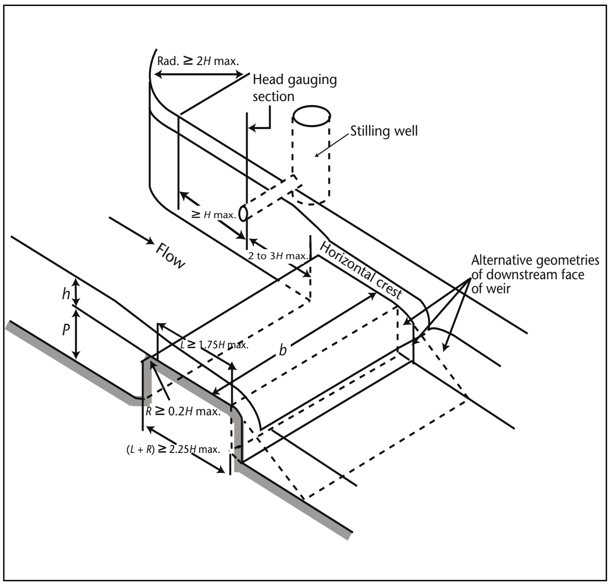


Figure I.7.3. Round-nosed horizontal-crest weir

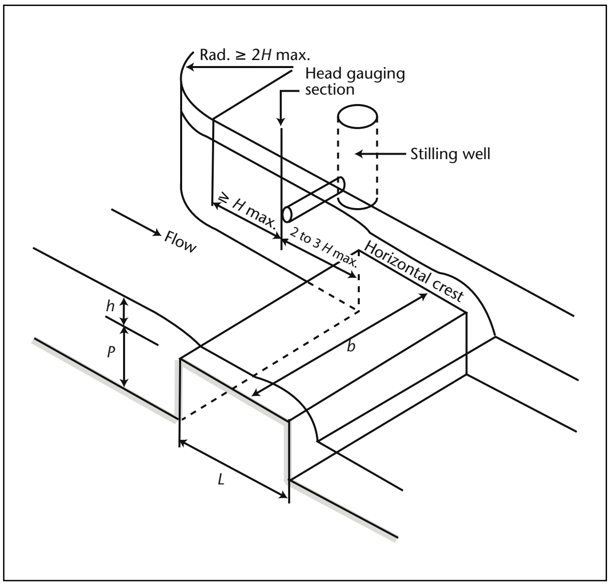


Figure I.7.5. Rectangular profile weir

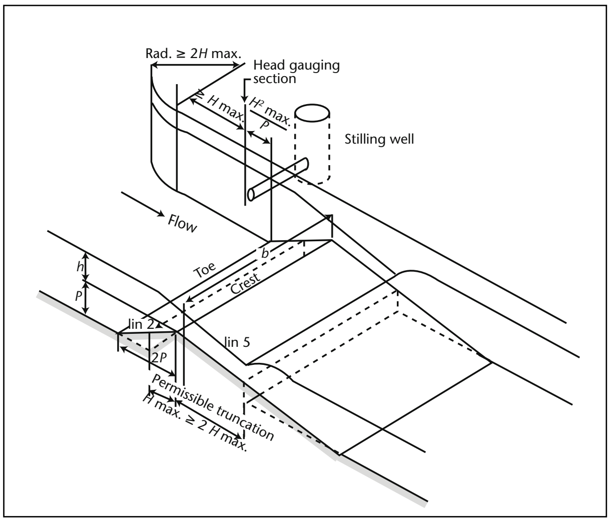


Figure I.7.4. Triangular profile weir

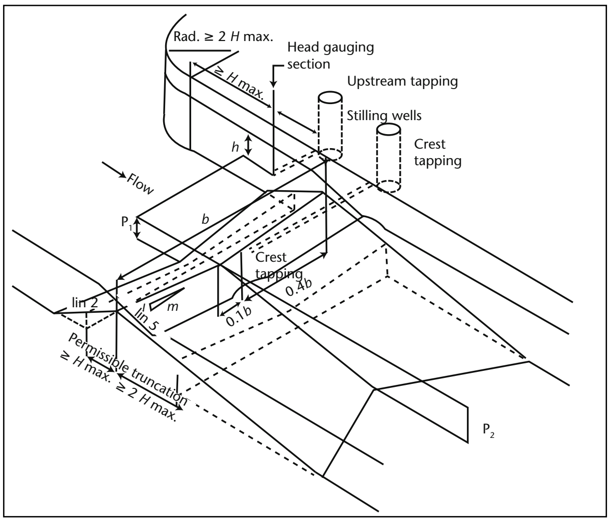


Figure I.7.6. Triangular profile flat-V weir

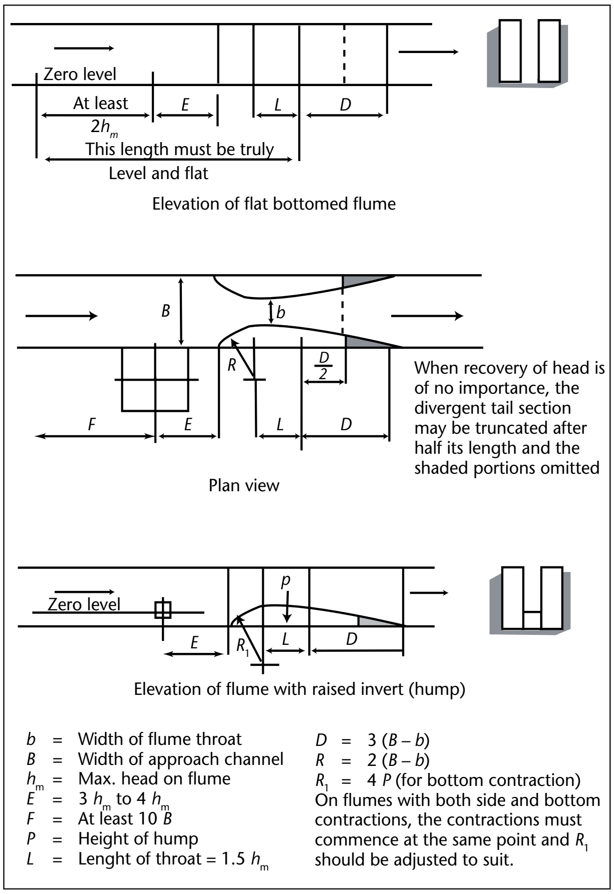


Figure I.7.7. Rectangular throated flume

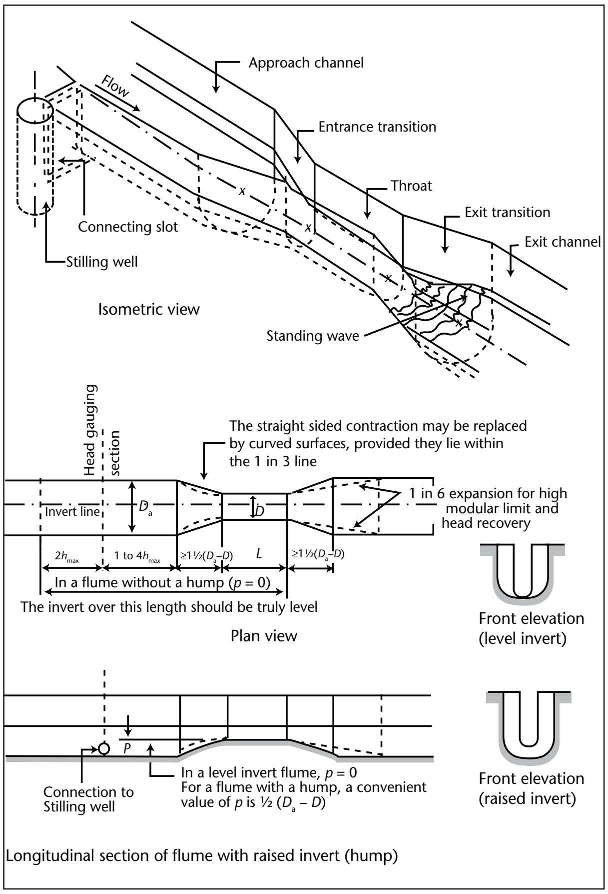


Figure I.7.9. U-shaped flume

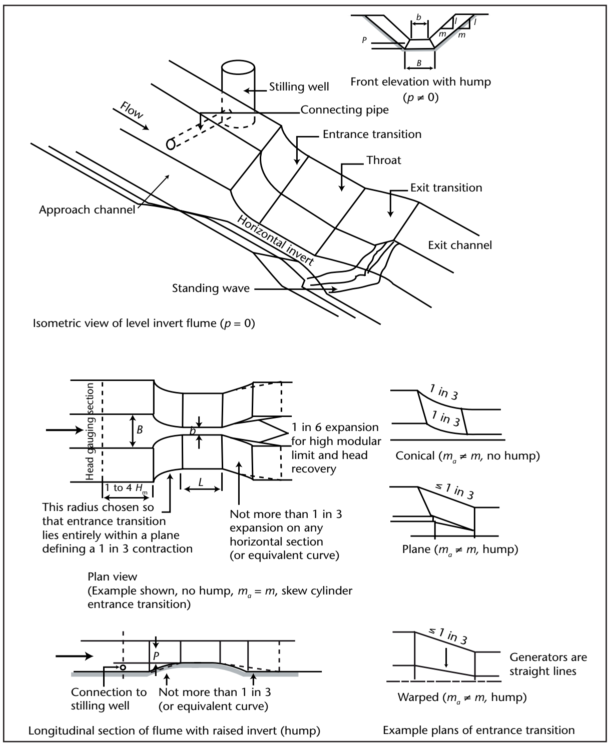


Figure I.7.8. Trapezoidal throated flume

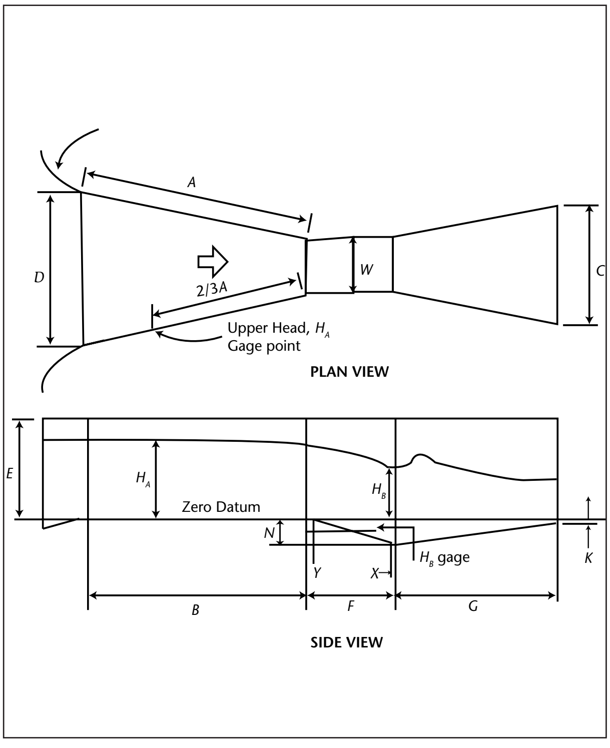


Figure I.7.10. Parshall flume

Thin-plate weirs are generally used in small clear-flowing streams, particularly where high accuracy is desired and adequate maintenance can be provided, as in small research watersheds. Flumes are preferred for use in small streams and canals that carry sediment and debris, and in other situations where the head loss (backwater) associated with a thin-plate weir is unacceptable. Certain types of flumes may also be used under conditions of submergence, as opposed to free-flow conditions, thereby permitting them to operate with even smaller head loss but with some loss of accuracy. The broad-crested weirs are commonly used in the larger streams.

7.3 TRIANGULAR (V-NOTCH) THIN-PLATE WEIR

The equation for discharge through a triangular, V-notch, thin-plate weir is as follows:

$$Q = \frac{8}{15}\sqrt{2g}C_D \tan\frac{\theta}{2}h^{5/2} \tag{7.1}$$

where Q = discharge, C_D = coefficient of discharge, θ = angle included between sides of notch, and h = gauged head referred to vertex of notch.

The coefficient C_D varies from 0.608 at h = 0.050 m to 0.585 at h = 0.381 for a 90° notch. A table of discharges for 90°, 1/2 90° and 1/4 90° V-notches is given in pages I.7.14 to I.7.27.

7.4 RECTANGULAR THIN-PLATE WEIR

The basic discharge equation for a rectangular thin-plate weir is as follows:

$$Q = C\frac{2}{3}\sqrt{2g}bh^{3/2} \tag{7.2}$$

from which the equation 7.3 have been evolved.

7.4.1 Kindsvater-Carter

The Kindsvater-Carter discharge equation is:

$$Q = C_e\frac{2}{3}\sqrt{2g}b_e h_e^{3/2} \tag{7.3}$$

where b_e = the effective width of the notch = $b + k_b$ and k_b = the effective head on the crest = $h + k_h$ where k_b and k_h are absolute measures of the combined effects of viscosity and surface tension. Kindsvater-Carter found k_h to have a constant value of 0.90 mm. Values of k_b are obtained from Table I.7.1 (see ISO 1438-1, 1980).

Table I.7.1. Values of k_b for related values of *b/B*

b/B	k_b, *mm*
0.10	2.4
0.20	2.4
0.40	2.7
0.60	3.6
0.80	4.2
0.90	3.1
1.00	0.9

For the full width (suppressed) weir (b/B = 1.0), the coefficient of discharge C_e is found from:

$$C_e = 0.602 + 0.075\frac{h}{P} \tag{7.4}$$

For other values of *b/B* (weirs with end contractions), C_e can be obtained from Table I.7.2.

Table I.7.2. Values of C_e for related values of *b/B*

b/B	C_e
0.9	0.598 + 0.064 *h/P*
0.8	0.596 + 0.045 *h/P*
0.7	0.594 + 0.030 *h/P*
0.6	0.593 + 0.018 *h/P*
0.4	0.591 + 0.0058 *h/P*
0.2	0.589 – 0.0018 *h/P*
0	0.587 – 0.0023 *h/P*

7.4.2 Rehbock

The Rehbock discharge equation for the full width weir is:

$$Q = \frac{2}{3}\sqrt{2g}C_D b h_c^{3/2} \tag{7.5}$$

where $h_c = h + 0.0012$ m, and $C_D = 0.602 + 0.083\, h/P$.

7.4.3 Hamilton-Smith

The Hamilton-Smith discharge equation for weirs with fully developed contractions is:

$$Q = \frac{2}{3}\sqrt{2g}C_D bh^{3/2} \tag{7.6}$$

where $C_D = 0.616\,(1 - 0.1h/b)$.

7.4.4 Hydraulics Research Station equation (HRS) (White 1977)

The HRS discharge equation is:

$$Q = 0.5621 + \frac{0.153h}{P} b\sqrt{g}(h + 0.001)^{3/2} \qquad (7.7)$$

In all the above equations gauged head is used and the total head correction is included in the equations. The limits of application of the above equations are listed in Table I.7.3.

The location of the head measurement section is recommended as being three to four times the maximum head upstream of the weir except in the case of the HRS equation where the recommendation is 2.67*P*.

7.5 BROAD-CRESTED WEIRS

The general discharge equation for broad-crested weirs is:

$$Q = C_D\sqrt{g}bH^{3/2} \qquad (7.8)$$

where H = total head (gauged head plus velocity head), or:

$$Q = C_V C_D (2/3)^{3/2}\sqrt{g}bh^{3/2} \qquad (7.9)$$

where h = gauged head and C_V may be obtained from Table I.7.4.

7.5.1 Triangular profile (Crump) weir

Under modular flow conditions, for h greater than 0.1 m, C_D is constant and equals 0.633. For h less than 0.1 m, C_D is found from the following equation:

$$C_D = 0.633\left(1 - \frac{0.0003}{H}\right)^{3/2} \qquad (7.10)$$

where H is in metres.

Table I.7.4. Values of coefficient of approach velocity C_V for broad-crested weirs

C_Dbh/A	C_V
0.1	1.003
0.2	1.010
0.3	1.020
0.4	1.039
0.5	1.057
0.6	1.098
0.7	1.146
0.8	1.217

7.5.2 Round-nosed horizontal crest weir

The value of C_D is given by the following equation:

$$C_D = \left(1 - \frac{0.006L}{b}\right)\left(1 - \frac{0.003L}{h}\right)^{3/2} \qquad (7.11)$$

where L is the length of the horizontal section of the crest in the direction of flow.

7.5.3 Rectangular profile weirs

The coefficient of discharge C_D is constant between the limits of h/L = 0.08 and 0.33 and of $h/(h + P)$ = 0.18 and 0.36 and = 0.848. To obtain the coefficient in the variable range it is necessary to multiply the basic coefficient by a factor F. This factor may be obtained from Table I.7.5.

7.5.4 Flat-V weir

The discharge equation for a flat-V weir is:

$$Q = \frac{4}{5} C_D\sqrt{g}nH^{5/2} \qquad (7.12)$$

when H is within the V-notch, or:

Table I.7.3. Limits of application for rectangular thin-plate weir equations

Equation	*Minimum head h (m)*	*Minimum width b (m)*	*Minumum crest height P (m)*	*Maximum h/P ratio*
Kindsvater-Carter	0.030	0.15	0.10	2.0
Rehbock	0.030	0.30	0.30	1.0
Hamilton-Smith	0.075	0.30	0.30	–
HRS	0.030	0.40	0.15	2.2

Table I.7.5. Values of coefficient correction factor *F* for values of *h/L* and *h/(h +P)* for rectangular profile weirs

h/L	*h/(h + P)*			
	0.600	*0.500*	*0.400*	*0.350*
0.35	1.059	1.032	1.011	1.001
0.40	1.062	1.035	1.014	1.002
0.45	1.066	1.040	1.018	1.007
0.50	1.074	1.047	1.025	1.014
0.60	1.094	1.068	1.044	1.034
0.70	1.120	1.092	1.070	1.058
0.80	1.144	1.115	1.093	1.080
0.95	1.152	1.123	1.101	1.089

$$Q = \frac{4}{5} C_D \sqrt{g}\, nH^{5/2} \left[1 - \left(1 - H'/\,H\right)^{5/2}\right] \qquad (7.13)$$

when H is above the V (that is greater than H'), where n = crest cross slope (1 vertical : n horizontal), H' = difference between lowest and highest crest elevations, C_D = 0.63 for weirs having a 1:2/1:5 profile.

7.6 STANDING-WAVE FLUMES

Discharge equations for four types of standing wave flumes are described in the following sections.

7.6.1 Rectangular throated flume

The discharge equation is:

$$Q = (2/3)^{3/2} \sqrt{g}\, C_V C_D b h^{3/2} \qquad (7.14)$$

Values of C_V may be obtained from Table I.7.6 and of C_D from Table I.7.7.

7.6.2 Trapezoidal-throated flumes

The discharge equation for a trapezoidal-throated flume is:

$$Q = (2/3)^{3/2} \sqrt{g}\, C_V C_S C_D b h^{3/2} \qquad (7.15)$$

where C_S is a shape coefficient.

The direct application of this equation is not very convenient because h ≠ H and a theoretical calibration for a range of discharge in one computation is recommended (stage-discharge relation). See ISO 4359 (1983).

Table I.7.6. Values of velocity coefficient, C_V

b/B	*h/(h + P)*				
	1.0	*0.8*	*0.6*	*0.4*	*0.2*
0.10	1.002	1.001	1.001	1.000	1.000
0.20	1.009	1.006	1.003	1.001	1.000
0.30	1.021	1.013	1.007	1.003	1.001
0.40	1.039	1.024	1.013	1.006	1.001
0.50	1.064	1.039	1.021	1.009	1.002
0.60	1.098	1.058	1.031	1.013	1.003
0.70	1.147	1.083	1.043	1.018	1.004
0.80	–	1.115	1.058	1.024	1.006
0.90	–	–	1.076	1.031	1.007
1.00	–	–	1.098	1.039	1.009

Table I.7.7. Values of discharge coefficient, C_D

L/b	*h/L*			
	0.70	*0.50*	*0.30*	*0.10*
0.2	0.992	0.990	0.984	0.954
1.0	0.988	0.985	0.979	0.950
2.0	0.982	0.979	0.973	0.944
3.0	0.976	0.973	0.968	0.938
4.0	0.970	0.968	0.962	0.932
5.0	0.965	0.962	0.956	0.927

7.6.3 U-shaped (round-bottomed flumes)

The discharge equation for a U-shaped, round-bottomed flume, is:

$$Q = (2/3)^{3/2} \sqrt{g}\, C_V C_U C_D D h^{3/2} \qquad (7.16)$$

where C_U is a shape coefficient and D is the diameter of the base of the U-shaped flume.

As in the case of the trapezoidal flume, the direct application of this equation is not very convenient because h ≠ H and a technique of successive approximation is required. See ISO 4359 (1983).

7.6.4 Parshall flumes

The flume consists of a converging section with a level floor (see Figure I.7.10), a throat section with a downward sloping floor, and a diverging section with an upward sloping floor. Calibration is carried out against a piezometric head H_A measured at a prescribed location in the converging section, the downstream head H_B being measured in the throat. H_A and H_B are each set with the zero of the gauge at the elevation of the crest, or level floor. Table I.7.8 summarizes the dimensions for 22 flumes of various sizes. The free flow stage-discharge relations for all flumes were derived experimentally. These relations are given in Table I.7.9 with both the range in head and discharge (Bos, 1976). The installation in the field, however, should be checked and calibrated with current meter measurements. If the Parshall flume is not to be operated above a submergence ratio, H_B/H_A, of 0.60 the flume may be truncated and the portion downstream of the throat omitted.

7.7 DESIGN OF MEASURING STRUCTURES

7.7.1 Installation

A flow-measuring structure consists of an approach channel, the measuring structure itself and a downstream channel. The structure should be rigid, watertight and capable of withstanding peak flows without damage. Its axis should be in line with the direction of flow. The surfaces should be

Table I.7.8. Dimensions of all sizes of standard Parshall flumes

Widths			*Axial lengths*			*Wall depth*	*Vertical distance below crest*		*Converging*	*Gauge points*		
Size throat width	*Upstream end*	*Down-stream end*	*Converging section*	*Throat section*	*Diverging section*	*Converging section*	*Dip at throat*	*Lower end of flume*	*Wall length**	*H_a dist upstream of crest***	*H_b*	
W	*D*	*C*	*B*	*F*	*G*	*E*	*N*	*K*	*A*		*X*	*Y*
m	m	m	m	m	m	m	m	m	m	m	m	m
0.025	0.167	0.093	0.357	0.076	0.204	0.153-0.229	0.029	0.019	0.363	0.241	0.008	0.013
0.051	0.213	0.135	0.405	0.114	0.253	0.153-0.253	0.043	0.022	0.415	0.277	0.016	0.025
0.076	0.259	0.178	0.457	0.152	0.30	0.305-0.610	0.057	0.025	0.466	0.311	0.025	0.038
0.152	0.396	0.393	0.610	0.30	0.61	0.61	0.114	0.076	0.719	0.415	0.051	0.076
0.229	0.573	0.381	0.862	0.30	0.46	0.76	0.114	0.076	0.878	0.588	0.051	0.076
0.305	0.844	0.610	1.34	0.61	0.91	0.91	0.228	0.076	1.37	0.914	0.051	0.076
0.457	1.02	0.762	1.42	0.61	0.91	0.91	0.228	0.076	1.45	0.966	0.051	0.076
0.610	1.21	0.914	1.50	0.61	0.91	0.91	0.228	0.076	1.52	1.01	0.051	0.076
0.914	1.57	1.22	1.64	0.61	0.91	0.91	0.228	0.076	1.68	1.12	0.051	0.076
1.22	1.93	1.52	1.79	0.61	0.91	0.91	0.228	0.076	1.83	1.22	0.051	0.076
1.52	2.30	1.83	1.94	0.61	0.91	0.91	0.228	0.076	1.98	1.32	0.051	0.076
1.83	2.67	2.13	2.09	0.61	0.91	0.91	0.228	0.076	2.13	1.42	0.051	0.076
2.13	3.03	2.44	2.24	0.61	0.91	0.91	0.228	0.076	2.29	1.52	0.051	0.076
2.44	3.40	2.74	2.39	0.61	0.91	0.91	0.228	0.076	2.44	1.62	0.051	0.076
3.05	4.75	3.66	4.27	0.91	1.83	1.22	0.34	0.152	2.74	1.83		
3.66	5.61	4.47	4.88	1.22	2.44	1.52	0.34	0.152	3.05	2.03		
4.57	7.62	5.59	7.62	1.83	3.05	1.83	0.46	0.229	3.50	2.34		
6.10	9.14	7.31	7.62	1.83	3.66	2.13	0.68	0.31	4.27	2.84		
7.62	10.67	8.94	7.62	1.83	3.96	2.13	0.68	0.31	5.03	3.35		
9.14	12.31	10.57	7.62	1.83	4.27	2.13	0.68	0.31	5.79	3.86		
12.19	15.48	13.82	8.23	1.83	4.88	2.13	0.68	0.31	7.31	4.88		
15.24	18.53	17.27	8.23	1.83	6.10	2.13	0.68	0.31	8.84	5.89		

* For sizes 0.3 m to 2.4 m, $A = \frac{W}{2} + 1.2$.

** H_a located $\frac{2}{3}$ A distance from crest for all sizes, distance is wall length, not axial.

NOTE: Flume sizes 0.076 m through 2.4 m have approach aprons rising at a 1 : 4 slope and the following entrance roundings: 0.076 m through 0.228 m, radius 0.4 m; 0.30 m through 0.90, radius 0.51 m; 1.2 m through 2.4 m, radius 0.60 m.

Table I.7.9. Discharge characteristics of Parlshall flumes.

Throat width b in metres	*Discharge range in* m³ s⁻¹ x 10⁻³		*Equation* $Q = K\, h_a^u$	*Head range in metres*		*Modular limit*
	minimum	*maximum*	*(metric)*	*minimum*	*maximum*	h_b/h_a
0.025	0.09	5.4	$0.0604\ h_a^{1.55}$	0.015	0.21	0.50
0.051	0.18	13.2	$0.1207\ h_a^{1.55}$	0.015	0.24	0.50
0.076	0.77	32.1	$0.1771\ h_a^{1.55}$	0.03	0.33	0.50
0.152	1.50	111	$0.3812\ h_a^{1.58}$	0.03	0.45	0.60
0.229	2.50	251	$0.5354\ h_a^{1.53}$	0.03	0.61	0.60
0.305	3.32	457	$0.6909\ h_a^{1.522}$	0.03	0.76	0.70
0.457	4.80	695	$1.056\ h_a^{1.538}$	0.03	0.76	0.70
0.610	12.1	937	$1.428\ h_a^{1.550}$	0.046	0.76	0.70
0.914	17.6	1427	$2.184\ h_a^{1.566}$	0.046	0.76	0.70
1.219	35.8	1923	$2.953\ h_a^{1.578}$	0.06	0.76	0.70
1.524	44.1	2424	$3.732\ h_a^{1.587}$	0.06	0.76	0.70
1.829	74.1	2929	$4.519\ h_a^{1.595}$	0.076	0.76	0.70
2.134	85.8	3438	$5.312\ h_a^{1.601}$	0.076	0.76	0.70
2.438	97.2	3949	$6.112\ h_a^{1.607}$	0.076	0.76	0.70
	in m³ s⁻¹					
3.048	0.16	8.28	$7.463\ h_a^{1.60}$	0.09	1.07	0.80
3.658	0.19	14.68	$8.859\ h_a^{1.60}$	0.09	1.37	0.80
4.572	0.23	25.04	$10.96\ h_a^{1.60}$	0.09	1.67	0.80
6.096	0.31	37.97	$14.45\ h_a^{1.60}$	0.09	1.83	0.80
7.620	0.38	47.14	$17.94\ h_a^{1.60}$	0.09	1.83	0.80
9.144	0.46	56.33	$21.44\ h_a^{1.60}$	0.09	1.83	0.80
12.192	0.60	74.70	$28.43\ h_a^{1.60}$	0.09	1.83	0.80
15.240	0.75	93.04	$35.41\ h_a^{1.60}$	0.09	1.83	0.80

smooth and the structure should be built to close tolerances. Parallel vertical side walls should flank the structure and these should extend upstream to at least the head measuring position. The approach channel should extend a distance upstream for at least five times the width of the structure.

7.7.2 **Design curves**

Design curves should be established by a making few direct discharge measurements in order to determine the elevation of the structure and the effect on the upstream and tailwater levels. The design curves also provide an approximation of the modular limit. Flow is modular when it is independent of variations in tailwater level. Figure I.7.11 shows typical design curves for a compound Crump weir measuring in both the modular and non-modular range, curves B and C being the predicted modular discharge curves for the low and upper crests respectively. In the case of the Crump weir this is 75 per cent of the upstream head, curve A. (Herschy et al., 1977.)

7.7.3 **Measurement of head**

The water surface profile upstream of the measuring structure is not horizontal and the choice of location of the head measurement depends on this

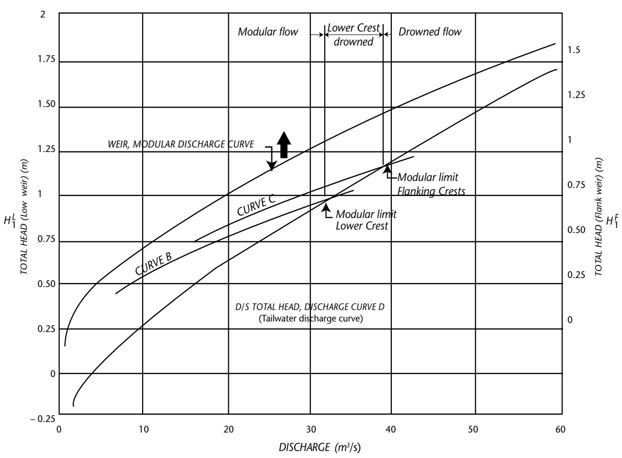


Figure I.7.11. Typical design curves for a measuring structure

profile. The water surface immediately upstream of the structure has the form of a drawdown curve and the pressure is not therefore hydrostatic. Further upstream friction forces produce a water surface slope towards the structure. The location for measuring the upstream head should avoid the drawdown area but should not be so far upstream to be influenced by significant friction losses. Usually this location is specified as 2 to 4 times the maximum head since the above effects will be predominant at high heads rather than at low heads. Table I.7.10 gives the recommendations for the location of the head measuring section for various structures.

Table I.7.10. Measuring structure and location of head measuring section

Measuring structure		*Location of head measuring section*
Thin plate weirs	Triangular notch	3-4h_{max}
	Rectangular notch	3-4h_{max}
Broad crested weirs	Triangular (Crump)	2H_{max} (from crest line)
	Round nosed	2-3H_{max}
	Rectangular	2-3H_{max}
	Flat V	10H' or 2H_{max} from crest line whichever is the greater
Standing-wave flumes	Rectangular	3-4hmax
	Trapezoidal	1-4h_{max}*
	U-shaped	1-4h_{max}*
	Parshall	2/3A**

* Upstream of the leading edge of the entrance transition.
** Upstream from the crest along wall (see Table I.7.8).

7.7.4 Zero setting

Initial zero setting of the reference gauge and recorder from the weir crest (or flume floor) and regular checking thereafter is essential if overall accuracy is to be attained. The reference gauge (outside) should be carefully levelled-in by an instrument giving an accuracy of at least 1 mm. Staff gauges fixed to a wall are difficult to read to ± 3 mm and electronic data loggers (EDLs) are more accurately checked by means of 2 datums, one fixed on top of the wing wall directly above the upstream head measuring section and the second above the stilling well. Unless the head is accurately recorded, serious error in discharge will occur. For example, a 10 mm uncertainty in recording a 150 mm head over a Crump weir will give a 10 per cent uncertainty in discharge. It has been found that for broad-crested weirs an intake pipe of 100 mm diameter is a suitable size giving best results. For flumes, about half that size is acceptable to ensure that the lag between a rise or fall in the river level and the corresponding rise or fall in the gauge well is within the recommended limits. The soffit of the inlet pipe should be set at crest level and the pipe should run straight to the well. It may run obliquely from the side wall but the end should be cut flush with the wall face and a removable cap or plate should be attached to the end of the pipe, through which a number of tapping holes are drilled. The edges of the holes should not be rounded or burred. A typical single crest Crump weir installation is shown in Figures I.7.12 and I.7.13.

7.7.5 Froude number

Measuring structures should not be built in rivers having a high Froude number (F_r), where $F_r = v / \sqrt{gd}$, (d = average depth of flow and v = average velocity). In practice it is sufficient to

Figure I.7.12. River Kennet at Theale. South East England. Single crest crump weir.

Figure I.7.13. River Grwyne at Millbrook: South Wales. Single crest crump weir

determine F_r in the natural channel corresponding to the highest discharge to be gauged accurately, before design of the weir or flume. For the best results the value of F_r should not exceed 0.5. It can be assumed that the value calculated before construction of the weir or flume will not be exceeded after construction.

7.7.6 Compound weirs

It is permissible to incorporate weirs at different crest elevations in a single gauging structure provided the sections with different crest elevations are physically separated by means of intermediate piers so that two-dimensional flow is preserved over each section. The intermediate piers should be of sufficient height to exceed maximum flow levels. The parallel section of the piers should commence at a point located at a distance corresponding to h_{max} from the upstream vertical face of the weir or its toe in the case of triangular weirs, and should extend to the end of the weir at its junction with the downstream invert.

Intermediate piers should have streamlined noses, for example of semicircular or semi-elliptical profile. Flow conditions at or near the pier cutwaters will be improved if the bed level upstream of the low crest section is set below that of the high crest sections, to yield similar values of h/P at the highest discharge to be measured. To compute discharge, total head level is assumed constant over the full width of the weir and is obtained by adding onto the observed static head, the velocity head appropriate to the individual crest section at which the water level is observed.

Compound Crump weirs enable a wide range of flows to be measured and have operated successfully in the United Kingdom national network. They have, however, certain disadvantages: the need for divide (intermediate) walls, the loss of accuracy when the head over the upper crest is small and the fact that the length of straight channel upstream is no longer equal to five times the width of structure. Measurement of a thin sheet of water (low head) on a broad-crested weir is subject to several problems, including uncertainty of the discharge coefficient, adhesion of nappe to the weir face, and dispersion by wind. A large percentage error in the results may occur even with small errors in measuring the head in such cases. The Flat-V weir (see Figure I.7.14) was also introduced to measure a wide range of flows and by careful selection of crest breadth and cross slope provides sensitivity at low flows without the necessity of divide piers. Nevertheless in many circumstances the compound Crump weir is still preferred.

7.7.7 Computation of discharge

Where the discharge formula is quoted in terms of total head (H) the discharge should be computed using a successive approximation technique. Alternatively, the coefficient of velocity (C_v) method may be used (see Herschy et al., 1977).

7.8 TRANSFERABILITY OF LABORATORY RATINGS

The transfer of a laboratory discharge rating to a structure in the field requires the existence, and maintenance, of similitude between laboratory model and prototype, not only with regard to the structure, but also with regard to the approach channel. For example, scour and/or fill in the approach channel will change the head-discharge relation, as will algae growth on the control structure. Both the structure and the approach channel must be kept free from accumulation of debris, sediment, and vegetal growth. Flow conditions downstream from the structure are significant only to the extent that they control the tailwater elevation, which may influence the operation of structures designed for free-flow conditions.

However there are now some 800 measuring structures in operation in the United Kingdom in the national network. These structures, some of which are compound (see Figure I.7.15) have generally been installed where the velocity area method was unsuitable. Many of them operate in the non-modular range by means of a crest tapping. See Herschy et al., (1977) and ISO 4360 (2005). Crest tapping is an array of holes along the crest of the weir. These tapping holes (usually 5 to 10 holes

Figure I.7.14. River Ellen at Bullgill. North West England: Flat-v low flow control weir.

Figure I.7.15. River Colne at Berrygrove. South East England. Compound crump weir.

of 10 mm diameter) are connected through an arrangement of tubes and a manifold to transmit the static head along the crest to the gauge. Check calibrations have been carried out over the years and it has been found that no significant departure from the laboratory rating has occurred (White 1975). The critical factor in all measuring structures is the accurate measurement of head. Any change in the coefficient of discharge is usually negligible compared to the uncertainty in measuring head.

7.9 ACCURACY

The following discussion on accuracy and uncertainty for weirs and flumes relate directly to equations and tables in the preceding discussion. For additional information on determination of uncertainty see Chapter 10 of this Manual.

The basic error equation for the estimation of the uncertainty in a single determination of discharge for a weir is as follows:

if $Q = Cbh^n$

then $$X_Q = \pm\left(X_c^2 + X_b^2 + n^2 X_h^2\right)^{1/2} \quad (7.17)$$

where C = coefficient of discharge; b = length of crest; h = gauged head; n = exponent of h, usually 3/2 for a weir and 5/2 for a V-notch; X_Q = percentage uncertainty in a single determination of discharge; X_c = percentage random uncertainty in the value of the coefficient of discharge; X_b = percentage random uncertainty in the measurement of the length of crest; X_h = percentage random uncertainty in the measurement of gauged head.

All values of uncertainties are standard deviations at the 95 per cent level. The percentage uncertainty will vary with discharge and should be computed for a range of expected stage values.

7.9.1 Thin-plate weirs

The value of n in equation 7.17 is taken as 1.5 for rectangular thin-plate weirs and 2.5 for V-notches and the uncertainty in the coefficient X_c may be taken as ± 1.0 (ISO 1438, 2005).

7.9.2 Triangular profile (Crump) weirs

The random uncertainty X_c, in percent, is given by the following equation:

$$X_c = \pm(10C_v - 9) \quad (7.18)$$

Values of C_v for the purpose of this equation are given in Table I.7.4. However, for normal field installations X_c can generally be taken as ± 2 per cent.

7.9.3 Round-nosed horizontal crest weirs

The random uncertainty, X_c, in percent, in the value of the coefficient is:

$$X_c = \pm 2(21 - 20C_D) \quad (7.19)$$

where C_D is computed by equation 7.11.

7.9.4 Rectangular profile weirs

The random uncertainty X_c, in percent, in the value of the coefficient is:

$$X_c = \pm(10F - 8) \quad (7.20)$$

where F is the coefficient correction factor. Values of F for given values of h/L and $h/(h + P)$ are obtained from Table I.7.5.

7.9.5 Standing wave flumes

For a flume, the error equation may be expressed as follows:

$$X_C = (X_c^2 + A^2 X_b^2 + B^2 X_h^2 + C^2 X_s^2)^{1/2} \quad (7.21)$$

where X_Q = random uncertainty in a single determination of discharge; X_c = random uncertainty in the value of the coefficient; X_b = random uncertainty in the width of throat; X_h = random uncertainty in the measurement of gauged head; X_s = random uncertainty in the slope at the throat; *A, B, C* = numerical coefficients depending on the flume geometry, for example for a rectangular flume *A* = 1, *B* = 1.5 and *C* = 0 and for a U-shaped flume *C* = 0.

Values of *A, B* and *C* can be obtained from Table I.7.11 for trapezoidal and U-shaped flumes.

Table I.7.11. Values of numerical coefficients for trapezoidal and U-shaped flumes

Trapezoidal flumes				*U-shaped flumes*		
mh/b	*A*	*B*	*C*	*h/D*	*A*	*B*
0.01	0.99	1.51	0.01	–	–	–
0.03	0.97	1.53	0.03	–	–	–
0.10	0.94	1.57	0.07	0.10	0.53	1.97
0.20	0.88	1.62	0.12	0.20	0.55	1.94
0.50	0.74	1.77	0.27	0.50	0.65	1.85
1.00	0.58	1.93	0.43	1.00	0.81	1.69
2.00	0.40	2.09	0.59	2.00	0.91	1.59
5.00	0.21	2.30	0.80	5.00	0.97	1.53
10.00	0.12	2.39	0.89	10.00	0.98	1.51
20.00	0.07	2.44	0.94	20.00	–	–
50.00	0.03	2.48	0.98	50.00	–	–
100.00	0.01	2.49	0.99	100.00	–	–

The equation for the random uncertainty X_c, in percent, in the value of the coefficient for all three flumes is:

$$X_c = \pm\left[1 + 20(C_v - C_D)\right] \quad (7.22)$$

The value of C_v and C_D for all three flumes for the purpose of estimating the uncertainty X_c may be found from Tables I.7.6 and I.7.7, respectively. These tables give approximate values and it should be emphasized if values of C_v and C_D are required for calculating discharge the relevant standard should be referred to, in this case ISO 4359 (1983).

7.9.6 Uncertainty in the measurement of the crest length or flume width, X_b

The length of crest or width of flume should be measured to within ± 0.10 per cent.

7.9.7 Random uncertainty in the head measurement, X_h

The random uncertainty X_h, in percent, may be found from the following equation:

$$X_h = \pm \frac{(E_h + E_z)^{1/2}}{h} \times 100 \quad (7.23)$$

where E_h = uncertainty in the measurement of head by the recorder, in mm; E_Z = uncertainty in the zeroing error in setting the recorder, in mm; *h* = recorded head value, in mm.

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ANNEX

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·050	0·608 0	0·008 03	0·615 3	0·004 06	0·650 8	0·002 15
0·051	0·607 5	0·008 43	0·614 9	0·004 27	0·649 8	0·002 25
0·052	0·606 9	0·008 84	0·614 5	0·004 48	0·648 8	0·002 36
0·053	0·606 4	0·009 26	0·614 1	0·004 69	0·647 8	0·002 47
0·054	0·605 9	0·009 70	0·613 7	0·004 91	0·646 8	0·002 59
0·055	0·605 4	0·010 15	0·613 3	0·005 14	0·645 9	0·002 71
0·056	0·604 9	0·010 61	0·613 0	0·005 37	0·644 9	0·002 83
0·057	0·604 5	0·011 08	0·612 6	0·005 61	0·644 0	0·002 95
0·058	0·604 1	0·011 56	0·612 2	0·005 86	0·643 2	0·003 08
0·059	0·603 6	0·012 06	0·611 8	0·006 11	0·642 4	0·003 21
0·060	0·603 2	0·012 57	0·611 4	0·006 37	0·641 7	0·003 34
0·061	0·602 8	0·013 09	0·611 1	0·006 63	0·641 0	0·003 48
0·062	0·602 3	0·013 62	0·610 8	0·006 91	0·640 3	0·003 62
0·063	0·601 9	0·014 17	0·610 5	0·007 18	0·639 6	0·003 76
0·064	0·601 5	0·014 73	0·610 1	0·007 47	0·639 0	0·003 91
0·065	0·601 2	0·015 30	0·609 8	0·007 76	0·638 3	0·004 06
0·066	0·600 8	0·015 88	0·609 5	0·008 06	0·637 6	0·004 21
0·067	0·600 5	0·016 48	0·609 2	0·008 36	0·637 0	0·004 37
0·068	0·600 1	0·017 10	0·609 0	0·008 67	0·636 4	0·004 53
0·069	0·599 8	0·017 72	0·608 7	0·008 99	0·635 8	0·004 70
0·070	0·599 4	0·018 36	0·608 4	0·009 32	0·635 2	0·004 86
0·071	0·599 0	0·019 01	0·608 1	0·009 65	0·634 6	0·005 03
0·072	0·598 7	0·019 67	0·607 9	0·009 99	0·634 0	0·005 21
0·073	0·598 3	0·020 35	0·607 6	0·010 33	0·633 5	0·005 39
0·074	0·598 0	0·021 05	0·607 3	0·010 69	0·632 9	0·005 57

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·075	0·597 8	0·021 76	0·607 1	0·011 05	0·632 4	0·005 75
0·076	0·597 5	0·022 48	0·606 8	0·011 41	0·631 8	0·005 94
0·077	0·597 3	0·023 22	0·606 6	0·011 79	0·631 3	0·006 13
0·078	0·597 0	0·023 97	0·606 4	0·012 17	0·630 8	0·006 33
0·079	0·596 7	0·024 73	0·606 1	0·012 56	0·630 3	0·006 53
0·080	0·596 4	0·025 51	0·606 0	0·012 96	0·629 8	0·006 73
0·081	0·596 1	0·026 30	0·605 8	0·013 36	0·629 3	0·006 94
0·082	0·595 8	0·027 10	0·605 6	0·013 77	0·628 9	0·007 15
0·083	0·595 5	0·027 92	0·605 4	0·014 19	0·628 5	0·007 37
0·084	0·595 3	0·028 76	0·605 2	0·014 62	0·628 0	0·007 59
0·085	0·595 0	0·029 61	0·605 0	0·015 05	0·627 6	0·007 81
0·086	0·594 8	0·030 48	0·604 8	0·015 49	0·627 2	0·008 03
0·087	0·594 5	0·031 36	0·604 6	0·015 94	0·626 7	0·008 26
0·088	0·594 2	0·032 25	0·604 4	0·016 40	0·626 4	0·008 50
0·089	0·594 0	0·033 16	0·604 2	0·016 86	0·626 0	0·008 74
0·090	0·593 7	0·034 09	0·604 0	0·017 34	0·625 6	0·008 98
0·091	0·593 5	0·035 03	0·603 8	0·017 82	0·625 2	0·009 22
0·092	0·593 3	0·035 98	0·603 6	0·018 30	0·624 8	0·009 47
0·093	0·593 1	0·036 96	0·603 4	0·018 80	0·624 4	0·009 73
0·094	0·592 9	0·037 95	0·603 2	0·019 30	0·624 0	0·009 98
0·095	0·592 7	0·038 95	0·603 0	0·019 81	0·623 6	0·010 25
0·096	0·592 5	0·039 97	0·602 8	0·020 33	0·623 3	0·010 51
0·097	0·592 3	0·041 01	0·602 6	0·020 86	0·622 9	0·010 78
0·098	0·592 1	0·042 06	0·602 4	0·021 39	0·622 6	0·011 06
0·099	0·591 9	0·043 12	0·602 2	0·021 94	0·622 2	0·011 33

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·100	0·591 7	0·044 20	0·602 1	0·022 49	0·621 9	0·011 61
0·101	0·591 4	0·045 30	0·601 9	0·023 05	0·621 5	0·011 90
0·102	0·591 2	0·046 41	0·601 7	0·023 62	0·621 2	0·012 19
0·103	0·591 0	0·047 54	0·601 6	0·024 20	0·620 9	0·012 49
0·104	0·590 8	0·048 69	0·601 4	0·024 78	0·620 5	0·012 78
0·105	0·590 6	0·049 85	0·601 3	0·025 37	0·620 2	0·013 09
0·106	0·590 4	0·051 03	0·601 1	0·025 98	0·619 9	0·013 39
0·107	0·590 2	0·052 22	0·600 9	0·026 59	0·619 6	0·013 71
0·108	0·590 1	0·053 44	0·600 8	0·027 20	0·619 3	0·014 02
0·109	0·589 9	0·054 67	0·600 6	0·027 83	0·619 0	0·014 34
0·110	0·589 8	0·055 92	0·600 5	0·028 47	0·618 7	0·014 66
0·111	0·589 7	0·057 19	0·600 3	0·029 11	0·618 4	0·014 99
0·112	0·589 6	0·058 47	0·600 2	0·029 76	0·618 1	0·015 33
0·113	0·589 4	0·059 77	0·600 0	0·030 42	0·617 9	0·015 66
0·114	0·589 2	0·061 08	0·599 8	0·031 09	0·617 6	0·016 01
0·115	0·589 1	0·062 42	0·599 7	0·031 77	0·617 3	0·016 35
0·116	0·589 0	0·063 77	0·599 5	0·032 46	0·617 1	0·016 70
0·117	0·588 9	0·065 14	0·599 4	0·033 15	0·616 9	0·017 06
0·118	0·588 8	0·066 53	0·599 2	0·033 86	0·616 6	0·017 42
0·119	0·588 6	0·067 93	0·599 1	0·034 57	0·616 4	0·017 78
0·120	0·588 5	0·069 35	0·598 9	0·035 29	0·616 2	0·018 15
0·121	0·588 3	0·070 79	0·598 8	0·036 02	0·616 0	0·018 53
0·122	0·588 2	0·072 24	0·598 7	0·036 77	0·615 8	0·018 91
0·123	0·588 1	0·073 72	0·598 5	0·037 51	0·615 5	0·019 29
0·124	0·588 0	0·075 22	0·598 4	0·038 27	0·615 3	0·019 68

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·125	0·588 0	0·076 73	0·598 2	0·039 04	0·615 1	0·020 07
0·126	0·587 9	0·078 27	0·598 1	0·039 82	0·614 8	0·020 46
0·127	0·587 8	0·079 82	0·598 0	0·040 60	0·614 6	0·020 86
0·128	0·587 7	0·081 39	0·597 9	0·041 40	0·614 4	0·021 27
0·129	0·587 6	0·082 98	0·597 8	0·042 20	0·614 1	0·021 68
0·130	0·587 6	0·084 58	0·597 6	0·043 02	0·613 9	0·022 09
0·131	0·587 5	0·086 21	0·597 5	0·043 84	0·613 7	0·022 51
0·132	0·587 4	0·087 85	0·597 3	0·044 67	0·613 5	0·022 94
0·133	0·587 3	0·089 51	0·597 2	0·045 51	0·613 3	0·023 37
0·134	0·587 2	0·091 19	0·597 1	0·046 36	0·613 1	0·023 80
0·135	0·587 2	0·092 89	0·597 0	0·047 22	0·612 9	0·024 24
0·136	0·587 1	0·094 61	0·596 8	0·048 09	0·612 7	0·024 68
0·137	0·587 0	0·096 34	0·596 7	0·048 97	0·612 5	0·025 13
0·138	0·586 9	0·098 10	0·596 6	0·049 86	0·612 3	0·025 59
0·139	0·586 9	0·099 87	0·596 5	0·050 75	0·612 1	0·026 04
0·140	0·586 8	0·101 67	0·596 4	0·051 66	0·611 9	0·026 51
0·141	0·586 7	0·103 48	0·596 2	0·052 58	0·611 7	0·026 97
0·142	0·586 7	0·105 32	0·596 1	0·053 51	0·611 5	0·027 44
0·143	0·586 6	0·107 17	0·596 0	0·054 44	0·611 3	0·027 92
0·144	0·586 6	0·109 04	0·596 0	0·055 39	0·611 2	0·028 40
0·145	0·586 5	0·110 93	0·595 9	0·056 35	0·611 0	0·028 89
0·146	0·586 4	0·112 84	0·595 8	0·057 32	0·610 8	0·029 38
0·147	0·586 3	0·114 76	0·595 7	0·058 30	0·610 6	0·029 88
0·148	0·586 2	0·116 71	0·595 6	0·059 29	0·610 5	0·030 38
0·149	0·586 2	0·118 67	0·595 6	0·060 29	0·610 3	0·030 89

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·150	0·586 1	0·120 66	0·595 5	0·061 30	0·610 2	0·031 40
0·151	0·586 1	0·122 67	0·595 4	0·062 31	0·610 0	0·031 92
0·152	0·586 0	0·124 71	0·595 2	0·063 34	0·609 9	0·032 45
0·153	0·586 0	0·126 76	0·595 2	0·064 37	0·609 7	0·032 97
0·154	0·585 9	0·128 83	0·595 1	0·065 42	0·609 5	0·033 50
0·155	0·585 9	0·130 93	0·595 0	0·066 48	0·609 3	0·034 04
0·156	0·585 9	0·133 04	0·594 9	0·067 55	0·609 1	0·034 58
0·157	0·585 8	0·135 17	0·594 8	0·068 63	0·609 0	0·035 13
0·158	0·585 8	0·137 32	0·594 8	0·069 71	0·608 8	0·035 68
0·159	0·585 7	0·139 50	0·594 7	0·070 81	0·608 7	0·036 24
0·160	0·585 7	0·141 69	0·594 6	0·071 92	0·608 5	0·036 80
0·161	0·585 7	0·143 91	0·594 5	0·073 04	0·608 3	0·037 37
0·162	0·585 6	0·146 14	0·594 4	0·074 17	0·608 2	0·037 94
0·163	0·585 6	0·148 40	0·594 4	0·075 31	0·608 0	0·038 52
0·164	0·585 5	0·150 67	0·594 3	0·076 46	0·607 9	0·039 11
0·165	0·585 5	0·152 97	0·594 2	0·077 62	0·607 7	0·039 69
0·166	0·585 5	0·155 29	0·594 1	0·078 79	0·607 6	0·040 29
0·167	0·585 4	0·157 63	0·594 1	0·079 98	0·607 4	0·040 89
0·168	0·585 4	0·159 99	0·594 0	0·081 17	0·607 3	0·041 49
0·169	0·585 3	0·162 37	0·593 9	0·082 37	0·607 1	0·042 10
0·170	0·585 3	0·164 77	0·593 8	0·083 58	0·607 0	0·042 72
0·171	0·585 3	0·167 19	0·593 7	0·084 81	0·606 9	0·043 34
0·172	0·585 2	0·169 64	0·593 7	0·086 04	0·606 8	0·043 97
0·173	0·585 2	0·172 10	0·593 6	0·087 28	0·606 7	0·044 60
0·174	0·585 1	0·174 59	0·593 5	0·088 54	0·606 5	0·045 24

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·175	0·585 1	0·177 09	0·593 4	0·089 80	0·606 3	0·045 88
0·176	0·585 1	0·179 63	0·593 3	0·091 08	0·606 2	0·046 53
0·177	0·585 1	0·182 19	0·593 3	0·092 37	0·606 1	0·047 18
0·178	0·585 1	0·184 78	0·593 2	0·093 67	0·606 0	0·047 84
0·179	0·585 1	0·187 38	0·593 1	0·094 97	0·605 9	0·048 51
0·180	0·585 1	0·190 01	0·593 0	0·096 29	0·605 7	0·049 18
0·181	0·585 1	0·192 65	0·592 9	0·097 62	0·605 6	0·049 86
0·182	0·585 0	0·195 31	0·592 9	0·098 96	0·605 5	0·050 54
0·183	0·585 0	0·198 00	0·592 8	0·100 32	0·605 4	0·051 22
0·184	0·585 0	0·200 71	0·592 7	0·101 68	0·605 3	0·051 92
0·185	0·585 0	0·203 45	0·592 6	0·103 05	0·605 1	0·052 61
0·186	0·585 0	0·206 21	0·592 6	0·104 44	0·605 1	0·053 32
0·187	0·585 0	0·208 99	0·592 5	0·105 84	0·605 0	0·054 03
0·188	0·585 0	0·211 80	0·592 5	0·107 26	0·604 9	0·054 75
0·189	0·585 0	0·214 63	0·592 4	0·108 67	0·604 8	0·055 47
0·190	0·585 0	0·217 48	0·592 3	0·110 10	0·604 7	0·056 20
0·191	0·585 0	0·220 34	0·592 3	0·111 55	0·604 5	0·056 93
0·192	0·584 9	0·223 22	0·592 2	0·113 00	0·604 4	0·057 66
0·193	0·584 9	0·226 12	0·592 2	0·114 47	0·604 3	0·058 41
0·194	0·584 9	0·229 06	0·592 1	0·115 95	0·604 2	0·059 16
0·195	0·584 9	0·232 03	0·592 0	0·117 43	0·604 1	0·059 92
0·196	0·584 9	0·235 01	0·592 0	0·118 93	0·604 1	0·060 68
0·197	0·584 9	0·238 02	0·591 9	0·120 44	0·604 0	0·061 45
0·198	0·584 9	0·241 06	0·591 9	0·121 97	0·603 9	0·062 22
0·199	0·584 9	0·244 11	0·591 9	0·123 51	0·603 8	0·063 00

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·200	0·584 9	0·247 19	0·591 8	0·125 06	0·603 8	0·063 79
0·201	0·584 9	0·250 28	0·591 8	0·126 62	0·603 7	0·064 58
0·202	0·584 8	0·253 39	0·591 7	0·128 19	0·603 5	0·065 37
0·203	0·584 8	0·256 52	0·591 7	0·129 77	0·603 4	0·066 17
0·204	0·584 8	0·259 69	0·591 6	0·131 36	0·603 3	0·066 98
0·205	0·584 8	0·262 88	0·591 6	0·132 96	0·603 3	0·067 80
0·206	0·584 8	0·266 10	0·591 5	0·134 57	0·603 2	0·068 62
0·207	0·584 8	0·269 34	0·591 5	0·136 20	0·603 1	0·069 44
0·208	0·584 8	0·272 61	0·591 4	0·137 84	0·603 0	0·070 28
0·209	0·584 8	0·275 90	0·591 3	0·139 49	0·602 9	0·071 11
0·210	0·584 8	0·279 21	0·591 3	0·141 15	0·602 9	0·071 96
0·211	0·584 8	0·282 54	0·591 2	0·142 82	0·602 8	0·072 81
0·212	0·584 8	0·285 88	0·591 2	0·144 50	0·602 7	0·073 66
0·213	0·584 7	0·289 24	0·591 1	0·146 20	0·602 6	0·074 53
0·214	0·584 7	0·292 64	0·591 1	0·147 92	0·602 5	0·075 39
0·215	0·584 7	0·296 07	0·591 0	0·149 64	0·602 5	0·076 27
0·216	0·584 7	0·299 53	0·591 0	0·151 38	0·602 4	0·077 15
0·217	0·584 7	0·303 01	0·591 0	0·153 13	0·602 3	0·078 03
0·218	0·584 7	0·306 51	0·590 9	0·154 89	0·602 2	0·078 93
0·219	0·584 7	0·310 04	0·590 9	0·156 66	0·602 2	0·079 82
0·220	0·584 7	0·313 59	0·590 8	0·158 44	0·602 1	0·080 73
0·221	0·584 7	0·317 17	0·590 8	0·160 24	0·602 0	0·081 64
0·222	0·584 7	0·320 77	0·590 8	0·162 04	0·601 9	0·082 55
0·223	0·584 7	0·324 39	0·590 7	0·163 86	0·601 8	0·083 47
0·224	0·584 7	0·328 03	0·590 7	0·165 70	0·601 8	0·084 41

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·225	0·584 6	0·331 68	0·590 6	0·167 54	0·601 7	0·085 35
0·226	0·584 6	0·335 35	0·590 6	0·169 40	0·601 7	0·086 29
0·227	0·584 6	0·339 07	0·590 6	0·171 27	0·601 6	0·087 24
0·228	0·584 6	0·342 82	0·590 5	0·173 15	0·601 5	0·088 19
0·229	0·584 6	0·346 59	0·590 5	0·175 04	0·601 5	0·089 15
0·230	0·584 6	0·350 39	0·590 4	0·176 95	0·601 4	0·090 11
0·231	0·584 6	0·354 21	0·590 4	0·178 86	0·601 3	0·091 08
0·232	0·584 6	0·358 06	0·590 4	0·180 79	0·601 3	0·092 07
0·233	0·584 6	0·361 93	0·590 3	0·182 74	0·601 2	0·093 06
0·234	0·584 6	0·365 82	0·590 3	0·184 69	0·601 2	0·094 05
0·235	0·584 6	0·369 74	0·590 2	0·186 66	0·601 1	0·095 04
0·236	0·584 6	0·373 69	0·590 2	0·188 64	0·601 0	0·096 05
0·237	0·584 6	0·377 66	0·590 2	0·190 69	0·601 0	0·097 06
0·238	0·584 6	0·381 66	0·590 1	0·192 63	0·600 9	0·098 08
0·239	0·584 6	0·385 68	0·590 1	0·194 65	0·600 8	0·099 10
0·240	0·584 6	0·389 73	0·590 1	0·196 68	0·600 8	0·100 13
0·241	0·584 6	0·393 80	0·550 0	0·198 72	0·600 7	0·101 16
0·242	0·584 6	0·397 90	0·590 0	0·200 79	0·600 6	0·102 20
0·243	0·584 6	0·402 02	0·590 0	0·202 87	0·600 6	0·103 25
0·244	0·584 6	0·406 17	0·590 0	0·204 96	0·600 5	0·104 30
0·245	0·584 6	0·410 34	0·590 0	0·207 05	0·600 4	0·105 36
0·246	0·584 6	0·414 54	0·589 9	0·209 16	0·600 3	0·106 42
0·247	0·584 6	0·418 77	0·589 9	0·211 27	0·600 3	0·107 50
0·248	0·584 6	0·423 02	0·589 8	0·213 40	0·600 2	0·108 58
0·249	0·584 6	0·427 30	0·589 8	0·215 55	0·600 2	0·109 67

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·250	0·584 6	0·431 60	0·589 8	0·217 72	0·600 2	0·110 77
0·251	0·584 6	0·435 93	0·589 8	0·219 90	0·600 1	0·111 87
0·252	0·584 6	0·440 28	0·589 8	0·222 09	0·600 1	0·112 99
0·253	0·584 6	0·444 66	0·589 7	0·224 29	0·600 0	0·114 10
0·254	0·584 6	0·449 07	0·589 7	0·226 49	0·600 0	0·115 23
0·255	0·584 6	0·453 50	0·589 7	0·228 73	0·600 0	0·116 35
0·256	0·584 6	0·457 96	0·589 7	0·230 98	0·599 9	0·117 49
0·257	0·584 6	0·462 45	0·589 7	0·233 23	0·599 9	0·118 63
0·258	0·584 6	0·466 96	0·589 6	0·235 49	0·599 8	0·119 78
0·259	0·584 6	0·471 50	0·589 6	0·237 77	0·599 8	0·120 94
0·260	0·584 6	0·476 06	0·589 6	0·240 05	0·599 7	0·122 10
0·261	0·584 6	0·480 65	0·589 5	0·242 35	0·599 6	0·123 26
0·262	0·584 6	0·485 27	0·589 5	0·244 66	0·599 6	0·124 43
0·263	0·584 6	0·489 91	0·589 4	0·246 99	0·599 5	0·125 61
0·264	0·584 6	0·494 53	0·589 4	0·249 33	0·599 5	0·126 80
0·265	0·584 6	0·499 28	0·589 4	0·251 68	0·599 5	0·127 99
0·266	0·584 6	0·504 00	0·589 3	0·254 04	0·599 4	0·129 20
0·267	0·584 6	0·508 76	0·589 3	0·256 42	0·599 4	0·130 41
0·268	0·584 6	0·513 53	0·589 2	0·258 81	0·599 3	0·131 62
0·269	0·584 6	0·518 34	0·589 2	0·261 21	0·599 3	0·132 84
0·270	0·584 6	0·523 17	0·589 2	0·263 63	0·599 2	0·134 07
0·271	0·584 6	0·528 02	0·589 1	0·266 06	0·599 2	0·135 29
0·272	0·584 6	0·532 91	0·589 1	0·268 51	0·599 1	0·136 53
0·273	0·584 6	0·537 82	0·589 1	0·270 98	0·599 1	0·137 78
0·274	0·584 6	0·542 76	0·589 1	0·273 47	0·599 0	0·139 03

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·275	0·584 6	0·547 72	0·589 1	0·275 96	0·599 0	0·140 30
0·276	0·584 6	0·552 72	0·589 0	0·278 45	0·598 9	0·141 57
0·277	0·584 6	0·557 74	0·589 0	0·280 97	0·598 9	0·142 84
0·278	0·584 6	0·562 83	0·589 0	0·283 51	0·598 9	0·144 13
0·279	0·584 7	0·567 94	0·589 0	0·286 07	0·598 8	0·145 42
0·280	0·584 7	0·573 06	0·589 0	0·288 63	0·598 8	0·146 71
0·281	0·584 7	0·578 19	0·588 9	0·291 19	0·598 7	0·148 02
0·282	0·584 7	0·583 35	0·588 9	0·293 77	0·598 7	0·149 33
0·283	0·584 7	0·588 53	0·588 9	0·296 38	0·598 7	0·150 65
0·284	0·584 7	0·593 75	0·588 9	0·299 01	0·598 6	0·151 97
0·285	0·584 7	0·598 99	0·588 9	0·301 63	0·598 6	0·153 30
0·286	0·584 7	0·604 25	0·588 8	0·304 27	0·598 5	0·154 64
0·287	0·584 7	0·609 55	0·588 8	0·306 91	0·598 5	0·155 98
0·288	0·584 7	0·614 87	0·588 8	0·309 59	0·598 5	0·157 34
0·289	0·584 7	0·620 23	0·588 8	0·312 29	0·598 4	0·158 70
0·290	0·584 7	0·625 60	0·588 8	0·314 99	0·598 4	0·160 06
0·291	0·584 7	0·631 01	0·588 7	0·317 69	0·598 3	0·161 43
0·292	0·584 7	0·636 45	0·588 7	0·320 40	0·598 3	0·162 81
0·293	0·584 7	0·641 95	0·588 7	0·323 15	0·598 3	0·164 20
0·294	0·584 8	0·647 48	0·588 7	0·325 91	0·598 2	0·165 59
0·295	0·584 8	0·653 03	0·588 7	0·328 69	0·598 2	0·166 99
0·296	0·584 8	0·658 58	0·588 6	0·331 46	0·598 1	0·168 40
0·297	0·584 8	0·664 16	0·588 6	0·334 24	0·598 1	0·169 82
0·298	0·584 8	0·669 76	0·588 6	0·337 04	0·598 1	0·171 24
0·299	0·584 8	0·675 39	0·588 5	0·339 85	0·598 0	0·172 67

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·300	0·584 8	0·681 06	0·588 5	0·342 68	0·598 0	0·174 10
0·301	0·584 8	0·686 75	0·588 4	0·345 52	0·597 9	0·175 55
0·302	0·584 8	0·692 46	0·588 4	0·348 37	0·597 9	0·177 00
0·303	0·584 8	0·698 21	0·588 4	0·351 24	0·597 9	0·178 45
0·304	0·584 8	0·703 98	0·588 3	0·354 12	0·597 8	0·179 92
0·305	0·584 8	0·709 80	0·588 3	0·357 02	0·597 8	0·181 39
0·306	0·584 8	0·715 68	0·588 3	0·359 95	0·597 8	0·182 87
0·307	0·584 9	0·721 59	0·588 3	0·362 90	0·597 7	0·184 35
0·308	0·584 9	0·727 50	0·588 3	0·365 85	0·597 7	0·185 85
0·309	0·584 9	0·733 41	0·588 2	0·368 80	0·597 6	0·187 35
0·310	0·584 9	0·739 36	0·588 2	0·371 77	0·597 6	0·188 85
0·311	0·584 9	0·745 34	0·588 2	0·374 77	0·597 6	0·190 37
0·312	0·584 9	0·751 35	0·588 2	0·377 79	0·597 5	0·191 89
0·313	0·584 9	0·757 38	0·588 2	0·380 81	0·597 5	0·193 42
0·314	0·584 9	0·763 44	0·588 1	0·383 84	0·597 4	0·194 95
0·315	0·584 9	0·769 54	0·588 1	0·386 87	0·597 4	0·196 50
0·316	0·584 9	0·775 66	0·588 1	0·389 95	0·597 4	0·198 05
0·317	0·584 9	0·781 81	0·588 1	0·393 04	0·597 3	0·199 60
0·318	0·584 9	0·788 02	0·588 1	0·396 15	0·597 3	0·201 17
0·319	0·585 0	0·794 28	0·588 1	0·399 27	0·597 2	0·202 74
0·320	0·585 0	0·800 57	0·588 1	0·402 41	0·597 2	0·204 32
0·321	0·585 0	0·806 85	0·588 1	0·405 53	0·597 2	0·205 90
0·322	0·585 0	0·813 14	0·588 0	0·408 67	0·597 1	0·207 50
0·323	0·585 0	0·819 47	0·588 0	0·411 84	0·597 1	0·209 10
0·324	0·585 0	0·825 83	0·588 0	0·415 03	0·597 0	0·210 71

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·325	0·585 0	0·832 22	0·588 0	0·418 24	0·597 0	0·212 32
0·326	0·585 0	0·838 63	0·588 0	0·421 47	0·597 0	0·213 95
0·327	0·585 0	0·845 88	0·588 0	0·424 71	0·596 9	0·215 58
0·328	0·585 0	0·851 55	0·588 0	0·427 96	0·596 9	0·217 21
0·329	0·585 0	0·858 06	0·588 0	0·431 23	0·596 8	0·218 86
0·330	0·585 0	0·864 59	0·588 0	0·434 51	0·596 8	0·220 51
0·331	0·585 0	0·871 16	0·588 0	0·437 79	0·596 8	0·222 17
0·332	0·585 0	0·877 75	0·587 9	0·441 07	0·596 7	0·223 84
0·333	0·585 0	0·884 38	0·587 9	0·444 38	0·596 7	0·225 51
0·334	0·585 0	0·881 03	0·587 9	0·447 73	0·596 7	0·227 19
0·335	0·585 0	0·897 72	0·587 9	0·451 08	0·596 6	0·228 88
0·336	0·585 0	0·904 48	0·587 9	0·454 46	0·596 6	0·230 58
0·337	0·585 1	0·911 28	0·587 9	0·457 85	0·596 5	0·232 28
0·338	0·585 1	0·918 11	0·587 9	0·461 25	0·596 5	0·234 00
0·339	0·585 1	0·924 91	0·587 9	0·464 67	0·596 5	0·235 72
0·340	0·585 1	0·931 75	0·587 9	0·468 10	0·596 4	0·237 44
0·341	0·585 1	0·938 62	0·587 9	0·471 53	0·596 4	0·239 18
0·342	0·585 1	0·945 51	0·587 8	0·474 97	0·596 3	0·240 92
0·343	0·585 1	0·952 44	0·587 8	0·478 42	0·596 3	0·242 67
0·344	0·585 1	0·959 40	0·587 8	0·481 91	0·596 3	0·244 42
0·345	0·585 1	0·966 38	0·587 8	0·485 42	0·596 2	0·246 19
0·346	0·585 1	0·973 40	0·587 8	0·488 95	0·596 2	0·247 96
0·347	0·585 1	0·980 45	0·587 8	0·492 49	0·596 1	0·249 74
0·348	0·585 1	0·987 53	0·587 8	0·496 04	0·596 1	0·251 52
0·349	0·585 1	0·994 71	0·587 8	0·499 58	0·596 1	0·253 32

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·350	0·585 2	1·001 92	0·587 7	0·503 13	0·596 0	0·255 12
0·351	0·585 2	1·009 12	0·587 7	0·506 72	0·596 0	0·256 93
0·352	0·585 2	1·016 33	0·587 7	0·510 33	0·595 9	0·258 75
0·353	0·585 2	1·023 56	0·587 7	0·513 97	0·595 9	0·260 57
0·354	0·585 2	1·030 82	0·587 7	0·517 58	0·595 9	0·262 40
0·355	0·585 2	1·038 12	0·587 6	0·521 21	0·595 8	0·264 24
0·356	0·585 2	1·045 45	0·587 6	0·524 87	0·595 8	0·266 09
0·357	0·585 2	1·052 80	0·587 6	0·528 56	0·595 7	0·267 94
0·358	0·585 2	1·060 19	0·587 6	0·532 27	0·595 7	0·269 81
0·359	0·585 2	1·067 67	0·587 6	0·535 96	0·595 7	0·271 68
0·360	0·585 3	1·075 19	0·587 5	0·539 67	0·595 6	0·273 55
0·361	0·585 3	1·082 73	0·587 5	0·543 40	0·595 6	0·275 44
0·362	0·585 3	1·090 24	0·587 5	0·547 17	0·595 5	0·277 33
0·363	0·585 3	1·097 78	0·587 5	0·550 96	0·595 5	0·279 23
0·364	0·585 3	1·105 36	0·587 5	0·554 73	0·595 5	0·281 14
0·365	0·585 3	1·112 97	0·587 4	0·558 51	0·595 4	0·283 06
0·366	0·585 3	1·120 63	0·587 4	0·562 31	0·595 4	0·284 98
0·367	0·585 3	1·128 37	0·587 4	0·566 16	0·595 4	0·286 91
0·368	0·585 4	1·136 15	0·587 4	0·570 03	0·595 3	0·288 85
0·369	0·585 4	1·143 91	0·587 4	0·573 91	0·595 3	0·290 80
0·370	0·585 4	1·151 67	0·587 4	0·577 80	0·595 2	0·292 75
0·371	0·585 4	1·159 47	0·587 4	0·581 71	0·595 2	0·294 72
0·372	0·585 4	1·167 30	0·587 4	0·585 60	0·595 2	0·296 69
0·373	0·585 4	1·175 16	0·587 3	0·589 50	0·595 1	0·298 67
0·374	0·585 4	1·183 10	0·587 3	0·593 45	0·595 1	0·300 65

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

·DISCHARGE COEFFICIENTS FOR TRIANGULAR THIN-PLATE WEIRS (V-NOTCH) FOR HEADS IN METRES—continued

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·375	0·585 5	1·191 11	0·587 3	0·597 42	0·595 0	0·302 64
0·376	0·585 5	1·199 14	0·587 3	0·601 41	0·595 0	0·304 65
0·377	0·585 5	1·207 12	0·587 3	0·605 42	0·595 0	0·306 66
0·378	0·585 5	1·215 15	0·587 3	0·609 44	0·594 9	0·308 67
0·379	0·585 5	1·223 20	0·587 3	0·613 46	0·594 9	0·310 70
0·380	0·585 5	1·231 28	0·587 2	0·617 47	0·594 8	0·312 73
0·381	0·585 5	1·239 40	0·587 2	0·621 50	0·594 8	0·314 77

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

CHAPTER 8

MEASUREMENT OF DISCHARGE BY MISCELLANEOUS METHODS

8.1 INTRODUCTION

This chapter discusses briefly the following miscellaneous methods which, although not generally used for routine gauging, may be considered where applicable:
(a) Velocity-index methods;
(b) Floats;
(c) Volumetric measurement;
(d) Measurement of unstable flow – roll waves or slug flow;
(e) Measurement of discharge using tracers;
(f) Gauging from aircraft;
(g) Measurements of discharge using radar.

8.2 VELOCITY-INDEX METHODS

Computation of discharge for some streams is complicated by the fact that variable backwater does not allow the use of a simple stage-discharge relation. Backwater from downstream tributaries, reservoirs, and tides is the principle reason for this problem. A second variable, index velocity, can sometimes be used together with stage to develop a rating that provides a means to accurately compute discharge.

The velocity-index method is actually a rating procedure rather than a discharge measurement procedure. Various instruments for obtaining continuous measurements of stream velocity in a segment of a stream are described in Chapters 5 and 6. Brief descriptions of each type of instrument are given in the following paragraphs. One of these methods utilizes a relatively new technique of measuring velocity in a two-dimensional plane across part of the stream on a continuous basis using an acoustic Doppler instrument. Another method uses an ultrasonic (acoustic) line velocity across the entire stream. Both are considered velocity-index methods.

A continuously recorded index velocity, at a point or in a transverse line, can usually be related to mean velocity (single parameter index-velocity rating) or to stage and mean velocity (multi-parameter index-velocity rating) in the cross-section. Therefore, by knowing the index velocity or the stage and the index velocity, mean velocity in a standard cross section can be computed. The product of cross-sectional area and mean velocity gives the discharge at any time. The calibration of the velocity-index relation, that is, the relation of recorded index velocity to stage and mean velocity, requires discharge measurements for the determination of mean velocity. The cross-sectional area is determined at a standard, or reference, cross section. The standard cross section can be the same as that for the discharge measurement, or in some cases can be a surveyed cross section independent of the discharge measurement. Details of defining and using velocity-index ratings are described in Chapter 2, Volume II – Computation of Discharge of this Manual.

Six types of instruments have commonly been used to obtain an index velocity in a measurement cross-section. They are:
(a) Standard mechanical current meter (described in Chapter 5);
(b) Deflection meter;
(c) Acoustic Doppler point velocity meter (described in Chapter 5);
(d) Acoustic Doppler line velocity meter (described in Chapter 6);
(e) Ultrasonic (acoustic) path velocity meter (described in Chapter 6);
(f) Electromagnetic point velocity meter (described in Chapter 5);
(g) Echo correlation (described in Chapter 6);
(h) Acoustic Doppler – bed mounted (described in Chapter 6).

8.2.1 Standard mechanical current meter method

The use of an unattended standard mechanical current meter, securely anchored in a fixed position in the stream below the minimum expected stage, is attractive because of the simplicity of the device. The most desirable location for the meter will be in the central core of the flow away from the influence of the banks or any other impediment to flow, where streamlines are parallel and at right angles to the measurement cross-section. For streams of irregular alignment or cross-section it may be necessary to experiment with meter location to determine the most suitable site for the meter.

Various methods may be used for recording revolutions and elapsed time of the current meter.

For instance the Current-Meter Digitizer (CMD), as described in Chapter 5 can be used together with an Electronic Data Logger (EDL) to periodically record revolutions and elapsed time. Other electronic counters can connected to a recording device, even older recorders such as a digital punch recorder, to provide a continuous record of index velocity. All of these methods can be programmed to convert revolutions and elapsed time to velocity.

The utilization of a standard mechanical current meter to obtain an index of mean velocity has certain disadvantages that inhibit its use. The meter is susceptible to damage or impairment by submerged drift, but even where that hazard is negligible there is a strong tendency for the meter to become fouled, after long periods of immersion, by algae and other aquatic growth. Stoppage or impaired operation of the meter invariably results from the attachment of such growth, and constant servicing of the meter is usually a necessity. Suspended sediment in the stream also adversely affects the operation of an unattended current meter.

8.2.2 Deflection meter method

Deflection meters have been used extensively in the past to provide a velocity index in small canals and streams where no simple stage discharge relation can be developed. The deflection meter has a submerged vane that is deflected by the current. The amount of deflection, which is roughly proportional to the velocity of the current impinging on the vane, is transmitted either mechanically or electrically to a recorder. Values of the mean velocity of the stream are determined from discharge measurements, and mean velocity is then related to deflection and stage. Today, because of modern electronic methods, deflection meters are seldom, if ever, used.

8.2.3 Acoustic Doppler point velocity meter

The Acoustic Doppler point velocity meter (ADV), as described in Chapter 5 of this Manual, has the potential for use as a velocity-index meter. The United States Geological Survey (USGS) documented use of an ADV meter that was installed inside the principal orifice and discharge point of a natural spring. The ADV collects velocity data that can be used to enhance the accuracy of spring-flow data and identify previously unrecognized hydrologic patterns (Asquith, W.H. and Gary, M.O., 2005). The ADV is capable of measuring extremely low point velocities and should be considered a good possibility for providing accurate velocity-index data at a point in the stream. Advantages of using the ADV include: (a) no moving parts, (b) it is low maintenance and (c) velocity data can be easily recorded on an electronic data logger (EDL).

8.2.4 Acoustic Doppler velocity meter

The Acoustic Doppler Velocity Meter (ADVM) operates in a manner very similar to an Acoustic Doppler Current Profiler (ADCP) that was described in Chapter 6 of this Manual. The primary difference between the ADCP and the ADVM is that the ADVM is fixed to a stationary mount under the water surface instead of being mounted on a moving boat. ADVMs such as the SonTek Argonaut SW are fixed vertical profilers. Details of the instrumentation, installation and other aspects of several similar models of ADVMs are given in Chapter 6, which is based on a report by Morlock, Nguyan, and Ross (2002). There are many newer configurations and models of ADVMs available and models can be selected to best meet the physical characteristics of a site. For example, side-looking ADVMs with relatively low acoustic frequencies have a large horizontal range and may be appropriate for wide channels; while an up-looking ADVM mounted on the streambed may be appropriate for a channel with vertical bi-directional flow. All models are similar in that they use the same basic theory of operation as described in Chapter 6, and that the index-velocity method can be used to produce streamflow data from instrument-measured velocities.

The acoustic Doppler velocity meter, like the ADV, is a relatively new instrument and has been used in numerous field applications, particularly in the United Kingdom. It operates in a manner very similar to an Acoustic Doppler current profiler (ADCP) that was described in Chapter 6 of this Manual. The primary difference between the ADCP and the ADVM is that the ADVM is fixed to a stationary mount under the water surface instead of being mounted on a moving boat. Secondly, it transmits the acoustic beam horizontally through the water rather than vertically. Details of the instrumentation, installation, and other aspects of the ADVM are given in Chapter 6, which is based on a report by Morlock, Nguyan, and Ross (2002).

The ADVM has an advantage over point velocity-index methods in that it provides velocity-index data for a significant part of the flowing stream which can more easily be related to the stream mean velocity. It is useful in streams affected by variable backwater, sluggish, slow-moving streams, ice-affected streams, tidal streams and streams where flow reversals may occur.

8.2.5 Ultrasonic (acoustic) path velocity meter

The ultrasonic (acoustic) path velocity method described in Chapter 6 can be either a single path or multiple paths. When multiple paths are used, mean velocity can usually be computed directly from the multiple path data. Stage and cross-section area provide the additional data necessary to compute discharge. However, if only a single path velocity is available it is usually necessary to use that path velocity as an index velocity for the purpose of computing mean velocity. Like the ADVM, this has an advantage over a point velocity meter because it provides an index velocity over a significant part of the stream which can be more easily related to mean velocity of the stream.

8.2.6 Electromagnetic point velocity meter

The electromagnetic point velocity meter, described in Chapter 5, can be permanently mounted in a stream to provide an index velocity for use with a velocity-index rating. This is very similar to other point velocity meters such as the mechanical current meter and the Acoustic Doppler point Velocity Meter (ADVM). Primary advantages are that the electromagnetic meter has no moving parts and can easily be connected to an electronic data logger for recording continuous velocity data. A disadvantage is that the electromagnetic meter is affected by accumulation of algae or other aquatic growths that can affect the accuracy of the velocity readings. Therefore, it does require periodic and frequent maintenance. Rating development is similar to that of the other velocity-index methods and will be described in later chapters on rating curve development.

8.3 FLOATS

Floats have very limited use in stream gauging, but they can be used where flood measurements are needed and the measuring structure has been destroyed or it is impossible to use a meter. Both surface floats and rod floats are used. Surface floats may be almost anything that floats such as wooden disks, bottles partly filled, or oranges. Floating debris or ice cakes may serve as natural floats. Rod floats are usually made of wood, weighted on one end so they will float upright in the stream Rod floats are sometimes made in sections so their length can be adjusted to fit the stream depth, however care should be observed so they do not touch the streambed.

Two cross sections are selected along a reach of straight channel for a float measurement. The cross sections should be far enough apart so that the time the float takes to pass from one cross section to the other can be measured accurately. A travel-time of at least 20 seconds is recommended, but a shorter time can be used on small streams with high velocities where it is impossible to select an adequate length of straight channel. The edge of water for both cross sections should be referenced to stakes (or other markers) on each bank. Those points will be used at a later date, when conditions permit, to survey cross sections of the measurement reach, and to obtain the distance between cross sections. The surveyed cross sections will be used to determine the average cross section for the reach.

The procedure for a float measurement is to distribute a number of floats uniformly over the stream width, noting the position of each with respect to the bank. They should be placed far enough upstream from the first cross section so they attain the velocity of the stream before they reach the first cross section. A stopwatch is used to time their travel between the two cross sections. The distance of each float from the bank as it passes the second cross section should also be noted.

The velocity of the float is equal to the distance between the cross sections divided by the time of travel. The mean velocity of flow in the vertical is equal to the float velocity multiplied by a coefficient that is based on the shape of the vertical-velocity profile and relative depth of immersion of the float. A coefficient of about 0.85 is commonly used to convert surface velocity to mean velocity. The coefficient for rod floats varies from 0.85 to 1.00 depending on the shape of the cross section, the length of the rod, and the velocity distribution.

The procedure for computing discharge is similar to that for a conventional current meter measurement. The discharge in each partial section is computed by multiplying the average area of the partial section by the mean velocity in the vertical for that partial section. The total discharge is equal to the sum of the discharges for all the partial sections.

Discharge measurements made with floats under favourable conditions may be accurate to within ± 10 per cent. Wind may adversely affect the accuracy of the computed discharge by its effect on the velocity of the floats, especially if velocity is very slow. If a poor reach is selected and not enough float runs are made the results can be as much as 25 per cent in error.

Float measurement methods are described in detail in ISO 748 (2007).

8.4 VOLUMETRIC MEASUREMENT

The volumetric measurement of discharge is only applicable to small discharges, but it is the most accurate method of measuring such flows. In that method the hydrologist observes the time required to fill a container of known capacity, or the time required to partly fill a calibrated container to a known volume. The only equipment required, other than the calibrated container, is a stop-watch.

The container is calibrated in either of two ways. In the first method, water is added to the container by known increments of volume and the depth of water in the container is noted after the addition of each increment. In the second method, the empty container is placed on weighing scales and its weight is noted. Water is added to the container in increments and after each addition the total weight of container and water is noted, along with the depth of water in the container. The equation used to determine the volume corresponding to a depth that was read is:

$$V = \frac{W_2 - W_1}{w} \tag{8.1}$$

where V = volume of water in container, in cubic metres; W_2 = weight of water and container, in kilograms; W_1 = weight of empty container, in kilograms; w = unit weight of water, 1000 kilograms per cubic metre.

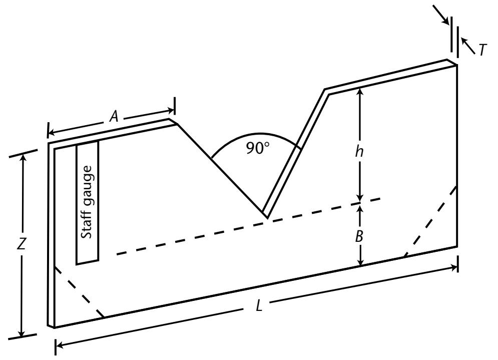
Volumetric measurements are usually made where the flow is concentrated in a narrow stream, or can be so concentrated, so that the entire flow may be diverted into a container. One example of sites presenting the opportunity for volumetric measurement of discharge is a V-notch weir; an artificial control where all the flow is confined to a notch or to a narrow width of catenary shaped weir crest; and a cross-section of natural channel where a temporary earth dam can be built over a pipe of small diameter through which the entire flow is directed. Sometimes it is necessary to place a trough against the artificial control to carry the water from the control to the calibrated container. If a small temporary dam is built, the stage behind the dam should be allowed to stabilize before the measurement is begun. The measurement is made three or four times to minimize errors and to be sure the results are consistent.

Volumetric measurements have also been made under particular circumstances when no other type of measurement was feasible. One such circumstance involved a small stream which was a series of deep pools behind broad-crested weirs that acted as drop structures to dissipate the energy of the stream. At low flows the depth of water on the weir crest was too shallow to be measured by current meter, and the velocity in the pools was too slow for such measurement. To measure the discharge a large container of known volume was placed on a raft held close to the downstream weir face by ropes operated from the banks. A sharp-edged rectangular spout of known width was held so that one end butted tightly against the downstream face of the weir, the base of the spout being held just below the weir crest. The other end of the spout led to the container of known volume. Timed samples of the flow, sufficient to fill the container, were taken at a number of locations along the downstream face of the weir. The raft was being moved laterally across the stream by the ropes. The procedure was analogous to making a conventional current meter discharge measurement. Instead of measuring depth and velocity at a series of observation sites in the cross-section, as is done in a current meter measurement, the discharge per width of spout opening was measured at a series of observation sites. The discharge measured at each site was multiplied by the ratio of subsection width to spout width to obtain the discharge for the subsection. The total discharge of the stream was the summation of the discharges computed for each subsection.

8.5 PORTABLE WEIR MEASUREMENTS

Current-meter measurements made in shallow depths and low velocities are usually inaccurate, if not impossible to obtain. Under these conditions a portable weir plate is a useful device for measuring discharge.

A 90-degree V-notch weir is suitable because of its favourable accuracy at low flows. A weir made of 10- to 16-gauge galvanized sheet iron or aluminium will produce a free-flowing nappe having the effect of a sharp-crested weir and will give satisfactory performance. The thickness of the plate should vary with the size of the weir. Refer to Figure I.8.1 for



Corners may be trimmed if desired

Weir	Z	h	B	C	A	T	Weight(lb)
Large	1.75	1	0.75	4	1	16ga.	24
Medium	1.25	0.80	0.45	3	0.7	14ga.	17
Small	0.75	0.47	0.28	2	0.53	10ga.	8

All dimensions, other than *T*, are in feet

Figure I.8.1. Portable weir-plate sizes

recommended proportions. Decreasing the plate thickness on larger weirs will help maintain portability. The notch is cut, without sharpening, leaving a flat, even edge. Framing, in the form of small angle irons, is required for medium and large sizes. Canvas attached on the downstream or upstream side prevents leakage under or around the weir. Eyebolts, properly placed, will secure rods driven in earth channels to stabilize the plate.

A staff gauge should be attached to the upstream face of the weir plate, with the zero point at the same elevation as the bottom of the weir notch. The staff should be far enough from the notch to be outside of the zone of drawdown which is a distance greater than twice the head on the notch. The staff gauge is used to obtain head on the weir.

The general equation for flow over a sharp-edged triangular weir with a 90-degree notch is:

$$Q = Ch^{5/2} \qquad (8.2)$$

where Q = discharge, in cubic metres per second; h = static head, in metres; and C = the coefficient of discharge.

The weir should be rated by determining the flow volumetrically for various values of head, or by having it rated in a hydraulics lab. In the absence of a laboratory rating, a value of C of 1.36 may be used.

Flows from 0.0005 to 0.05 $m^3 s^{-1}$ are measured with the large weir of Figure I.8.1. Discharges can be measured within 3 per cent accuracy if the weir is not submerged. A weir is not submerged when there is free circulation of air on all sides of the nappe.

To place the plate in a sand or silt channel the only tools required are a carpenter's level and a shovel. The weir is pushed into the streambed and rods are driven through the eye-bolts on each end to stabilize the weir. The level is used to make the top of the plate horizontal and the plate plumb. Another way to level the plate is by fastening a staff gauge or level bubble to each end of the weir. The staff gauges are set at the same elevation. The plate is levelled by making the staff-gauge readings identical or by using the level bubbles. Soil and streambed material is packed around the ends and bottom of the weir to prevent leakage. Canvas is placed immediately downstream from the weir to prevent the falling jet from undercutting the streambed. The flow should be allowed to stabilize before making a measurement. The gauge height should be read at half-minute intervals for a period of about 3-minutes and a mean value should be used in the above equation to compute the discharge. Ordinarily one person can measure with a weir of this type. Upon completion of the measurement remove the weir.

8.6 PORTABLE PARSHALL FLUME MEASUREMENTS

A portable Parshall measuring flume is useful for measuring discharge when the depths are shallow and the velocities are low. The standard Parshall flume has a converging section, a throat and a diverging section. The floor of the converging, or upstream section, is level both longitudinally and transversely when in place. The floor of the throat section slopes downward and the floor of the diverging or downstream section slopes upward. It can be used to measure discharge under free-flow conditions as well as submerged conditions.

The 3-inch (0.0762 metre) portable flume used by the USGS is a modified version of the standard Parshall flume. The modification consists primarily of the removal of the downstream section, which reduces the weight of the flume and makes it easier to install. However, because it has no downstream section it can only be used to measure free-flow conditions, that is, where the submergence ratio is 0.6 or less. This can usually be accomplished by building the streambed up by a few centimetres under the level converging floor of the flume when the flume is installed.

Free flow occurs when the ratio of the lower head to the upper head is less than 0.6. The discharge under this condition depends only on the length of crest (width of throat section) and depth of water at the upper gauge. A flume that is properly constructed has an accuracy of 2 to 3 per cent under free-flow conditions.

The flume is installed by placing it in the channel and by filling in around it to prevent any water from bypassing it. A carpenter's level is used to set the floor of the converging section level. Some flumes are equipped with levels attached to the braces on the flume. After the flume is in place, the streamflow is allowed to stabilize before reading the gauge. After the flow stabilizes gauge readings are taken at about half-minute intervals for about 3 minutes. An average of the gauge readings is used with the flume rating to determine the discharge. The flume should be removed after the measurement is complete.

Field use of a modified Parshall flume is shown in Figure I.8.2. It is virtually the same as the standard Parshall flume except that it does not have a diverging section. The plans are shown in Figure I.8.3. The gauge height, or upstream head on the throat, is read in the small stilling well that is hydraulically connected to the flow by a 1 centimetre hole.

The basic rating equation for a flume is:

$$Q = Cbh^{3/2} \qquad (8.3)$$

where Q = discharge, in cubic feet per second; C = a dimensionless coefficient of discharge which can vary with head and other factors; b = width of throat section, in feet, and h = head, or gauge reading, in the converging section, in feet.

The rating for the modified Parshall flume should be determined by calibration in a hydraulics laboratory. The USGS uses a rating (English units of measurement) defined by Buchanan and Somers (1969) as follows:

$$Q = 1.1392h^{1.5797} \qquad (8.4)$$

The original rating was published in tabular form, and later reduced to the above equation.

8.7 MEASUREMENT OF UNSTABLE FLOW — ROLL WAVES OR SLUG FLOW

8.7.1 Characteristics of unstable flow

Unstable or pulsating flow often occurs during flash floods in arid areas. In pulsating flow the

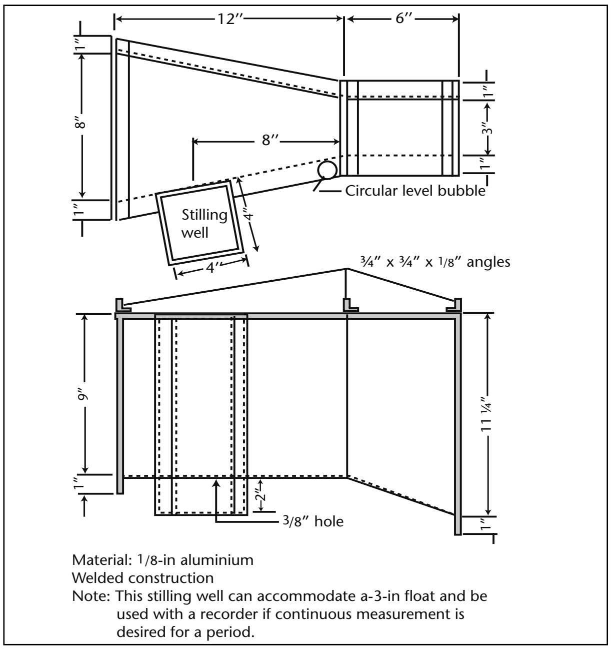


Figure I.8.2. Modified 3-inch Parshall flume in use

Figure I.8.3. Plans for modified 3-inch Parshall flume

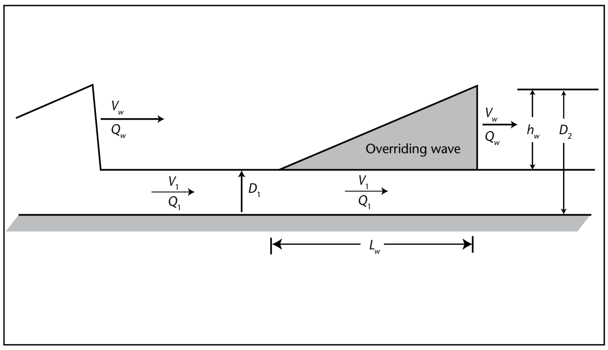


Figure I.8.4. Schematic sketch of longitudinal water surface profile during pulsating flow

longitudinal profile is marked by a series of abrupt translatory waves, as shown in Figure I.8.4, that move rapidly downstream. Translatory waves, commonly called roll waves or slug flow, can only develop in steep channels of super-critical slope and, therefore, are a matter of concern to the designer of steep gradient channels. Channels are usually designed for stable flow. Should pulsating flow occur at high stages in a channel so designed, the channel capacity may be inadequate at a discharge much smaller than the design flow. Furthermore, if the overriding translatory wave carries an appreciable part of the total flow, conventional stream-gauging methods cannot be used to determine the discharge. Conventional water stage recorders of either the float or pressure sensing type do not react quickly enough to record the rapidly fluctuating stage; depths and velocities change too rapidly to permit discharge measurement by current meter; no stage discharge relation exists for pulsating flow and the commonly used formulas for computing stable open channel discharge are not applicable.

8.7.2 Determination of discharge

In brief, the method of determining the discharge of pulsating flow requires:

(a) Computation of the discharge (Q_w) in the overriding waves, and
(b) Computation of the discharge (Q_1) in the shallow depth, or overrun, part of the flow.

The sum of the two discharges is the total discharge at the time of observation.

To compute the discharge (Q_w) in an overriding wave, which is usually wedge-shaped, as shown in Figure I.8.4, the dimensions of the wave are observed and the volume of the wave is divided by the elapsed time between the arrival of waves. For example, if the wedge-shaped wave in a train of waves has a volume of 20 m^3, and a wave arrives every 10 seconds, the discharge in the overriding wave is 20/10, or 2 $m^3 s^{-1}$.

Average values are usually used in the computation. For example, the average volumes of five consecutive waves and the average time interval between the arrivals of those consecutive waves are used. It should be mentioned at this point that the longitudinal profile of the wave is actually slightly concave and the wave front, while extremely steep, is not vertical. However, to simplify the computation of discharge the waves are assumed to have a simple wedge shape.

To compute the discharge (Q_1) in the shallow depth, or overrun, part of the flow, the cross-sectional area of the shallow depth flow is observed (D_1 × channel width), and that area is multiplied by its velocity, V_1. Seldom will there be time enough between waves to obtain velocity observations of V_1 with a conventional current meter. V_1 may be computed by some stable flow equation, such as the Manning equation, but preferably, the surface velocity of shallow depth flow should be measured by optical current meter as described in Chapter 5. The surface velocity can then be multiplied by an appropriate coefficient – 0.9 or 0.85, for example – to give the mean velocity, V_1. The final step is to compute the total discharge at the time of observations by adding Q_w and Q_1.

8.7.3 Examples of discharge determination

This section describes briefly two examples of discharge determinations made under conditions of pulsating flow in southern California, United States of America.

Holmes (1936) obtained photographic documentation of a train of translatory waves in a steep storm-flow channel. The rectangular channel was 13.10 m wide, 2.44 m high, and had a slope of 0.02. The waves themselves lapped at the top of the man-made channel giving them a height (D_2) of 2.44 m and their length (L_w) was about 180 m. The average distance between wave crests was about 360 m and the average time interval between arrivals of the waves (T_p) was 51 seconds. The channel between waves was dry or nearly so ($D_1 = 0$), meaning that the entire discharge was transported in the waves. The discharge computed from the equation:

$$Q = \frac{(volume)}{T_p} = \frac{0.5Wh_wL_w}{T_p} \quad (8.5)$$

was 56.4 $m^3 s^{-1}$. The channel had been designed for stable flow conditions, and according to the Manning equation, had a capacity of about 450 $m^3 s^{-1}$. We see that under the observed conditions of unstable flow the channel could accommodate only one-eighth of the design discharge.

Thompson (1968) described the experimental measurement of pulsating flow in the rectangular storm-flow channel of Santa Anita Wash in Arcadia, California, United States. The concrete channel was 8.5 m wide and had a slope of 0.0251. On the infrequent occasions when the channel had carried storm runoff in the past the flow had been observed to be pulsating. For the test water was released into the channel from an upstream reservoir at controlled rates of approximately 2.8 $m^3 s^{-1}$, 5.4 $m^3 s^{-1}$ and 7.9 $m^3 s^{-1}$. Unstable flow did not develop until the flow came out of a bend in the steep storm channel and entered a straight reach of the channel. In other words the released flow was stable upstream from the bend and pulsating downstream from the bend. During the release of water discharge measurements were made continuously by current meter in the stable flow upstream from the bend. The discharge hydrograph based on those measurements is shown by solid line in Figure I.8.5. Downstream from the bend a simultaneous attempt was made to measure the unstable flow by the method described in this paper, for the purpose of verifying the method.

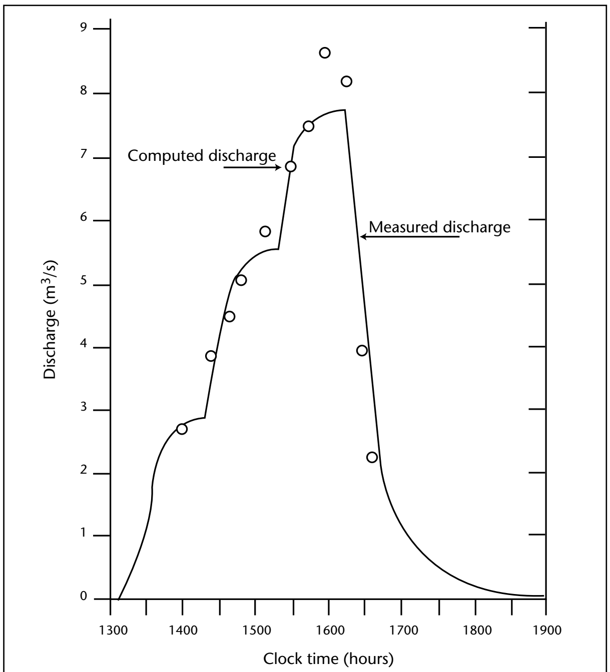


Figure I.8.5. Discharge hydrograph at Santa Anita Wash above Sierra Madre Wash, California, United States, 16 April 1965, and plot of discharge computed from observations of flow

At the test site, about 1 000 m downstream from the bend, the equivalent of a series of staff gauges, in the form of a grid, was painted on one of the vertical channel walls so that water-surface elevations could easily be read by a crew of observers. Some of the observers were equipped with both still- and motion-picture cameras to document the observations; others were equipped with stop-watches to time wave velocity over a measured 31 m course and to obtain the elapsed time between the arrivals of waves. The waves were not evenly spaced and occasionally one wave would overtake another, but in general the waves were fairly uniform in size, maintaining their spacing, and underwent little attenuation in the 1 000 m reach above the test site. For computation purposes the average volumes of five consecutive waves and their average elapsed time between arrivals (T_p) were used, and discharges were computed at 15-minute intervals using the procedure described earlier. For example, at the time of greatest discharge, the average value of T_p was 9.6 seconds and the average wave dimensions were: D_1 = 0.13 m, h_w = 0.20 m, and L_w = 48.2 m. The computed value of Q_w equalled 4.27 $m^3 s^{-1}$. No optical current meter was available at the time, and Q_1 was therefore computed by the Manning equation. The computed value of Q_1 equalled 4.30 $m^3 s^{-1}$, giving a computed total discharge of 8.57 $m^3 s^{-1}$ at the time of observation. The values of discharge that were computed at 15-minute intervals are plotted as open circles in Figure I.8.5 and show satisfactory agreement with the true discharge hydrograph. The field test of the method was therefore considered a success.

8.7.4 Proposed instrumentation

There is as yet no instrumentation that is operational for automatically recording the data required to compute discharge under conditions of pulsating flow. Thompson concluded the above-cited report (1968) by describing three types of automatic instrumentation – photographic, depth sensing and dye dilution – that might be developed for that purpose.

8.8 MEASUREMENT OF DISCHARGE USING TRACER DILUTION METHODS

Tracer dilution methods of measuring discharge have been known since at least 1863 (Spencer and Tudhope, 1958, p. 129). Originally salts (Ostrem,

1964) were generally used as the tracer injected into the stream. Radioactive tracers have been used successfully, but handling problems have limited widespread use (Schuster and Hansen, 1968). The development of stable fluorescent dyes and fluorometers that can measure them at very low concentrations has greatly enhanced the use of dilution methods (Morgan and others, 1977, and Kilpatrick, 1968, 1969). This section will provide only a brief introduction to the basic concepts of tracer dilution methods because a complete and thorough discussion is beyond the scope of this Manual. For more complete details see Kilpatrick and Cobb, 1985, Wilson, Cobb, and Kilpatrick (1984), Duerk (1983), ISO 9555-1 (1994), ISO 9555-2 (1992), ISO 9555-3 (1992), and ISO 9555-4 (1992).

8.8.1 Introduction

Typical examples of situations where the dilution methods might be used are turbulent mountain streams, pipes, canals, sewers, ice-covered streams and sand-channel streams.

Dilution methods are useful under the following flow conditions:

(a) Where it is difficult or impossible to use other methods because of high velocities, turbulence, or debris;
(b) Where, for physical reasons, the flow is inaccessible to other measuring devices;
(c) Where, for some conditions, the rate of change of flow is such that the time to make a current-meter measurement is excessive;
(d) Where the cross-sectional area cannot be accurately measured as part of the discharge measurement or is changing during the measurement.

Note that this section describes only the basic theory and underlying concepts of slug injection and constant-rate injection methods. For descriptions of equipment and measurement procedures the user should refer to the publications described in the first paragraph of this section. Also bear in mind that while many of the same techniques as those used in time-of-travel studies are used in making dye-dilution discharge measurements, a higher level of accuracy is desired, and therefore greater care in performance is required. The user is urged to perform a dye-dilution discharge measurement on an ordinary small stream before attempting more difficult measurements. The stream chosen for a trial should be one where a good current-meter or weir measurement can be made concurrently to provide verification.

8.8.2 Theory

Measurement of the degree of dilution of a known quantity of tracer after its mixing in a flowing stream of water is the basis of dilution gauging There are two main approaches: (a) the slug injection of a known amount of tracer into the flow, which requires that the dilution of the tracer be accounted for by the complete measurement of its mass downstream; for this reason, it is sometimes referred to as the total recovery method; and (b) the constant-rate injection of a tracer solution into the flow, which requires only the measurement of the plateau level of concentration that results downstream after equilibrium has been reached. The principles are simple, yet their successful application in streams, canals, pipes and elsewhere requires a good understanding of the dispersion process. No elaborate theoretical treatment is used in explaining these processes; however, the reader is urged to heed the following principles, because they can eliminate many of the problems others have had in performing such measurements and is helpful in understanding the application of tracers in hydrologic studies in general.

8.8.3 Slug injection

The slug injection of a tracer into a flowing stream is the simplest of all methods, from the standpoint of equipment needs. Where radioactive tracers are employed, handling problems make slug injection virtually the only feasible method. Figure I.8.6 shows the resulting response curves at different distances downstream that may result from a single mid-channel slug injection of tracer. These response curves are time-concentration curves familiar to

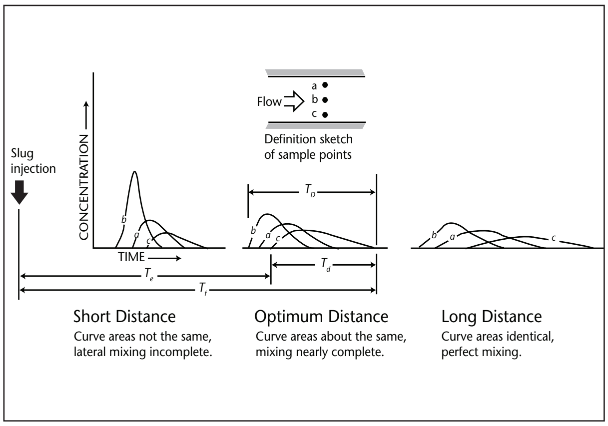


Figure I.8.6. Typical response curves observed laterally and at different distances downstream from a slug injection of a tracer in the centre of a stream

many who have been involved in time-of-travel studies; however, stream reaches used for a dilution-discharge measurement are relatively short and the duration of passage of the entire cloud, T_D rarely exceeds 1 hour. Large distances such as those in time-of-travel studies may result in significant loss of tracer and, hence, inaccurate measurements of discharge. Significant losses of tracer do not ordinarily occur in the short stream reaches used for dilution-discharge measurements, although the possibility must always be kept in mind.

The discharge as measured by the slug-injection tracer-dilution technique is:

$$Q = \frac{M}{A_c} \tag{8.6}$$

where Q is the volume rate of flow of the stream; M is the mass of tracer injected, and A_c is the area under the response curve obtained after adequate mixing of the tracer in the flow.

For clarity units are ignored in the above equation. A constant is required to yield a dimensionally correct equation.

Most important, therefore, is a measurement of the response curve far enough downstream that mixing is almost complete in a cross section. At short distances downstream from a slug injection the tracer is not fully mixed in the total flow of the stream, being more in the centre than along the banks, and this will lead to an error in discharge. Furthermore, the response curve measured in the centre may be much shorter in duration, T_d, than for those along the banks. This is a common occurrence because flow along the banks is usually slower and the channel banks tend to slow and elongate the tracer cloud. At such a short distance an accurate measurement of discharge by dilution cannot be made by ordinary methods. When uniform mixing is reached, the areas under the time-concentration curves are essentially the same regardless of shape while at too short a distance they are not.

From a practical standpoint, complete mixing does not need to be attained. A good dilution-discharge measurement can be made at what is defined here as an optimum distance, L_O, downstream. The optimum distance is usually where mixing is about 95 per cent complete. The distance is optimum because T_D is not too long, and thus sampling of the complete response curves at several points laterally across a section is feasible. Note in Figure I.8.6 that the peaks of the response curves are not the same and that their lengths, or durations, and arrival and departure times are different. Nevertheless, the areas under the individual response curves are nearly the same, which indicates good mixing and allows a good dilution-discharge measurement.

If the response to the tracer slug is measured farther downstream, mixing will be nearly perfect, and the individual time-concentration curves will be nearly identical in area; therefore, a very accurate measurement of discharge may be obtained. That is true, however, only if sampling has been performed for a sufficient time, particularly of the drawn-out tails of the tracer clouds, and if loss of tracer has not occurred because of excessive time of exposure. Frequently failure to measure the low-level concentrations of the trailing edge of the tracer is misinterpreted as a loss of tracer. It is not a real loss but merely a sampling or measurement failure.

The foregoing discussion shows that predicting the length of channel for optimum mixing is not a simple task. Formulas to aid in estimating mixing lengths and approaches to reducing the effective reach lengths can be found in the references.

8.8.4 Constant-rate injection

A continuous, constant-rate injection of tracer may be simulated from the response curve of a slug injection using the superposition principle (Yotsukura and Kilpatrick, 1973). This can best be understood by using any one of the slug-response curves in Figure I.8.6 to simulate the response of a continuous injection at the same location. In Figure I.8.7, the solid response curve is due to the slug injection, M_1; T_e and T_f are the elapsed times to the arrival of the leading edge and trailing edge of the response curve to M_1. Assuming the streamflow

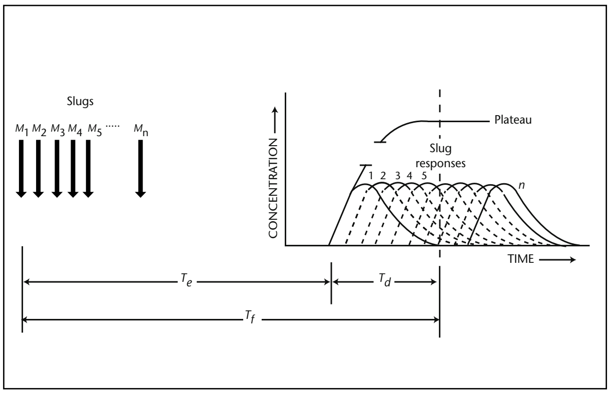


Figure I.8.7. Superposition of slug-response curves to simulate constant-injection build-up to a plateau at one location in a stream section

is steady, continuing to inject a series of tracer slugs of equal amounts, M_2, M_3,....M_n, at uniform time intervals (a constant-rate injection), would yield a series of identical response curves. Of course, if the same soluble tracer were continuously injected, the individual response curves would not be distinguishable and there would be an ever-increasing build-up of concentration with time until T_f was reached. In effect, the super-imposed slug-response curves are being added as they overlap. At time T_f, corresponding to when the trailing edge of the first slug would have passed the point of observation, a plateau of constant concentration is first reached at that point in the channel. At that time, for a constant injection, an equilibrium condition is reached. Continued injection after T_f would result in a plateau of constant concentration at that point, so long as the stream discharge and rate of tracer injection did not vary.

The results illustrated in Figure I.8.7 are shown also in Figure I.8.8 for the same three distances and the three lateral locations used in the example of Figure I.8.6. The same slug-response curves used in Figure I.8.6 are used in Figure I.8.8 to simulate the responses to a continuous injection of tracer as

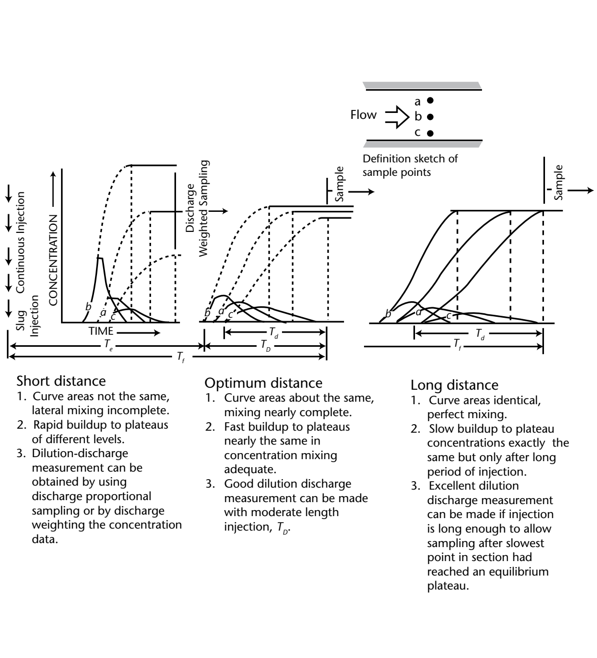


Figure I.8.8. Time-concentration curves for slug and simulated constant injections observed at three points laterally across the channel at three different distances downstream from the injection point

obtained by superimposing the uniformly repeated slug-response curves.

Figure I.8.8 shows that for the short distance, plateaus of different concentrations are obtained laterally. The degree of mixing is poor and identical for the continuous injection and the slug injection.

At the optimum distance the plateaus laterally tend to converge to the same concentration and reflect dilution by the stream discharge, but their concentrations are not identical; nevertheless, mixing is sufficient to produce a good tracer dilution-discharge measurement. Note that tracer would have to be injected continuously for at least a time, T_D, to first reach a completely stabilized plateau for all points across the channel at that distance. The minimum period corresponds to the difference in time for the earliest arriving tracer (centre or point *b*) and the latest departing tracer (side or point *c*). Of course it would be necessary to inject tracer for a time slightly greater than T_D (shown only for the optimum distance) to allow for sampling the fully developed plateau. At a long distance (Figure I.8.8), the resulting plateau concentrations are virtually identical if the constant injection is made over sufficient time and if sufficient time is allowed for build-up across the entire channel before sampling. Depending on the nature of the channel and the mixing distance selected, the injection duration and the lapsed time necessary before sampling the correct plateau concentration may be greater than anticipated.

Unlike the slug-injection method, it is not necessary to measure the entire response curve, only the plateau concentration. Once equilibrium plateau conditions have been reached, the conservation-of-mass principle and the continuity equation:

$$qC = Qc \tag{8.7}$$

apply, whereby the amount of tracer being injected, qC, equals that passing the sampling section, Qc, where q is the rate of tracer injection and is assumed to be very small relative to Q; C is the concentration of the tracer being injected; and c is the resulting plateau concentration after dilution by Q.

Certain conclusions can be drawn from examination of Figures I.8.6, I.8.7 and I.8.8 for a given stream and flow:

(a) Sampling of the response curves from a slug injection must be for a period of time, T_p, and until time T_f; T_f is the earliest time at which the plateau resulting from a constant injection can be sampled. Thus, the effective elapsed time required to make a discharge measurement is essentially the same by either method;

(b) The duration of a continuous injection needed to reach equilibrium plateau concentrations can be determined accurately by examination of the slug-response curves for a given site. The injection time must be at least equal to the time at which tracer is last present in the stream minus the earliest time of arrival of the leading edge of the tracer for that section, T_D. For practical purposes the injection time must be slightly longer than T_D to allow for sampling;

(c) For the constant-injection method, plateaus develop earlier in the main flow than they do close to the streambanks, where the flow is slower.

8.9 REMOTE SENSING AND AIRCRAFT MEASUREMENTS

During periods of large and extensive flooding, conventional discharge measurements may not be feasible or possible in remote areas. Floats, either natural such as trees or ice blocks, or manmade floats dropped from helicopters can be used to obtain estimates of surface velocity at various points in a stream. These data can then be combined with cross section surveys and vertical-velocity profile coefficients to compute stream discharge. Although not as accurate as conventional measurements, such measurements can be very useful in extending coverage of a large flood, or in the extension of stage-discharge rating curves.

Various types of active and passive imagery and sensors mounted on satellites or aircraft can be used to obtain direct quantitative information on several hydraulic variables, including channel area, width, elevation and velocity of the water surface. Computations can be made using these data in combination with topographic information to make estimates of stream discharge. Thus, it is feasible to compute discharge by remote sensing, especially for locations that are difficult to access or otherwise difficult to measure by conventional methods. However, this method is probably useful only for relatively large rivers and, even for these the measurement accuracy will be considerably lower than is possible with ground-based measurements. See Dingman and Bjerklie (2005) for additional information. The following section of this Manual describes radar methods of measuring discharge.

8.10 RADAR METHODS FOR MEASUREMENT OF DISCHARGE

In recent years water-monitoring agencies, universities, and private companies have been investigating the use of radar units to make streamflow measurements. The impetus driving the investigation of radar units has been the search for non-contact methods of making streamflow measurements. Non-contact methods have the potential to make the measurement of streamflow less costly and safer by not having instrumentation in the water or requiring personnel to be in or above the water. To realize this full potential cost-effective and reliable instrumentation is needed.

Radar, an acronym for Radio Detection and Ranging, uses radio waves propagated through the air. When reflected from the water surface, radio waves can be used to determine distance to the water through computation of the round trip travel time from the radar unit to the water surface. There are several commercially available radar units designed to measure river stages using this technology.

Doppler radar can be used to compute the surface velocity of a water column by receiving scattered signals reflected from random short waves that roughen the water surface. The motion of these short waves produces a Doppler shift in the reflected radio waves that is used to compute the velocity of the water surface. For surface velocities to be successfully measured by this technique, some water surface roughness must be present. This roughness can be generated by turbulent boils on the water surface, wind or rain (Costa and others, 2006). Surface velocity, as has been noted in other sections of this Manual, can be used as a surrogate for mean velocity in the vertical-velocity profile by application of a coefficient. The mean velocity multiplied by the channel area yields discharge. Thus a means of computing channel area is needed for radar applications. This could be accomplished by surveying a channel cross-section, measuring stage and applying a stage-area rating as has been discussed for ADVM and other methods.

Another experimental method is the use of Ground-Penetrating Radar (GPR) to directly measure a river cross-section. GPR involves transmission of an electromagnetic pulse with carrier frequency in the MHz range toward the ground from a transmitting antenna at the surface. Some of the radiated electromagnetic energy is reflected back to the receiving antenna from interfaces of materials having different dielectric properties. GPR systems are light, portable or digital, and can provide real-time images of the subsurface (Costa and others, 2006). The USGS has experimented with the deployment of radars for streamflow measurement applications, including deployments from helicopters, cableways, bridges, banks and from hand-held radar guns.

Costa and others (2006) evaluated the use of continuous wave microwave radar, a monostatic UHF Doppler radar, a pulsed Doppler microwave radar and a ground-penetrating radar to measure river flows continuously over long periods and without touching the water with any instruments. The purpose of the study was to directly measure the parameters necessary to compute flow: surface velocity (converted to mean velocity) and cross-sectional area, thereby avoiding the uncertainty, complexity and cost of maintaining rating curves. The results have been very promising, indicating that flow records can be obtained with non-contact radar to acceptable accuracy levels, in many cases to within 5 per cent. Non-contact methods of flow measurement appear to: (a) be as accurate as conventional methods; (b) obtain data when standard contact methods are dangerous or cannot be obtained and (c) provide insight into flow dynamics not available from detailed stage records alone.

The use of radars and other non-contact methods of stream-gauging show great promise, but there is still much to be learned. Conductivity has a significant negative effect on radar energy in water and there are physical limits to the depth of penetration of radar energy in water. When relying on surface velocity data, an assumption of the shape of the velocity profile is needed to convert surface to mean velocity. USGS experiments to date indicate that the assumption of a logarithmic velocity profile produces good agreement with measured velocity profiles but this assumption may not hold in all cases (Hirsch and Costa, 2004).

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CHAPTER 9

INDIRECT DETERMINATION OF PEAK DISCHARGE

9.1 INTRODUCTION

During floods, it is frequently impossible or impractical to measure the peak discharges when they occur, because of the following conditions:
(a) Roads may be impassable;
(b) Structures from which direct discharge measurements might have been made may be non-existent, not suitably located, or destroyed;
(c) Knowledge of the flood rise may not be available sufficiently in advance to permit reaching the site near the time of the peak;
(d) The peak may be so sharp that a satisfactory direct discharge measurement could not be made even if a hydrographer is present at the time;
(e) The flow of debris or ice may be such as to prevent use of sounding equipment;
(f) Limitations of personnel might make it impossible to obtain direct measurements of high-stage discharge at numerous locations during a short flood period.

Consequently, peak discharge must be determined after the flood passes by indirect methods such as slope-area, contracted-opening, flow-over-dam or flow-through-culvert.

Most indirect determinations of discharge make use of the energy equation for computing streamflow. The specific equations differ for different types of flow, such as unobstructed open-channel flow, flow over dams and flow through culverts.

This chapter provides only a brief general discussion of the procedures used in collecting field data and in computing discharge by the various indirect methods. This highly specialized subject is treated in detail in the manuals of the series, "Techniques of Water-Resources Investigations of the United States Geological Survey", that are listed as references at the end of this chapter.

It should be remembered that the discharge that is determined by either direct measurement or by indirect methods includes not only the water, but also any substances suspended or dissolved in the water. The concentration of suspended or dissolved solids in the mixture varies widely, being dependent on climatic and physiographic conditions in the watershed. The volume of dissolved solids, while important in itself, is usually insignificant in comparison with the volume of the water in the discharge, but during floods the quantity of sediment transported by a stream may be highly significant. In many areas of the world, notably in the arid regions, the mixture of sediment and water in a peak flood discharge commonly contains more than 10 per cent of sediment by volume. Sediment concentrations in excess of 30 per cent have been measured. Thought is now being given to qualifying published figures of discharge by indicating the percentage of sediment in the flow, where that percentage is significantly large. A discussion of the measurement of transported sediment is beyond the scope of this Manual, but for a comprehensive discussion of that subject the interested reader is referred to a recent text by the American Society of Civil Engineers (1975), and to ISO 4363 (2002), ISO 4364 (1997) and ISO 3716 (2006).

9.2 COLLECTION OF FIELD DATA

The data required for the computation of discharge by indirect methods are obtained by surveying a reach of channel. The survey includes the elevation and location of high-water marks corresponding to the peak stage; cross-sections of the channel along the reach; selection of roughness coefficients; and description of the geometry of dams, culverts, or bridges, depending on the type of peak-discharge determination to be made. The selection of a suitable site is an important element in the application of each of the indirect methods of discharge determination. See Benson and Dalrymple (1967) for a complete description of data collection for indirect measurements.

It is recommended that a surveying instrument such as a total-station be used to make the survey of the selected site. That type of instrument combines vertical and horizontal control surveys in one operation and is accurate, simple and speedy. All data are automatically recorded electronically for easy plotting and analysis.

Selection of a roughness coefficient remains essentially an art that is developed through experience. The factors that exert the greatest influence on the coefficient of roughness are the character of the stream-bed material, cross-section irregularity, the presence of vegetation and the alignment of the channel. In the Manning equation the roughness coefficient, n, ranges from as low as 0.012 for a concrete-lined channel in good condition or for a smooth sand channel of regular geometry to more than 0.1 for over-bank areas having a heavy cover of brush. However, in channels with severe weed growth, values of n as high as 4.48 have been recorded by Powell (1978). An excellent reference on roughness coefficients, including photographs of channels with verified roughness coefficients, is Barnes (1967).

9.3 SLOPE-AREA METHOD

The slope-area method is one of the most commonly used techniques of indirect discharge determination. See ISO 1070 (1992) and Dalrymple and Benson (1967) for a more complete description of this method than is given herein. In the slope-area method, discharge is computed on the basis of a uniform-flow equation involving channel characteristics, water-surface profiles and a roughness or retardation coefficient. The drop in water-surface profile for a uniform reach of channel represents energy losses caused by bed and bank roughness. In applying the slope-area method, any one of the well-known variations of the Chezy equation may be used. However, the Manning equation is preferred in most countries because it is simple to apply, and the many years of experience in its use have shown that it produces reliable results.

The Manning equation, written in terms of discharge, is:

$$Q = \frac{1}{n} A R^{2/3} S^{1/3} \tag{9.1}$$

where Q = discharge in $m^3\ s^{-1}$; A = cross-sectional area in m^2; S = friction slope, and n = roughness coefficient; R = hydraulic radius in m, = $\frac{A}{P}$ where P is the wetted perimeter.

The Manning equation was developed for conditions of uniform flow in which the water-surface profile and energy gradient are parallel to the stream-bed, and the area, hydraulic radius and depth remain constant throughout the reach. For lack of a better solution, it is assumed that the equation is also valid for the non-uniform reaches that are invariably encountered in natural channels, if the water-surface gradient is modified by the difference in velocity-head between cross-sections. The energy equation for a reach of non-uniform channel between cross-section 1 and cross-section 2 shown in Figure I.9.1 is:

$$(h + h_{v_1}) = (h + h_{v_2}) + (h_{f_{1\text{-}2}}) + k(\Delta h_{v_{1\text{-}2}}) \tag{9.2}$$

where h = elevation of the water surface at the respective cross-sections above a common datum; h_v = velocity head at the respective cross-sections = $\alpha V^2/2g$; h_f = energy loss due to boundary friction in the reach; Δh_v = upstream velocity head minus the downstream velocity head, used as a criterion for expansion or contraction of reach, and $k(\Delta h_v)$ = energy loss due to acceleration or deceleration in a contracting or expanding reach, where k = energy loss coefficient.

Note: The energy head loss due to convergence or expansion of the channel in the measuring reach is assumed to be equal to the difference in the velocity heads at the two sections considered multiplied by a coefficient (1-k). The value of k is taken to be zero for uniform and converging reaches and 0.5 for expanding reaches. The energy loss coefficient of 0.5 for expanding reaches is an approximation, and therefore rapidly expanding reaches should not be selected for slope-area measurements.

The friction slope (S) to be used in the Manning equation is thus defined as:

$$S = \frac{h_f}{L} = \frac{\Delta h + \Delta h_v - k(\Delta h_v)}{L} \tag{9.3}$$

where Δh is the difference in water-surface elevation at the two sections, and L is the length of the reach.

In using the Manning equation the conveyance, K, is computed for each cross-section as $(1/n)AR^{2/3}$. The mean conveyance in the reach is then computed as the geometric mean of the conveyance at the

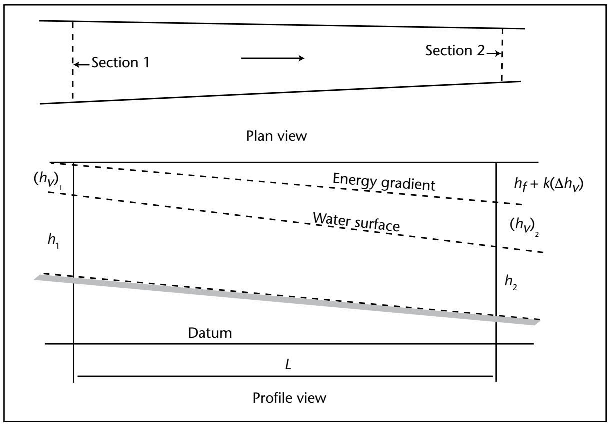


Figure I.9.1. Definition sketch of a slope-area reach

two sections. This procedure is based on the assumption that the conveyance varies uniformly between sections. The discharge is computed by use of the equation:

$$Q = \sqrt{K_1 K_2 S} \qquad (9.4)$$

where S is the friction slope as previously defined.

9.4 CONTRACTED-OPENING METHOD

The contraction of a stream channel by a bridge creates an abrupt drop in water-surface elevation between an approach section and the contracted section under the bridge. The contracted section framed by the bridge abutments and the channel bed is in a sense a discharge meter that can be utilized to compute flood-flows. The head on the contracted section is defined by high-water marks and the geometry of the channel and bridge is defined by field surveys.

In computations of peak discharge at a contraction, the drop in water-surface level between an upstream section and a contracted section is related to the corresponding change in velocity. The discharge equation results from writing the energy and continuity equations for the reach between these two sections, designated as sections 1 and 3 in Figure I.9.2.

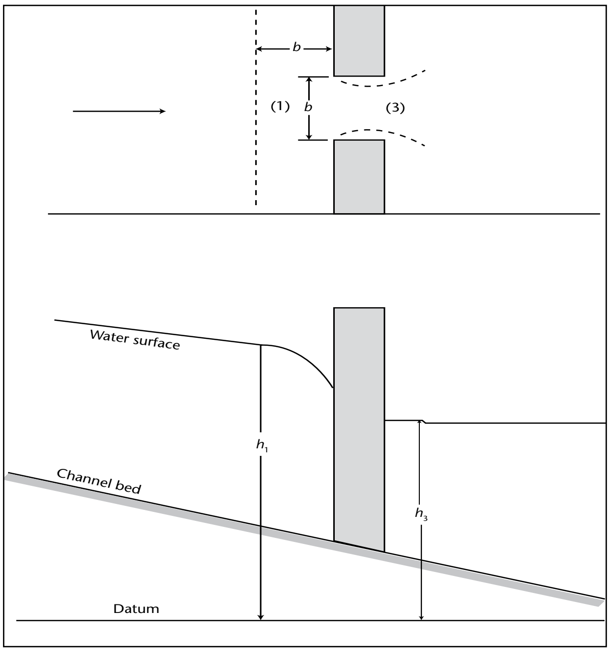


Figure I.9.2. Definition sketch of an open channel contraction

The discharge equation is:

$$Q = CA_3\sqrt{2g\left(\Delta h + \alpha_1 \frac{V_1^2}{2g} - h_f\right)} \qquad (9.5)$$

where Q = discharge; g = acceleration of gravity (9.81 ms^2); C = coefficient of discharge based on the geometry of the bridge and embankment; A_3 = gross area of section 3, this is the minimum section parallel to the constriction between the abutments and it is not necessarily located at the downstream side of the bridge; Δh = difference in elevation of the water surface between sections 1 and 3; $\alpha_1 \frac{V_1^2}{2g}$ = weighted average velocity head at section 1, where V_1 is the average velocity, Q/A_1, and α_1 is a coefficient that takes into account the variation in velocity in that section; h_f = the head loss caused by friction between sections 1 and 3.

The head loss caused by friction, h_f, as computed by the Manning equation, is only an approximation of the actual loss because of the rapid change in velocity from section 1 to section 3. Therefore, satisfactory results are attainable only if h_f is small relative to the difference in head, Δh. For a complete description of the contracted-opening method see Matthai (1967).

9.5 FLOW OVER DAMS AND WEIRS

The term "dams", as used here, also includes highway and railway embankments that act as broad-crested dams during floods. The peak discharge over a dam or weir can be determined on the basis of a field survey of high-water marks and the geometry of the particular structure. The terms dam and weir are used interchangeably.

The basic equation for flow over a dam is:

$$Q = CbH^{3/2} \qquad (9.6)$$

where Q = discharge; C = a coefficient of discharge having the dimensions of the square root of the acceleration of gravity; b = width of the dam normal to the flow, excluding the width of piers, if any, and H = total energy head ($h + V_a^2/2g$) referred to the crest of the dam, where h = static head, and V_a = mean velocity at the approach section to the dam.

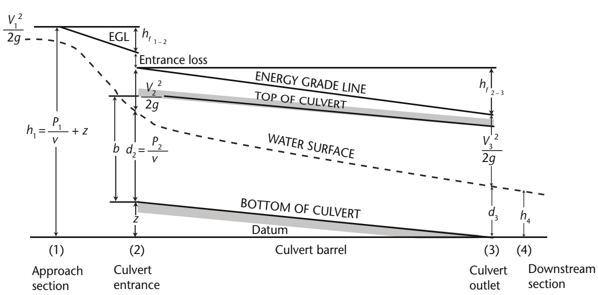
It is apparent from equation 9.6 that the reliability of a computation of flow over a dam is dependent primarily on using the correct dam coefficient, C. Values of C vary with the geometry of the dam and with the degree of submergence of the dam crest by tailwater. Hulsing, (1967) treats in detail the coefficients associated with weirs of unusual shape. Because the technical details in that manual cannot be readily summarized here, the reader is referred to the Hulsing report.

9.6 FLOW THROUGH CULVERTS

The peak discharge through culverts can be determined from high-water marks that define the head-water and tail-water elevations. This indirect method is used extensively to measure flood discharges from small drainage areas.

The placement of a roadway fill and culvert in a stream channel causes an abrupt change in the character of flow. This channel transition results in rapidly varied flow where acceleration, rather than boundary friction, plays the primary role. The flow in the approach channel to the culvert is usually tranquil and fairly uniform. However, within the culvert the flow may be tranquil, critical, or rapid if the culvert is partially filled, or the culvert may flow full under pressure.

The physical features associated with culvert flow are illustrated in Figure I.9.3. They are: the cross-section in the approach channel located upstream from the culvert entrance at a distance that is equivalent to the width of the culvert opening; the culvert entrance; the culvert barrel; the culvert outlet; the farthest downstream section of the barrel; and the tailwater downstream from the culvert barrel.



Note: The loss of energy near the entrance is related to the sudden contraction and subsequent expansion of the live stream within the culvert barrel.

Figure I.9.3. Graphical presentation of the Bernoulli equation in culvert flow

The change in the water-surface profile in the approach channel reflects the effect of acceleration that results from the contraction of cross-sectional area. Loss of energy near the entrance is related to the sudden contraction and subsequent expansion of the live stream within the barrel, and entrance geometry has an important influence on this loss. The important features that control the stage-discharge relation at the approach section can be the occurrence of critical depth in the culvert the elevation of the tailwater, the entrance or barrel geometry or a combination of these elements.

The peak discharge through a culvert is determined by application of the continuity equation and the energy equation between the approach section and a section within the culvert barrel. The location of the downstream section depends on the state of flow in the culvert barrel. For example, if critical flow occurs at the culvert entrance, the head-water elevation is not a function of either the barrel friction loss or the tailwater elevation, and the terminal section is located at the upstream end of the culvert.

Information obtained in the field survey includes the peak elevation of the water surface upstream and downstream from the culvert and the geometry of the culvert and approach channel. Reliable high-water marks can rarely be found in the culvert barrel; therefore, the type of flow that occurred during the peak flow cannot always be determined directly from field data, and classification becomes a trial-and-error procedure.

For convenience in computation, culvert flow has been classified into six types on the basis of the location of the control section and the relative heights of the headwater and tailwater elevations. The six types of flow are illustrated in Figure I.9.4 and pertinent characteristics of each type are given in Table I.9.1. From that information the following general classification of types of flow can be made:

(a) If h_4/D is equal to or less than 1.0 and $(h_1 - z)/D$ is less than 1.5, only types I, II, and III flow are possible;
(b) If h_4/D is greater than 1.0, only type IV flow is possible;
(c) If h_4/D is equal to or less than 1.0 and $(h_1 - z)/D$ is equal to or greater than 1.5, only types V and VI flow are possible.

A manual by Bodhaine (1968) discusses trial-and-error procedures for further identification of the type of culvert flow and for the computation of discharge.

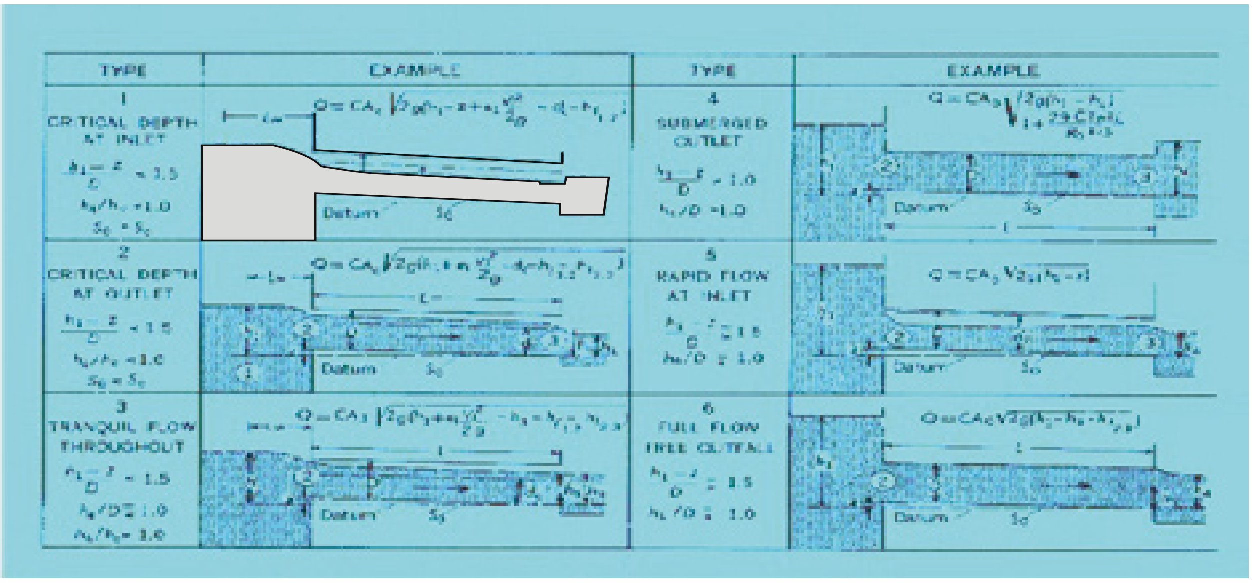


Figure I.9.4. Classification of culvert flow

9.7 ESTIMATING DISCHARGE FROM SUPERELEVATION IN BENDS

Situations exist where none of the four methods previously described for determining peak discharge can be reliably applied. For example, in a highly sinuous, steep canyon stream, there may be no straight reaches of sufficient length for reliable application of the slope-area method; no abrupt area contractions may exist; and no man-made structures, such as dams or culverts, may have been built. In the above situation the peak discharge may often be estimated from the superelevation in the bend of the stream (Apmann, 1973). The discharge should be estimated at each of several bends, after which the several estimated discharges are averaged.

A fundamental characteristic of open channel flow is the deformation of the free surface in a bend because of the action of centrifugal force. The water surface rises on the concave or outside bank of the bend and lowers along the convex or inside bank of the bend. The difference in water-surface elevation between the banks is the superelevation. Superelevation varies with angular distance in the bend because of acceleration of the fluid entering and leaving the curve and because of the varying curvature of streamlines within the bend.

The discharge equation given by Apmann (1973) is:

$$Q = A\sqrt{\frac{gh}{K}} \qquad (9.7)$$

Table I.9.1. Characteristics of types of culvert flow

Flow type	*Barrel flow*	*Location of terminal section*	*Kind of control*	*Culvert slope*	*$h_1 - z/D$*	*h_4/h_c*	*h_4/D*
I	Partly full	Inlet	Critical depth	Steep	< 1.5	< 1.0	₹ 1.0
II	Partly full	Outlet	Critical depth	Mild	< 1.5	< 1.0	₹ 1.0
III	Partly full	Outlet	Backwater	Mild	< 1.5	> 1.0	₹ 1.0
IV	Full	Outlet	Backwater	Any	> 1.0	–	> 1.0
V	Partly full	Inlet	Entrance geometry	Any	⋝ 1.5	–	₹ 1.0
VI	Full	Outlet	Entrance and barrel geometry	Any	⋝ 1.5	–	₹ 1.0

Note: D = maximum vertical height of barrel and diameter of circular culverts.

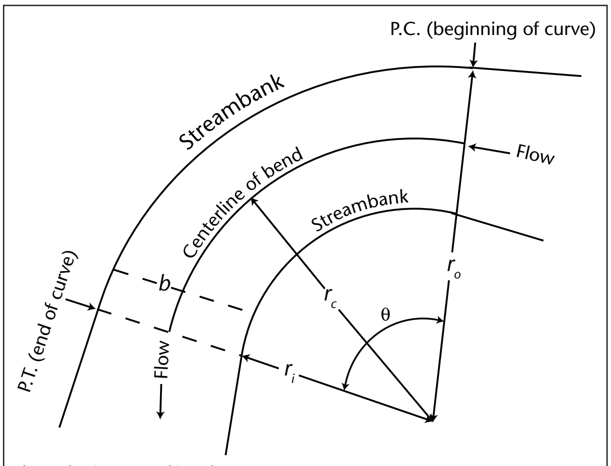
where Q = discharge; A = average radial cross-section area in the bend; g = acceleration of gravity (9.81 ms²), h = superelevation, that is, the maximum difference in water-surface elevation, measured along a radius of the bend, between inner and outer banks of the bend; and K = superelevation coefficient.

The value of K is determined from the equation:

$$K = \frac{5}{4} tgh\left(\left(\frac{r_c \theta}{b}\right)\right) log_e \left(\left(\frac{r_o}{r_i}\right)\right) \tag{9.8}$$

Note: *tgh* is the hyperbolic tangent.

The symbols in equation 9.8 are shown in the sketch in Figure I.9.5.



The angle θ is measured in radians.

Figure I.9.5. Idealized sketch of a bend (plan view)

The method described applies only to bends having no overbank flow. The limited amount of work that has been done with the method indicates that it should not be applied where superelevations are less than 0.076 m because of uncertainties regarding the elevations of high-water marks at the banks. These uncertainties result from wave action and from the thickness of the bank-deposited debris that is often used as a high-water mark.

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CHAPTER 10

UNCERTAINTY OF DISCHARGE MEASUREMENTS

10.1 INTRODUCTION

The accuracy, or uncertainty, of a discharge measurement is very important for purposes of assessing the quality and reliability of that measurement. This type of information provides a means of comparison with other discharge measurements as well as providing a defence against legal challenges. The concept of error and error analysis is a long-standing practice in the field of hydraulics and hydrology; however the concept of uncertainty is relatively new. Methods for evaluating, defining and expressing uncertainty of scientific measurements has developed to the point that these methods are now reasonably uniform in most countries. Several International Organization for Standardization (ISO) standards relating to uncertainty include ISO 1088 *Hydrometry – Velocity-area Methods using current-meters* (2007), ISO 5168 *Measurement of fluid flow – Procedures for the evaluation of uncertainties* (2005), and ISO/IEC Guide 98 *Guide to the expression of Uncertainty in Measurements* (GUM) (1995) and ISO/TS 25377 *Hydrometric uncertainty Guidance-HUG* (2007). It should be noted that ISO 1088 (2007) cancels and replaces ISO/TR 7178, all provisions of which have been incorporated in ISO 1088.

The purpose of this chapter is to provide a detailed description and analysis for the computation of uncertainty for a conventional discharge measurement using procedures accepted by the international community. This is carried out in Section 10.3 of this chapter. In addition, Section 10.4 of this chapter describes the large group of variables that must be considered in the eventual determination of uncertainty of Acoustic Doppler Current Profiler (ADCP) measurements. At the time of this writing, analysis and quantification of uncertainty for ADCP discharge measurements is still under study by groups such as ISO, World Meteorological Organization (WMO), American Institute of Aeronautics and Astronautics (AIAA) and others. Brief descriptions of uncertainty analysis for structure measurements and electromagnetic full channel-width coils are given in their respective chapters. In summary, it is emphasized that all types of discharge measurements should be subject to a rigorous uncertainty analysis based on accepted procedures defined in the references noted above.

10.2 DEFINITION OF UNCERTAINTY AND ACCURACY

Uncertainty and accuracy are terms that are sometimes used interchangeably even though they have two very distinct meanings. Accuracy (or error) refers to the agreement, or disagreement, between the measurement of stream discharge and the true or correct value of the discharge at the time of measurement. Because we can never know the true value of the discharge we can never know the exact amount of error in the discharge measurement.

The uncertainty of a discharge measurement, on the other hand, acknowledges that no measurement is perfect. It is defined, therefore, as a parameter associated with the result of a measurement that characterizes the dispersion of values that could reasonably be attributed to the measurement. It is typically expressed as a range of values in which the measurement value is estimated to lie, within a given statistical confidence. It does not attempt to define or rely on a unique true value.

In summary, common usage of the word accuracy for quantitatively describing the characteristics of a discharge measurement is incompatible with its correct meaning. The proper term for expressing the dispersion and statistical confidence of possible values for a discharge measurement is uncertainty.

10.3 UNCERTAINTY OF CONVENTIONAL CURRENT METER MEASUREMENTS

The procedures described in this section are intended for the determination of uncertainty of discharge measurements made by the measurement of stream depth at several selected verticals in a cross section of the stream, and point velocities measured at a limited number of points in each vertical. It is not intended for computation of uncertainty in other types of discharge measurements such as Acoustic Doppler Current Profiler (ADCP) measurements where velocity profiles and depth are measured continuously during the traverse of the stream. The following procedures are based on those defined in ISO 1088 (2007).

10.3.1 Calculation of uncertainty for current meter discharge measurements

The basic procedure for making conventional current meter discharge measurements is described in Chapter 5 of this Manual and in ISO 748, *Hydrometry – Measurement of liquid flow in open channels using current-meters or floats* (2007). In summary, it consists of the measurement of depth at a number of verticals in a cross section of the stream. The measured depths and the corresponding widths between verticals are used to compute the cross-section area for that segment of the cross section. Stream velocity is measured at a limited number of points in each vertical. Segment discharge is computed as the product of segment area and mean velocity of the corresponding vertical. Total discharge is computed as the summation of the segment discharges.

The discharge computed from a current meter measurement is only an estimate of the true discharge. The difference between the measured discharge and the true discharge is the measurement error, which cannot be determined for reasons explained earlier. This results in measurement uncertainty.

A discharge measurement is the result of the measurement of several component parts as noted above, each of which have inherent errors. The total error of the discharge measurement is a combination of the individual component measurement errors. The determination of measurement uncertainty involves identification and quantification of all contributing component errors. Computation of measurement uncertainty uses concepts and formulas for probability distributions, expected values, standard deviations and correlations of random variables. The standard deviation of the measurement error is taken as the quantitative measure of uncertainty. See ISO 1088 (2007).

It must be noted that the procedures described in ISO 1088 (2007) and in ISO/IEC Guide 98 (GUM) (1995) do not use traditional categorization of errors into random and systematic categories, although technical specifications as the ISO/TS 25377 (HUG) recognise random and systematic effects in measurement procedures. The guide makes it clear that correct mathematical formulation of the uncertainty equations are sufficient to account for all uncertainty sources without reliance on the concepts of systematic and random error. The guide does, however, introduce the concepts of Type A and Type B methods of evaluating component uncertainty. Type A evaluation is by statistical analysis of repeated observations to obtain statistical estimates of the standard deviations of the observations. Type B evaluation is by calculation of an assumed distribution based on scientific judgment and experience.

All uncertainties are expressed as percentages. Standard uncertainty values correspond to percentage coefficients of variation (standard deviation divided by the mean). Expanded uncertainties are identified as such, and correspond to a level of confidence of approximately 95 per cent.

Sources of uncertainties

The sources of uncertainties may be identified by considering a generalized form of the working equation used for gauging by the velocity-area method:

$$Q = \sum_{i=1}^{m} b_i d_i \bar{v}_i \tag{10.1}$$

where Q is the total discharge; b_i, d_i, and $\bar{v}_i$ are the width, depth and mean velocity of the water in the ith of the m verticals or segments into which the cross-section is divided.

The overall uncertainty in the discharge is then composed of:

(a) Uncertainties in widths;
(b) Uncertainties in depths;
(c) Uncertainties in velocities and the use of the velocity-area method, particularly those concerned with the number of verticals and the number of points in each vertical.

Determination of individual components of uncertainty

Uncertainties in width

The measurement of the width between verticals is normally based on distance measurements from a reference point on the bank. If the determination is based on the use of a tag line or measurement of the movement of the wire in the case of a trolley suspension, then the uncertainty in the distance measurement is usually negligible. Where optical means are used to determine the distances, the uncertainty will depend on the distance measured. If the distance is measured by electronic means, a constant uncertainty and an uncertainty depending of the distance measured occurs. The uncertainties result mainly from instrument errors.

Uncertainties in depth

The uncertainty in depth should be determined by the user, based on the particular method which has been adopted, with due regard to variations in water level during the measurement. Uncertainties also arise from interpolation of the depth between verticals at which depths are measured.

Uncertainties in determination of the mean velocity

It is not possible to predict accurately the uncertainties which may arise but there are three main sources: the first arising from the limited time of exposure of the current meter, the second arising from the use of a limited number of points in a vertical and the third arising from the use of a limited number of verticals. These error types do not include errors arising from instrument calibration, but which must be included in the determination of the uncertainty of mean velocity:

(a) *Error Type i – Uncertainties in time of exposure or pulsations*. The velocity of any point in the cross-section is continuously and randomly fluctuating with time because of turbulence. The mean velocity at any point as determined from measurement during a certain time interval is an approximation of the true mean velocity at that point. Pulsations in flow are not independent of each other. The velocity at time t_2 is influenced by the velocity at time t_1. This influence decreases as the time interval $t_2 - t_1$ increases. The effect of increasing the measuring time on the uncertainty is described in a later part of this section of this Manual;
(b) *Error Type ii – Number of points in the vertical*. This uncertainty arises from a limited number of sampling points in a vertical. Computation of the mean velocity in a vertical as an average or weighted average of a number of point velocities results in an approximation of the true mean velocity in the vertical;
(c) *Error Type iii – Number of verticals*. This uncertainty results from a limited number of verticals used for the purpose of defining the size and shape of the channel, the variations in the bed profile and the horizontal distribution of the velocity profile. The uncertainty from this cause decreases with an increase in the number of verticals.

Total uncertainty in discharge

The uncertainties in the individual components of discharge are expressed as relative standard uncertainties, in percent, corresponding to percentage coefficients of variation. The relative (percentage) combined standard uncertainty in the measurement of discharge is given by the following equation, as defined in ISO 1088 (2007):

$$u(Q) = \sqrt{u_m^2 + u_s^2 + \frac{\sum_{i=1}^{m}\left((b_i d_i \bar{v}_i)^2 (u_{b,i}^2 + u_{d,i}^2 + u_{\bar{v},i}^2)\right)}{\left(\sum_{i=1}^{m} b_i d_i \bar{v}_i\right)^2}} \quad (10.2)$$

where $u(Q)$ is the relative (percentage) combined standard uncertainty in discharge; $u_{b,i}$, $u_{d,i}$, $u_{\bar{v},i}$ are the relative (percentage) standard uncertainties in the width, depth, and mean velocity measured at vertical i; u_s is the relative uncertainty due to calibration errors in the current meter, width measurement instrument, and depth sounding instrument; u_m is the relative uncertainty due to the limited number of verticals; and m is the number of verticals.

The relative uncertainty due to calibration errors, u_s, can be expressed as:

$$u_s = \sqrt{\left(u_{cm}^2 + u_{bm}^2 + u_{ds}^2\right)} \quad (10.3)$$

where u_{cm}, u_{bm} and u_{ds} are the relative uncertainties due to calibration errors in the current meter, width measurement instrument, and depth sounding instrument, respectively. An estimated practical value of 1 per cent can be taken for the value of u_s.

The mean velocity, $\bar{v}_i$ at the vertical i is the average of point measurements of velocity made at several depths in the vertical. The uncertainty in $\bar{v}_i$ is computed as follows:

$$u(\bar{v}_i) = \sqrt{u_{p,i}^2 + \frac{1}{n_i}\left(u_{c,i}^2 + u_{e,i}^2\right)} \quad (10.4)$$

where $u_{p,i}$ is the uncertainty in mean velocity $\bar{v}_i$ due to the limited number of depths at which velocity measurements are made at vertical i; n_i is the number of depths in the vertical i at which velocity measurements are made; $u_{c,i}$ is the uncertainty in point velocity at a particular depth in vertical i due to variable responsiveness of the current meter; and $u_{e,i}$ is the uncertainty in point velocity at a particular depth in vertical i due to velocity fluctuations (pulsations) in the stream.

Combining equations 10.2 and 10.4 yields:

$$u(Q) = \sqrt{u_m^2 + u_s^2 + \frac{\sum_{i=1}^{m}\left((b_i d_i \bar{v}_i)^2 \left[u_{b,i}^2 + u_{d,i}^2 + u_{p,i}^2 + \frac{1}{n_i}\left(u_{c,i}^2 + u_{e,i}^2\right)\right]\right)}{\left(\sum_{i=1}^{m} b_i d_i \bar{v}_i\right)^2}} \quad (10.5)$$

If the measurement verticals are placed so that the segment discharges ($b_i d_i \bar{v}_i$) are approximately equal and if the component uncertainties are equal from vertical to vertical, then equation 10.5 simplifies to:

$$u(Q) = \sqrt{u_m^2 + u_s^2 + \frac{1}{m}\left(u_b^2 + u_d^2 + u_p^2 + \frac{1}{n}\left(u_c^2 + u_e^2\right)\right)} \quad (10.6)$$

Equation 10.6 can be used for uncertainty computation for a particular measurement if the segment discharges ($b_i d_i \bar{v}_i$) and the component uncertainties are nearly equal from vertical to vertical. More generally, however, equation 10.6 is useful for developing a qualitative understanding of how the various component uncertainties contribute to the total uncertainty of the discharge measurement. Equation 10.5 is needed to properly account for the effects of unequal distribution of flow among the segments. On the basis of these equations, it can be seen that the total standard uncertainty of a discharge measurement can be reduced by increasing the number of verticals, improving the measurement of the individual components, or both.

The user should, if possible, determine independently the values of the various component uncertainties. However, for routine stream gauging, recommended values are provided in paragraphs which follow in this Manual. These values are expressed in terms of standard uncertainties (level of confidence approximately 68%) for conformance with ISO/IEC Guide 98 (1993). The user should note that because the individual components of uncertainty given in the following paragraphs are based on statistical analysis of the spread of replicate measurements, on prior observations, rather than on repeated observations during the course of the measurement of discharge, they should be treated as Type B evaluations of uncertainties.

Recommended values of component uncertainties

It should be noted that the values of component uncertainties given in the following paragraphs are the result of investigations carried out over a period of many years as described in ISO 1088 (2007). It is recommended that, if possible, the user should determine independently the values of component uncertainties which will apply. The collection and processing of data for the purpose of determining uncertainty of the various components of a discharge measurement are described in detail in ISO 1088 (2007) and will not be included here.

Uncertainties in width, u_b

The uncertainty in the measurement of width should not be greater than 0.5 per cent. As an example, the uncertainty introduced for a particular range-finder having a base distance of 80 cm varies approximately as follows:

Range of width *m*	*Absolute uncertainty* *m*	*Relative uncertainty* *%*
0 to 100	0 to 0.15	0.15
101-150	0.15 to 0.25	0.20
151-250	0.3 to 0.6	0.25

Uncertainties in depth, u_d

For depths up to 0.300 m the uncertainty should not exceed 1.5 per cent and for depths over 0.300 m the uncertainty should not exceed 0.5 per cent.

As an example, the error in depth in an alluvial river whose depths varied from 2 m to 7 m and where the velocity varied up to 1.5 m s^{-1} was, for these conditions, on the order of 0.05 m using a suspension cable.

As another example, the uncertainties in depth were taken with a sounding-rod up to a depth of 6 m, and beyond that value by a log line with standard air-line and wet-line corrections. These observations were made within the range of 0.087 to 1.3 m s^{-1}. Absolute uncertainties (in metres) were determined and relative uncertainties were computed based on the mid-range depth, the results being as given in the following table:

Range of depth *m*	*Absolute uncertainty* *m*	*Relative uncertainty* *%*	*Remarks*
0.4 to 6	0.02	0.65	With sounding rod
6 to 14	0.025	0.25	With log-line and air- and wet-line corrections

Uncertainties in determination of the mean velocity for time of exposure, u_e

The standard uncertainty in point velocity measurement taken at different exposure times and points in the vertical are shown in the following table. These values are given as a guide and should be verified by the user. The values are given as standard uncertainties, in percent, at the 68 per cent level of confidence.

Velocity m s^{-1}	Point in vertical: 0.2 D, 0.4 D or 0.6 D				0.8 D or 0.9 D			
	Exposure time, min							
	0.5	1	2	3	0.5	1	2	3
0.050	25	20	15	10	40	30	25	20
0.100	14	11	8	7	17	14	10	8
0.200	8	6	5	4	9	7	5	4
0.300	5	4	3	3	5	4	3	3
0.400	4	3	3	3	4	3	3	3
0.500	4	3	3	2	4	3	3	2
1.000	4	3	3	2	4	3	3	2
Over 1.000	4	3	3	2	4	3	3	2

Uncertainties in determination of the mean velocity for number of points in the vertical, u_p

The following standard uncertainty values, in percent, were determined from many samples of irregular vertical velocity curves and are given as a guide. They should be verified by the user.

Method of measurement	*Uncertainties, in percent (68% confidence level)*
Velocity distribution	0.5
5 points	2.5
2 points (0.2*D* and 0.8*D*)	3.5
1 point (0.6*D*)	7.5
Surface	15

Uncertainties in determination of the mean velocity for current meter rating, u_c

The following standard uncertainty values, in percent, are given as a guide and are based on experiments performed in several rating tanks.

Measured Velocity m s^{-1}	*Uncertainties, in percent (68% confidence level)*: *Individual rating*	*Group or standard rating*
0.03	10	10
0.10	2.5	5.0
0.15	1.25	2.5
0.25	1.0	2.0
0.50	0.5	1.5
Over 0.50	0.5	1.0

Uncertainties in determination of the mean velocity for number of verticals, u_m

The following standard uncertainty values, in percent, are given as a guide and should be verified by the user.

Number of verticals	*Uncertainties, in percent (68 %confidence)*
5	7.5
10	4.5
15	3.0
20	2.5
25	2.0
30	1.5
35	1.0
40	1.0
45	1.0

Example computation of uncertainty

The following example illustrates the computation of uncertainty in a current meter discharge measurement made by using the velocity-area method. The measurement method consists of dividing the channel cross-section into segments of *m* verticals, and measuring the width, depth and mean velocity associated with each vertical, *i*. The mean velocity at each vertical is computed from point velocity measurements made at each of several depths in the vertical. Total discharge, *Q*, is computed using equation 10.1. The combined standard uncertainty, in percent, for the discharge measurement is computed by using equation 10.2.

The mean velocity, $\bar{v}_i$, at vertical *i* is the average of point measurements of velocity made at several depths in the vertical. The uncertainty in $\bar{v}_i$ is computed by using equation 10.4. If the measurement verticals are placed so that the segment discharges are approximately equal and if the component uncertainties are equal from vertical to vertical, then equation 10.6 can be used to compute the relative uncertainty of the discharge measurement in percent.

The following data are used as an example of the computation of uncertainty for a discharge measurement:

Number of verticals, *m*	20
Number of points in each vertical, *n*	2
Average velocity	0.3 m s^{-1}
Time of exposure at each point	3 minutes
Current meter rating	individual

Component uncertainties are obtained from the previously listed values, and as described in ISO 748 (2007) and ISO 1088 (2007). The following values of the component uncertainties are expressed as relative standard uncertainties, in percent, at the 68 per cent level of confidence:

u_m	2.5
u_s	1.0 (see equation 10.3)
u_b	0.5 (max)
u_d	0.5 (max)
u_p	3.5
u_c	0.9
u_e	3.0 (at 0.2 depth)
	3.0 (at 0.8 depth)
	Total $u_e = u_e = \sqrt{3^2 + 3^2} = 4.2$

Equation 10.6 can then be used to compute the relative combined standard uncertainty as follows:

$$u(Q) = \sqrt{u_m^2 + u_s^2 + \frac{1}{m}\left(u_b^2 + u_d^2 + u_p^2 + \frac{1}{n}(u_c^2 + u_e^2)\right)}$$

$$u(Q) = \sqrt{2.5^2 + 1^2 + \frac{1}{20}\left(0.5^2 + 0.5^2 + 3.5^2 + \frac{1}{2}(0.9^2 + 4.2^2)\right)}$$

$$u(Q) = 2.89\% \text{ or } 3\%$$

It should be noted that the above calculation of uncertainty is a type B evaluation of uncertainty because the component uncertainties are based on previous measurements and calibration data.

10.3.2 Alternative uncertainty method for Price current meter

The uncertainty of a conventional discharge measurement as described in the previous section of this Manual is used in most European countries, and other countries of the world. It is based on methods described in ISO 1088 (2007) and ISO 748 (2007) and is considered the primary method for evaluating uncertainty of discharge measurements. However, an alternative method has been developed primarily for use with the Price AA and Price Pygmy current meters which are used almost exclusively by the United States Geological Survey (USGS). Other differences, besides the meters, are that the two methods have slightly different individual error components.

Various investigators have defined the magnitude of errors in discharge measurements, primarily the errors inherent in the measurement of stream velocity. These include studies of Carter and Anderson (1963), Smoot and Carter (1968), Schneider and Smoot (1976) and Hershey (1971 and 1985). An extensive review of procedures for evaluating errors in discharge measurements was published by Dickinson (1967) and Pelletier (1988).

Sauer and Meyer (1992) used the above mentioned sources of data and information to define a method for computing discharge measurement error. This procedure, used primarily in the United States, computes the uncertainty, or standard error, using a root-mean-square error analysis of individual component errors. The uncertainty, or standard error, means that two-thirds (about 68 per cent) of all measurements will be within the percentage computed by equation 10.9. Each component error is defined by simple equations or tables. The component errors include errors in the measurement of width, depth, and velocity, and in computation procedures. This procedure can be used to compute the standard error for most discharge measurements made with the Price vertical axis, cup-type current meter. It does not apply to measurements made with other types of current meters, or other methods of making discharge measurements. Likewise, it does not apply to discharge measurements where wind, ice, boundary effects, flow obstructions, improper equipment, incorrect measuring procedures and carelessness are factors in the measurement.

Basic equation

The standard error, S_q, for an individual discharge measurement can be estimated by determining the individual component errors that are considered significant, and combining them into a root-mean-square error as shown in equation 10.9. It should be noted that the symbolism herein does not necessarily conform to that of the previously described method.

$$S_q = \sqrt{\left[\frac{\left(S_d^2 + S_t^2\right)}{N}\right] + S_i^2 + S_s^2 + S_h^2 + S_v^2 + S_{sb}^2 + S_{sd}^2 + S_{sv}^2} \quad (10.7)$$

where S_d = depth measurement error; S_t = velocity pulsation error; S_i = instrument error of current meter; S_s = velocity distribution error in the vertical; S_h = horizontal angle of flow error; S_v = horizontal distribution error of velocity and depth; S_{sb} = systematic error in width measurement; S_{sd} = systematic error in depth measurement; S_{sv} = systematic error in velocity measurement and N = number of verticals.

Depth measurement errors

Depth measurement errors, S_d, in equation 10.9 have been categorized according to streambed

Table I.10.1. Standard errors attributable to individual depth measurement errors in discharge measurements

Depth measuring conditions	*Average standard error, or method for computing average standard error, in percent, for indicated type of measurement*		
	Rod suspension *D ≤ 1.2 m*	*Cable suspension* *D ≥ 0.9 m*	*Acoustic* *D ≥ 1.5 m*
(A) Stable streambed (even, firm, smooth)	2	2	2
(B) Soft streambed (silt, mud, muck)	$SE = 2\sqrt{1+\left(\frac{5}{6.6D}\right)^2}$	$SE = 2\sqrt{1+\left(\frac{30}{6.6D}\right)^2}$	
(C) Stable streambed (Uneven, gravel, cobbles for rod suspension) (Uneven cobbles, boulders, for cable suspension)	$SE = 2\sqrt{1+\left(\frac{10}{6.6D}\right)^2}$	$SE = 2\sqrt{1+\left(\frac{30}{6.6D}\right)^2}$	
(D) Mobile streambed (shifting sand, dunes)	10	10	10
(E) Stable streambed (high velocity and some vertical angles)	–	5	–
(F) Unstable streambed (high velocity and some vertical angles)	–	15	–

Note: D = depth, in metres; ≤ equal to or less than; ≥ equal to or greater than; – not applicable.

conditions and the depth measurement method as shown in Table I.10.1. These depth measurement errors have been based largely on experience and are highly subjective. They do conform as much as possible to information noted by some investigators. For instance, Dickinson (1967), reports that for a well selected gauging site the standard deviation would be less than 1 per cent of the mean depth. Hershey (1971) states that depth measurements have a tolerance of 1 to 3 per cent, depending on the magnitude of the depth. Both of these investigators are reporting approximate standard deviations for individual depth measurements. The 2 per cent standard error shown for condition (A) in Table I.10.1 is partly based on these reports.

Velocity pulsation errors

Velocity pulsation errors, S_t, in equation 10.9 are based on studies by Carter and Anderson (1963), using data for 23 different rivers, for velocity observation time intervals ranging from 15 to 240 seconds. Depths ranged from 0.7 m to 8.1 m, velocities ranged from 0.13 m s^{-1} to 2.4 m s^{-1}, and observation points were 0.2, 0.4, 0.6 and 0.8 depth. The data from this study have been reduced to the following equations.

For the 0.6 depth method:

$$S_t = 16.6T^{-0.28} \qquad (10.8)$$

For the 0.2 and 0.8 depth method (2-point method):

$$S_t = 16.0T^{-0.36} \qquad (10.9)$$

In both equations, T is the time of exposure in seconds.

Instrument errors

Instrument errors, S_i, in equation 10.9 are defined for the vertical axis Price AA and Price Pygmy current meters. Studies by Smoot and Carter (1968) found that instrument errors for the Price AA were the same for individually rated and standard rated meters. Likewise, they found no significant differences between new and used meters provide the meters were in good condition. Based on their studies, the instrument error, S_t, for velocities greater than about 0.7 m s^{-1} is constant at about 0.3 per cent. For velocities ranging from 0.07 m s^{-1} to 0.7 m s^{-1}, instrument error can be determined from equation 10.10:

$$S_i = \frac{0.2}{V} \qquad (10.10)$$

where V is the mean velocity, in m s^{-1}. Where V is greater than 0.7 m s^{-1}, the standard error S_t, should be used as 0.3 per cent.

Instrument error for the Price Pygmy current meter was defined by Schneider and Smoot (1976). They found that there was a significant difference between standard rated meters and individually rated meters. However, there was no significant difference between new and used meters. Based on data from their study, four equations define the standard error, S_i, for the Price Pygmy meters.

For individually rated meters with velocity, V, in the range of 0.03 to 0.15 m s^{-1} use equation 10.13.

$$S_i = 0.114V^{-1.4} \tag{10.11}$$

For individually rated meters with velocity, V, in the range of 0.15 to 0.9 m s^{-1} use equation 10.14.

$$S_i = 0.515V^{-0.6} \tag{10.12}$$

For standard rated meters with velocity, V, in the range of 0.03 to 0.15 m s^{-1} use equation 10.15.

$$S_i = 0.192V^{-1.3} \tag{10.13}$$

For standard rated meters with velocity, V, in the range of 0.15 to 0.9 m s^{-1} use equation 10.16.

$$S_i = 0.126V^{-0.3} \tag{10.14}$$

Price Pygmy meters generally are not used for very slow velocities (less than about 0.03 m s^{-1}), or for velocities greater than about 0.9 m s^{-1}. Extrapolation of these equations above and below the defined limits may sometimes be required but should be avoided if possible.

Velocity distribution errors in the vertical

Velocity distribution errors in the vertical, S_s, were studied by Carter and Anderson (1963) using data from 1,800 verticals taken at more than 100 stream sites. Based on their data and analyses, two equations were defined that can be used to compute the standard error for vertical velocity distribution error, S_s, for an entire cross section. Equation 10.15 is used for the 1-point method (0.6 depth method), and equation 10.16 is used for the 2-point method (0.2/0.8 depth method).

$$S_s = \sqrt{\frac{120.4}{N} + 5.02} \tag{10.15}$$

$$S_s = \sqrt{\frac{17.75}{N} + 0.74} \tag{10.16}$$

where N in each equation is the number of verticals in the cross section.

Horizontal angle errors

An angle correction coefficient, the cosine of the horizontal angle at which flow approaches the measuring section, is usually applied to the computation of discharge for each vertical. The error for an individual vertical can be large because of the inability to observe the angle accurately, inability to observe the angle at depths below the water surface and fluctuation of the angle. If this correction is applied to only a few verticals, the error is small for the overall discharge measurement, and can be safely neglected. Where horizontal angles are present throughout most of the cross section, the overall error should be considered in the discharge measurement error. It has been estimated that a standard error, S_h, of 1 per cent (roughly equivalent to an angular error of about 5 degrees) should be used for cross sections where horizontal angles are present for most of the cross section.

Horizontal distribution errors

The assumption of linearity and uniformity of depth and velocity between verticals has been studied by a number of investigators, including Carter and Anderson (1963) and Herschy (1971). The ISO 748 (2007) recommends a standard for horizontal distribution errors based of these investigations. These errors are directly related to the number of verticals used for a discharge measurement. The standard error related to horizontal distribution, S_v, in percent, can be estimated from equation 10.17:

$$S_v = 32N^{-0.88} \tag{10.17}$$

Systematic errors

All of the uncertainties mentioned to this point are referred to as random errors, meaning they can be either positive or negative and are randomly distributed throughout the discharge measurement. There are, in addition to the random errors, the possibility of systematic errors in the measurement of depth, width, and velocity. These are errors caused by improperly calibrated equipment, or improper use of the equipment, so that a systematic error (either positive or negative) is introduced. Most investigators feel that systematic errors are small, generally less than 0.5 per cent each for width, depth and velocity. Therefore, the systematic

errors for use in equation 10.7 should be used as follows:
- S_{sb} = 0.5 per cent (for width);
- S_{sd} = 0.5 per cent (for depth); and
- S_{sv} = 0.5 per cent (for velocity).

Example computation using quantitative method 1

The following example illustrates the computation of uncertainty for a wading discharge measurement, using rod suspension, Price AA current meter and very good depth measuring conditions. The streambed is even, firm and smooth.

The measurement variables are as follows:
- Average depth = 0.67 m
- Average velocity = 0.76 m s^{-1}
- Average time of exposure, T = 50 seconds
- Number of verticals, N = 30
- Method = 2-point (0.2/0.8)
- Horizontal angles = none
- Suspension = rod
- Meter = Price AA
- Depth measuring conditions (Table I.10.1) = A (even, firm, and smooth).

The standard error computations for the individual components are as follows:
- Depth error, S_d = 2.0 per cent (Table I.10.1)
- Pulsation error, S_t = 3.9 per cent (equation 10.9)
- Instrument error, S_i = 0.3 per cent (average velocity = 0.76 m s^{-1} > 0.7 m s^{-1})
- Vertical distribution error, S_s = 1.2 per cent (equation 10.16)
- Horizontal angle error, S_h = 0 per cent (No horizontal angles)
- Horizontal distribution error (velocity and depth), S_v = 1.6 per cent (equation 10.17)
- Systematic errors, S_{sb}, S_{sd} and S_{sv} = 0.5 per cent each.

The discharge measurement error, S_q = 2.3 per cent, as computed with equation 10.7. This is the standard error (68 per cent level of uncertainty) for a very good wading measurement using rod suspension.

The complete details of the Sauer and Meyer (1992) method are described in their report, and will not be repeated in this Manual. A computer program is available to compute the standard error for individual discharge measurements, and it is recommended in the United States that this quantitative evaluation be made for each discharge measurement for which it applies. Computations using this method show that the standard error of individual discharge measurements can range from about 2 per cent for ideal conditions, to about 20 per cent for very poor measuring conditions. Standard errors range from about 3 per cent to 6 per cent for measurements having generally normal measuring conditions. The standard errors computed by this method are in close agreement with qualitative evaluations.

10.3.3 Qualitative method of estimating current meter measurement accuracy

It is recommended that every discharge measurement should be evaluated for accuracy using the qualitative method. Historically, this has been the preferred method in the United States, and the hydrographer should make this evaluation immediately after making the measurement. The evaluation should be based on the hydrographer's opinion of the accuracy of the measurement, and not on how well, or how poorly, the measurement plots on the stage-discharge relation. It is difficult to provide written guidelines for making a qualitative evaluation of accuracy. It depends greatly on the experience, skill, and training of the hydrographer. Several of the factors that should be considered by the hydrographer are as follows:

(a) *Measuring section*. Consider factors such as the uniformity of depths, the smoothness of the streambed, the streambed material (that is, smooth sand, small firm gravel, large rocks, soft muck, etc.), the ability to accurately measure the depth, the approach conditions, presence of bridge piers and other things that would affect measurement accuracy;

(b) *Velocity conditions*. Consider smoothness of velocity, uniformity of velocity, very slow velocity, very high velocity, turbulence, obstructions that may affect the vertical velocity distribution, use of 1-point or 2-point method, length of counting (40 or more seconds versus half-counts) and other factors that affect accuracy of velocity measurements;

(c) *Equipment*. Consider the type of current meter used (horizontal axis meter, vertical axis Price AA, vertical axis Price pygmy, acoustic, electromagnetic), the type of depth sounding equipment, and the condition of the equipment. Accurate measurements require that measurement equipment be properly assembled and maintained in good condition. To avoid damage in transport, the equipment should be packed in appropriate containers or compartments of the vehicle used by the hydrologist. Current meters are especially susceptible to damage when in use, as measurements must often be made when drift or floating ice is present in a stream;

(d) *Spacing of observation verticals*. It is generally recommended that about 25 to 30 verticals be used for a discharge measurement, and that they be spaced so that no more than 5 per cent of the total discharge is contained in each sub-section. This is frequently difficult to attain, and except for unusual cases, no more than 10 per cent of the total discharge should be in a sub-section. Otherwise, the accuracy will be affected negatively. A measurement vertical should be located fairly close to each bank and at breaks in depth. The spacing of the verticals should be reduced in the vicinity of bridge piers. If many bridge piers are present in the section or if the stream-bed is non-uniform, more verticals than the recommended 25-30 should be used;

(e) *Rapidly changing stage*. This has been discussed in previous sections of this Manual, but should also be considered when assessing the accuracy of the measurement. When the stage changes rapidly during a discharge measurement, the computed discharge figure loses some of its reliability and there is uncertainty as to the appropriate gauge height to apply to that discharge figure. Consequently, the standard procedure for making discharge measurements should be shortened when the stage is changing rapidly, as explained in a previous section of this report, even at the expense of some accuracy. The reduction in measurement time makes it possible to obtain a mean gauge height that is more representative of the measured discharge;

(f) *Ice measurements*. Making discharge measurements under ice cover is usually more difficult, and sometimes less accurate, than making open-water discharge measurements. Presence of slush ice, layered ice and anchor ice will have adverse affects on accurate measurement of depth and velocity. Velocity distribution will be affected if the water surface is in contact with the ice. Freezing of water in the meter cups and pivot chamber may affect performance of the equipment. If the ice cover is layered so that there is water flowing between ice layers, it is almost impossible to obtain a reliable discharge measurement particularly if the water layers are too thin to permit insertion of the meter between ice layers. The exposure of a wet current meter to subfreezing air temperatures may cause serious under-registration of the current meter as a result of ice forming in the meter bearings and contact chamber. Therefore, once the measurement is started, the current meter should be kept in the water as much as possible to avoid exposure to the cold air;

(g) *Wind*. Wind may affect the accuracy of a discharge measurement by obscuring the angle of the current or by creating waves that make it difficult to sense the water surface prior to making depth soundings. Wind may also affect the vertical velocity distribution, particularly near the surface. When making boat measurements, the wind-caused waves may induce vertical motion in a cable suspended meter, or the wind may cause an oscillatory horizontal movement of the boat against the tag line; either movement may affect the operation of the current meter. Table I.10.2 summarizes the results of an investigation by Kallio (1966) on the effect of vertical motion on the operation of Price, vane type, and Ott (cosine rotor 8646-A) current meters. The plus signs in Table I.10.2 indicate over-registration by the meter; the minus signs indicate under-registration.

The qualitative method of assessing the accuracy of a discharge measurement requires that the hydrographer consider all of the above items and their effect on the measurement accuracy. The front page of the discharge measurement note sheet has space for describing: (a) the cross section; (b) the flow; (c) the weather and (d) any other flow conditions that relate to the accuracy. These descriptions, along with the type of equipment, number of verticals, velocity measurement method and other measurement conditions should provide the basis for rating the measurement as excellent (2%), good (5%), fair (8%) or poor (over 8%).

For instance, a measurement might be rated as excellent (2%) if it has: (a) a smooth, firm and uniform cross section; (b) velocity that is smooth and evenly distributed; (c) equipment in good condition; (d) use of 2-point velocity measurement method and (e) good weather conditions (no wind or ice). On the other hand, if several of these factors make it difficult to accurately measure depth and/or velocity, the measurement might be rated fair (8%) or even poor (over 8%).

As stated previously, it is not possible to provide absolute guidelines for making the qualitative evaluation of accuracy. As a general rule, the accuracy of most discharge measurements will be about 5 per cent, or qualitatively a good measurement. This is sometimes used as the base-line accuracy, with accuracy upgraded to excellent when measuring conditions are significantly better than average, and accuracy downgraded to fair or poor when conditions are significantly worse than average. The qualitative accuracy evaluation is based on the hydrographer's judgement.

Table I.10.2. Registration errors, in percentage of stream velocity, caused by vertical motion of current meter

	Registration errors, in percentage of stream velocity						
Vertical motion ($m\ s^{-1}$) / *Stream velocity ($m\ s^{-1}$)*	*0.05*	*0.10*	*0.20*	*0.25*	*0.30*	*0.35*	*0.45*
	Price current meter (Suspended by a cable)						
0.15	– 2.0	+ 10	+ 36	+ 72	+ 120	+ 150	+ 210
0.30	– 3.0	– 1.0	+ 10	+ 24	+ 40	+ 50	+ 56
0.46	– 6.7	– 6.7	– 4.0	+ 1.3	+ 8.0	+ 25	+ 27
0.61	– 2.5	– 2.5	– 2.5	– 2.0	0	+ 4.0	+ 14.0
0.76	0	0	0	0	0	+ 0.8	+ 4.0
0.91	0	0	0	0	– 2.3	– 2.0	0
1.22	0	0	0	0	– 1.3	– 1.3	0
1.52	+ 0.4	+ 1.0	+ 0.6	0	– 0.2	0	+ 0.8
2.13	– 0.7	– 0.4	0	+ 0.1	– 0.4	– 0.7	– 0.4
3.05	– 0.5	– 0.3	0	0	– 0.3	– 0.7	– 1.3
	Vane-type current meter (Suspended by a rod)						
0.15	+ 4.0	+ 6.0	+ 20	+ 44	+ 72	+ 100	+ 160
0.30	+ 5.0	+ 10	+ 12	+ 10	+ 11	+ 15	+ 26
0.46	+ 3.3	+ 8.7	+ 10	+ 6.7	+ 3.3	+ 3.3	+ 10
0.61	+ 2.0	+ 6.5	+ 9.0	+ 9.5	+ 8.5	+ 6.0	+ 7.5
0.76	+ 2.0	+ 4.4	+ 6.4	+ 7.6	+ 8.0	+ 7.2	+ 6.4
0.91	– 1.7	+ 3.7	+ 5.3	+ 6.7	+ 7.3	+ 7.7	+ 6.7
1.22	+ 1.2	+ 0.8	+ 0.3	+ 1.0	+ 2.5	+ 3.8	+ 3.3
1.52	– 1.0	– 2.6	– 2.8	– 2.0	– 0.4	– 0.2	– 2.0
2.13	– 0.7	– 0.7	– 0.3	0	0	+ 0.3	– 0.4
	Ott current meter (Cosine rotor 8646-A, standard tailpiece without vertical stabilizer, and two-pin attachment to cable hanger)						
0.15	0	+ 6.0	+ 10	+ 20	+ 30	+ 44	+ 70
0.30	0	0	0	+ 4.0	+ 9.0	+ 15	+ 30
0.46	0	0	0	+ 1.3	+ 4.0	+ 7.3	+ 17
0.61	0	0	0	+ 0.5	+ 2.0	+ 4.5	+ 9.5
0.76	0	0	0	0	+ 1.6	+ 2.8	+ 6.4
0.91	0	0	0	+ 0.3	+ 1.0	+ 2.3	+ 6.0
1.22	0	0	+ 0.5	+ 1.0	+ 1.8	+ 2.5	+ 3.8
1.52	+ 0.4	+ 0.6	+ 0.4	+ 0.6	+ 1.0	+ 1.4	+ 2.0
2.13	0	0	0	0	+ 0.3	+ 0.7	+ 1.4

10.4 UNCERTAINTY OF ACOUSTIC DOPPLER CURRENT PROFILER DISCHARGE MEASUREMENTS

The Acoustic Doppler Current Profiler (ADCP) method of making discharge measurements was developed in the 1980s and has gone through numerous development stages. Today it has become a highly reliable method and is gradually replacing conventional instruments used for measuring stream flows in rivers, estuaries and tidal environments. See Chapter 6 of this Manual for a complete description of the ADCP method.

A limited amount of effort, however, has been devoted to the assessment of uncertainties associated with ADCP measurements. Development of uncertainty procedures is currently underway by various organizations such as ISO, using methodology defined in *Guide to the Expression of Uncertainty in Measurement* (1993). The WMO Commission for Hydrology (CHy), is likewise studying the uncertainty in flow measurements with an emphasis on ADCP measurements. In the United States, the South Florida Water Management District (SFWMD) started a program in the late 1990s to study the accuracy of ADCP measurements which is reported by González-Castro (2002). This has led to new programs to study the uncertainty of ADCP discharge measurements. Most recently, reports by Kim and others (2005) and González-Castro and Muste (2007) describe ADCP uncertainty analysis based on methodology of AIAA (1995) standards.

Because there is not yet an accepted international standard for computation of uncertainty of ADCP flow measurements, a detailed uncertainty analytical procedure is not presented herein. The following sections will give brief descriptions of the various ADCP measurement components and possible elemental error sources as described by González-Castro and Muste (2007).

10.4.1 Basic components of an Acoustic Doppler Current Profiler discharge measurement

A complete Acoustic Doppler Current Profiler (ADCP) discharge measurement is composed of four primary components as follows and as illustrated in Figure I.10.1:

(a) ADCP measured area;
(b) Unmeasured top area;
(c) Unmeasured bottom area, and
(d) Unmeasured left and right bank areas.

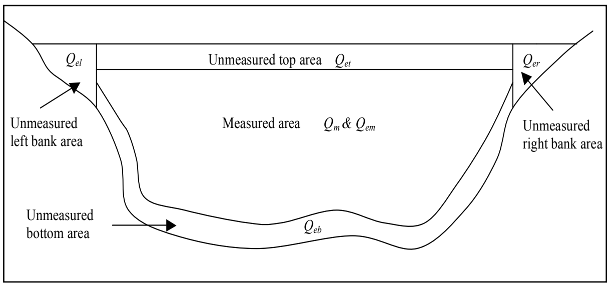

Figure I.10.1. Typical schematic of measurable and unmeasurable components of an Acoustic Doppler Current Profiler discharge measurement

ADCPs are not able to measure near solid boundaries or near free surfaces. Consequently the total discharge, Q_t, is the sum of:

(a) The discharge computed directly from the ADCP in the measurable area, Q_m;
(b) The discharge estimated in the measurable area when part of those data are missing, Q_{em};
(c) The discharge estimated for the free surface unmeasured top area, Q_{et};
(d) The discharge estimated for the unmeasured bottom area, Q_{eb}, and
(e) The discharge estimated for the unmeasured left and right bank areas, Q_{el} and Q_{er}.

$$Q_t = Q_m + Q_{em} + Q_{et} + Q_{eb} + Q_{el} + Q_{er} \qquad (10.18)$$

The largest and most significant subsection is, of course, the ADCP measured channel subsection. Some errors can be greatly reduced if factors such as moving bed, water temperature, salinity, cross-section choice and boat speed are carefully considered and accounted for, as described in previous sections of this report. Software is usually provided by the manufacturer that can be used to compute the ADCP instrument error for the measured subsection.

Errors for the extrapolated top, bottom and edge subsections will vary depending on the extrapolation methods and relative proportion of the total discharge represented in these subsections. Again, these errors can be kept to a minimum through proper choice of cross section and careful measurement of variables such as ADCP transducer depth and distances from each shore to the nearest ADCP section.

A study by Morlock (1996) concluded that ADCP discharge measurements can be used successfully for streamflow data collection under a variety of field conditions. In that report, 31 ADCP discharge measurements were compared to discharge ratings defined by conventional methods for the period over which the ADCP measurements were made. These comparisons showed that 25 ADCP measurements were within 5 per cent of the conventional measurements. Six of the ADCP measurements differed by more than 5 per cent, the maximum departure being 7.6 per cent.

The study by Morlock (1996) also concluded that ADCP discharge measurement error was indicated by the standard deviations of the ADCP discharge measurements. The standard deviations ranged from about 1 to 7 per cent of the measurement discharges. The estimated error of each ADCP discharge measurement also was computed from formulas derived by the manufacturer of ADCPs. The computations of estimated measurement error assume that ADCP instrument- and unmeasured subsection-extrapolation errors are the main source of measurement error. The standard deviations for most ADCP discharge measurements were higher than the estimated measurement errors, indicating that significant components of measurement error were not related to the instruments; errors of this nature include temporal variations of flow. As a result, measurement precision can be affected greatly by selection of a measurement location. Making ADCP measurements at locations where flow variations are minimized can improve measurement precision. Measurement precision also can be affected by instrument- and boat-operation factors.

10.4.2 Identification of Acoustic Doppler Current Profiler elemental measurement errors

There are a number of error sources that contribute to the overall uncertainty of an ADCP discharge measurement. Twenty such error sources, defined as elemental measurement errors, are reported by González-Castro and Muste (2007) and are listed in the following paragraphs. In most cases, these error sources are quoted verbatim, with minor wording changes, from their report. These are the inherent errors that affect the uncertainty of the ADCP measurement even when good measuring practices are observed. Good measuring practices should include things like correct ADCP draft setting, proper ADCP mounting, good choice of data collection mode, proper detection of moving bed, good boat navigation, small end sub-sections, adequate depth and other factors. Each of the elemental measurement error sources are described briefly in the following paragraphs.

Spatial resolution

The geometrical arrangement for a multi-component ADCP does not sample velocity components in the same location (point) in a water column. Because this is not practical, ADCPs employ a monostatic diverging multi-beam geometry and estimate the water velocities under the assumption that the flow is horizontally homogeneous. The spatial averaging of ADCPs filter some of the high frequency features of the flow and limits their ability to measure turbulence characteristics. This tends to bias the estimates of mean flow in three dimensional flows.

Doppler Noise

Noise in the Doppler-shift measured by broadband ADCPs (BB-ADCPs) comprises both noise added in the measurement volume to the return signal and noise added to the first-pulse return signal by reflections from scatterers in the vicinity of the path of the first return signal as it intercepts the second pulse. The random error in Doppler shift measurement by BB-ADCPs is inversely proportional to the pulse length, the transmit frequency and the signal-to-noise-ratio and the beam angle (Simpson, 2001). BB-ADCPs resolve the Doppler shift by two or more short identical pulses in phase with each other (pulse-to-pulse coherent) and adjust the time between pulses adjusted to minimize ping-to-ping interference. The uncertainty of the radial velocities measured by pulse-to-pulse coherent BB-ADCPs is directly proportional to the acoustic wave length and the Doppler bandwidth. It is also affected by environmental variables such as scatterer's residence time in the measuring volume, turbulence within the sample volume and acceleration during the averaging period.

Velocity ambiguity error

BB-ADCPs determine velocity by measuring the phase-angle differences between pulse pairs. These measurements are subject to velocity ambiguity errors because the reference yardstick is actually half a cycle at the transmitted frequency. Thus, if the velocity exceeds the expected velocity range, a corresponding phase s hift outside of the expected 180° to 180° range occurs.

Side-lobe interference error

Most modern transducers have parasitic side lobes that form a 30-40° angle with the main acoustic beam (Simpson, 2001). When the parasitic side lobes hit a solid boundary before the main beam, the resulting reflections contaminate the reflections from the main lobe resulting in so-called side-lobe interference. Because the signal return from the solid boundary is stronger than that from scatterers in the water column, the side lobe energy travels the shorter path directly to the surface and adds the velocity of the boundary to the water velocities measured along the mean lobe. ADCPs with 20°-beam angles lose reception from echoes coming from the lower 6 per cent of the flow depth.

Temporal resolution error

ADCPs typically sample data in each ping at equally spaced time intervals. The sampling time interval has only a small effect in the estimates of the mean velocities, as long as the sampling time is long enough. However, in measuring time-resolved flow features (for example autocorrelations), the choice of the time interval should be set to as high a frequency sampling as possible. Currently, the frequencies captured by ADCPs are limited by the ping sampling frequency, cell size and noise level of the data collection mode, which constraints their ability to capturing only relatively high frequency components of the turbulence.

Sound speed error

Errors in the speed of sound propagate as errors in the range gating for both geometric cell mapping, Doppler shift estimation, and ultimately in the computation of the water and boat velocities. ADCPs calculate range gating and Doppler shift assuming that salinity and temperature are constant throughout the water column, so they cannot account for refraction of the acoustic waves in water bodies stratified due to gradients in salinity or temperature or both.

Beam-angle error

This error is the result of manufacturing imperfections, so it is bound by fabrication tolerances and not by probe design features. This error has been substantially reduced in ADCPs built after 1993 by storing the ADCP-specific matrix into their firmware to more accurately transform velocities from radial to instrument coordinates. The manufacturing tolerances in measuring these angles, however, still contribute to the total systematic uncertainty.

Boat speed error

Accelerations produced by fast boat maneuvering may force the ADCP compass to swing out of its vertical position and induce compass errors (Gaeuman & Jacobson, 2005). In addition, the faster the boat transects a channel, the fewer samples it will collect and consequently the larger the precision error. It is also been known that high boat-to-flow velocity ratios result in systematic heading errors, Global Positioning System (GPS) and bottom tracking.

Sampling time error

Sampling time errors occur when the sampling time is too short to provide an unbiased variance estimate of the mean flow. This error can be minimized by collecting data over time spans that cover enough large-scale flow structures. This uncertainty typically affects more the estimation of turbulence-averaged mean velocities than transect mean discharge because, in the latter, time variability is somewhat compensated by spatial variability. Uncertainties due to limited sampling bias the estimates of discharge through the top and bottom layers because the velocity extrapolation algorithms rely on turbulence-averaged, boundary-layer velocity distribution models. The magnitude of this uncertainty depends on whether the data are collected from a moving boat or at fixed channel span-wise locations. And whether data are collected in single- or multi-ping ensembles, and more importantly, whether the pings per ensemble are chosen to properly obtain physically meaningful averages.

Near-transducer error

There are two prominent sources of error in measuring velocities near ADCP transducers. These are ringing and ADCP-induced flow disturbance. Ringing is due to the resonance of the transducers after transmitting an acoustic pulse. Ringing error depends upon the ADCP and its signal-processing algorithm. Flow disturbances induced by ADCPs introduce bias in the magnitude of the velocities near the transducers. The magnitude of this bias is a function of the mean flow, and decreases inversely with distance to the ADCP transducers. These errors are referred to as near-transducer errors since they both bias the velocities measured near the ADCP.

Reference boat velocity error

ADCPs measure velocities in the water column relative to the ADCP, so the velocity of the ADCP relative to the channel bed must be measured to calculate the water velocity. Acoustical bottom tracking and boat GPS tracking are the current options for measuring boat velocities. Moving bed, bed sediment transport, uneven bed, high suspended-sediment concentration, and boat operation induce bias in the boat velocity measurements by acoustical bottom tracking. High boat-to-water velocity ratios, GPS positioning errors, GPS frequency position update, and poor reception bias boat velocity measurements by GPS tracking.

Depth error

This error is associated with bottom-tracking profiling. The transmit time for the bottom-tracking profiling is longer than that used for the water profiling and the echo is processed in a different way. Bias errors in depth measurement can be introduced by: (a) errors in measuring the distance of the ADCP-transducers to the free surface or draft; (b) sampling errors due to limitations of the acoustic beams and bin size; (c) error in the speed of sound and (d) random errors in the reflected echoes from the bed. Uncertainties in depth errors depend upon uncertainties in estimating transmit pulse length, blank beyond transmit, cell size, average measured depth and depth of the ADCPs transducers with respect to the free surface.

Cell mapping error

The position of the top first cell is determined by range gating and the sound speed, transmit pulse length, blank beyond transmit, bin size, transducer beam angle, transmit frequency, and, when the data is collected using water mode 11, the lag between transmit pulses or correlation lag. Thus, the ADCP configuration settings may bias the cell mapping. In flows with salinity or temperature stratification, ADCPs map cells inaccurately.

Rotation (pitch, roll, and heading) error

Errors in pitch and roll affect the water velocity estimates through the transformations from radial to instrument and ship coordinates. Heading errors propagate through the transformation from ship to earth coordinates. Rotation errors are directly related to the configuration of the instrument. ADCPs are equipped with internal tilt sensors to measure pitch and roll and an internal compass to measure heading. The fluxgate compass and gyrocompass are conventional external instruments used with ADCPs for measuring heading. Compass errors due to incorrect magnetic declination have no effect on ADCP transect measurements with bottom tracking. However, errors in compass calibration bias the discharge estimates from boat velocity measurements by GPS tracking.

Timing errors

Timing is needed to establish the boat velocity and cell gating to measure Doppler shift, hence clock errors propagate as errors in time measurements, which will in turn propagate as uncertainties in cell mapping, velocity calculations and discharge computations.

Edge error

The distance to shore from the first and last points of ADCP transects must be measured for estimating edge discharges by an independent measurement device. The uncertainty in edge discharge estimates depends upon the uncertainty of the device that measures the distances to banks, the environmental and operational conditions when the velocity at the transect edges is measured, the shape of the edges and the model for computing edge discharge.

Vertical-velocity distribution error

The effect of this source of error strongly depends on the method used for collecting the ADCP data (either moving or fixed boat) and the model used for extrapolating the velocity in the area not directly measured by the ADCP. The random uncertainty on velocity profiles from long sampling records collected at fixed locations is substantially smaller than in profiles estimated from short records. This error must be quantified as the deviation of short-time velocity profiles with long-term profiles. It is recommended that the power-law with free parameters as reference velocity distribution be used, with the near-transducer bias removed. Barenblatt (1993) has shown that the power-law distribution follows incomplete similarity scaling laws in turbulent boundary layers, in which both parameters of the distribution vary with the Reynolds number. However, it is expected that in open-channel flow, the parameters will vary the pressure and acceleration gradients and geometric factors as well.

Discharge model error

The optimal model should use actual turbulence averaged point estimates of the water velocity, with high spatial resolution. In ADCP measurements, too many pings induce correlated errors, and too few, resolution errors. The spatial averaging of ADCP measurements induces correlated errors between velocities measured in contiguous cells. Estimates of these errors are available only for contiguous cells in the same ping.

Finite summation error

The error in flow discharge integration due to finite summation in ADCP measurements is similar to the finite summation error in measurements with conventional mechanical current meters (Pelletier, 1988). The error is a direct function of the spatio-temporal averaging strategies specified by the cell size, number of pings per ensemble, sampling frequency and water and bottom tracking modes.

Measuring environment and operational errors

This error group lumps errors due to poor use of good ADCP-measurement practices as those cited in the literature. Secondary currents, hydraulic structures, small channel aspect ratios and other situations that might induce considerable three dimensional characteristics of the channel flow, result in measurement conditions that may violate the assumptions involved in the ADCP operational principles and good measurement practices. In addition, the bed roughness (gravel, sandy) and the level of turbulence are factors that affect the velocity distribution models and the sampling procedures used for estimating the velocity and eventually discharge estimates. This error group depends so much on the site and operation mode that it is difficult to assess. However, it is listed here for completeness.

10.4.3 **Summary**

In summary, ADCP discharge measurements are becoming more and more widespread and are considered an excellent and accurate means of obtaining flow data. However, the determination of uncertainty for these measurements is an extremely complex procedure. Many factors and error sources enter into any mathematical derivation of ADCP uncertainty. The preceding sections list and describe some of error sources that must be considered in an uncertainty analysis. Some of these sources actually lump uncertainties from several elemental sources of different nature (data acquisition, data processing, data reduction, operational) and different type (bias or precision). For example, the noise error, associated with the measurement of the water velocity, lumps the effect of noise in Doppler-shift measurement, self-noise, finite bin size and non-uniform signal absorption. Work is still needed to be sure all error sources are considered and to define methods of quantifying each error source.

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